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CARBONATION SERVICE LIFE PREDICTION OF EXISTING CONCRETE VIADUCT/BRIDGE USING TIME-DEPENDENT RELIABILITY ANALYSIS

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Key words: bridge, carbonation life, probability, reliability, viaduct.

ABSTRACT

This paper examines the predicted carbonation life of an existing concrete viaduct/bridge in the atmospheric environment based on probability and reliability indices. The probability is dependent upon the carbonation rate, carbonation remainder, concrete quality, and concrete cover. The carbonation life is defined as the service life of reinforced concrete (RC) or prestressed concrete (PC) structure at the beginning of time of steel corrosion when concrete cover surfers from carbonation and loses the protection function to steel. The carbonation life is equivalent to the initiation time of corrosion of reinforcement. Both the Chorng-ching viaduct and Wann-fwu bridge were offered as illustrative examples for verifying the analytical method and carbonation life prediction. The predicted carbonation life results for the Chorng-ching viaduct and Wann-fwu bridge are 55 and 55, 40 and 17, and 18 and 0 years at β_c = 0, 0.5, and 1.25 reliability indices, respectively. The results show that both structures are serviceable and reliable comparing the initiation time calculated using Fick's second law, Guirguis, and AJMF methods. The results can provide a basis for repair, strengthening, and demolition of existing RC and PC viaducts or bridges.

I. INTRODUCTION

Concrete carbonation is the premise condition for steel corrosion in concrete under general atmospheric environment. Carbonation depth of concrete is not only of randomization but also of random process. The carbonation problem of concrete structures may be suitably studied using the random process, probability, and reliability for predicting carbonation life of the concrete structures. The study of a practical prediction model for carbonation is significantly important in the durability analysis and service life evaluation of concrete structures.

Fagerlund [9] predicted the service life of structures using the relation between serviceability and time. This method can be used in carbonation, sulphate attack, and cement-aggregate reaction cases. Sjostrom [28] considered that the service life prediction for building and construction materials is a true example of multi-disciplinary work. A complete prediction model must contain a mathematical model that documents the observed degradation and enables acceleration factor calculation. Sjostrom described the methodologies for service life prediction. Fagerlund [10] pointed out that a service life calculation requires extensive data concerning the functional requirements, environment, deterioration mechanism and material. He estimated the service life of structures using the relationship between serviceability and time. Martin [20] applied the reliability theory and life test analysis techniques to predict the service life of building materials from accelerated aging test results. Pommersheim and Clifton [27] examined the basis for making concrete service-life prediction based on accelerated testing and mathematical modeling of the factors controlling concrete durability. Masters and Brandt [21] outlined a systematic method for the service life prediction of building materials and components, including the identifying the needed information, test selection or development, data interpretation, and results reporting. Clifton [6] summarized methods for predicting the service life of construction materials including (a) estimates based on experience, (b) deductions from the performance of similar materials, (c) accelerated testing, (d) mathematical modeling based on the chemistry and physics of the degradation processes, and (e) the application of reliability and stochastic concepts. Niu *et al*. [24] constructed a mathematical model that presented carbonation depth with concrete strength and a carbonation depth standard error model that changed with time. They

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estimated the carbonation service life of reinforced concrete (RC) members using the degree of carbonation reliability versus time based on the carbonation limit state equation. Cheung and Kyle [5] described the development and application of a reliability-based system for the service life prediction and management of maintenance and repair procedures for RC structures. Ng and Moses [22] used the time-dependent reliability theory ascribed to a Markov chain model generalization for predicting bridge service life. Liang *et al*. [15] employed a semi-empirical formula derived from Fick's second law of diffusion to establish the relationship between carbonation and time to predict the service life of existing RC bridges under carbonation-induced corrosion. Niu *et al*. [25] applied the fuzzy and reliability theory based on a random carbonation depth model and the conditions for initial steel bar corrosion in concrete to predict the carbonation lifetime of concrete structures. Liang *et al*. [16] made service life predictions for existing RC bridges in a chloride laden environment based on a mathematical model applied to calculate each initiation time, depassivation time and propagation time. They also gave a summary of the service life prediction methods. Although much research has been done on service life prediction for concrete or RC structures, little work has been done on service life prediction for existing RC or PC viaduct/bridge due to concrete carbonation.

The objective of this study was to determine the carbonation life of existing concrete viaduct/bridge. The carbonation life of existing concrete viaduct/bridge was predicted using the probability and reliability index analytical method. The results from this study may help determine the need for repair, strengthening or demolition of existing RC or PC viaducts and bridges.

II. LITERATURE SURVEY FOR CARBONATION SERVICE LIFE ANALYSIS

Fick's first law of linear diffusion was applied to state the carbonation depth as a function of time. Based on Fick's first law of linear diffusion, Kropp [14] derived a formula that identified the CO_2 diffusion controlled carbonation process in concrete

$$x(t) = \sqrt{\frac{2D(C_1 - C_2)}{a} t} \tag{1}$$

where $x(t)$ is the carbonation depth at time (m), t is the time(s), D is the diffusion coefficient for CO_2 through carbonated concrete (m^3/g), C_1 is the external carbon dioxide concentration (g/m^3), C_2 is the carbon dioxide concentration at the carbonation front (g/m^3), and a is the required amount of CO_2 for the carbonation of the alkaline compounds contained in a unit volume of concrete (g/m^3).

If all constant parameters in Eq. (1) are combined into one single constant k, Eq. (1) will lead to the well-known equation

$$x(t) = k\sqrt{t} \tag{2}$$

where k is the rate of carbonation.

Now consider the Fick second law of linear diffusion with initial and boundary conditions [17]

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} \tag{3a}$$

$$C(x, 0) = C_i \tag{3b}$$

$$C(0, t) = C_s \tag{3c}$$

$$C(x \to \infty, t) = C_i \tag{3d}$$

where $C(x, t)$ is the CO_2 concentration at space x and time t, C_i and C_s are the initial and surface CO_2 concentrations in the concrete and on the concrete structure surface, respectively.

Taking the Laplace transform method, the analytical solution [17] for Eq. (3) is

$$C(x,t) = C_i + (C_s - C_i)\,erfc\left(\frac{x}{2\sqrt{Dt}}\right) \tag{4}$$

where $erfc(.)$ is the complimentary error function.

Eq. (4) may be written as Eq. (2), where $k = 2\sqrt{D}erfc^{-1}\left(\frac{C - C_i}{C_s - C_i}\right)$. About the rate of carbonation, k, in Eq. (2), according to the theoretical carbonation and analysis model with adequate field and weather data, Niu *et al*. [25] and Niu [26] provided that

$$k = 2.56 k_{mc} k_j k_{CO_2} k_p k_s \sqrt[4]{T}\,(1 - RH)\,RH\left(\frac{57.94}{f_{cu}} m_c - 0.76\right) \tag{5}$$

where k_{mc} is an uncertainty coefficient, which is a random variable, in the calculation model. The main reflection is the difference between the concrete carbonation calculation model results and the results from practical measurements. f_{cu}(= $f_c'(cube)$) is the compressive strength of cube concrete (MPa = N/mm^2), in which $f_c'(cylinder)$ = 0.85 $f_c'(cube)$ = 1.10 f_c'(prism) [12], m_c is the ratio of the mean and standard compressive strength value for cube concrete. RH is the relative humidity (%), T is the environmental temperature (K), k_j = 1.4 for corner, k_j = 1.0 for non-corner, k_p = 1.2 for grouting surface, k_s = 1.0 for concrete subjected to compression, k_s = 1.1 for concrete suffering from tension, and k_{CO_2} is the influence coefficient of the CO_2 concentration. k_{CO_2} = 1.1~1.4 for the

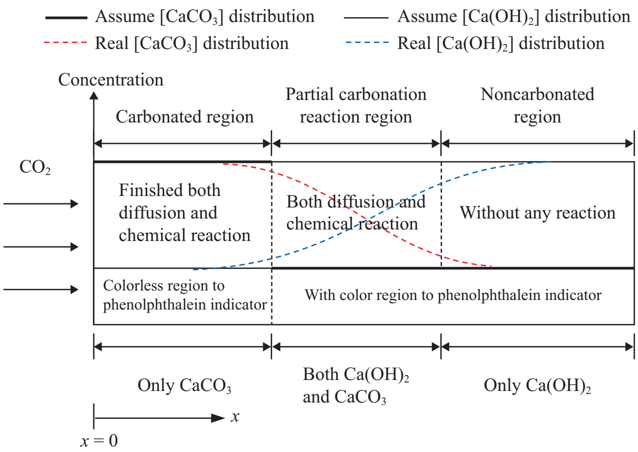

Fig. 1. Formed region and its inner reaction behaviour during the concrete carbonation process [4].

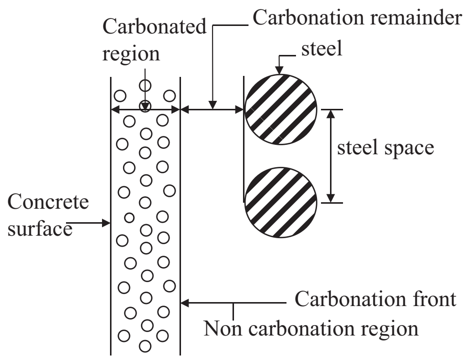

Fig. 2. Schematic diagram of carbonation remainder. Assume [Ca(OH)₂] distribution.

exterior environment, or using $k_{CO_2} = \sqrt{\dfrac{C_s}{0.03}}$ for computation, in which C_s is the CO_2 concentration on the surface of concrete [23].

Under general atmospheric environmental conditions, the commencement time for steel corrosion in concrete is straightforwardly considered as the required carbonation time for the concrete cover. Nevertheless, the testing and engineering survey results indicate that when the carbonation depth tested using the phenolphthalein indicator arrives at certain lengths away from the steel surface, the steel is beginning to corrode. This is because the phenolphthalein indicator shows color under the presence of $Ca(OH)_2$ in concrete. Thus, the phenolphthalein indicator can only detect the completely carbonated concrete with its limited boundary. In the concrete carbonation process, the completely carbonated concrete front exhibits a partial carbonation region. When the environmental relative humidity (RH) is low (RH < 70%), the partial carbonation region plays an important role. Chang and Chen [4] applied phenolphthalein indicator, thermal gravimetric analysis, X-ray diffraction analysis, and Fourier transformation infrared spectroscopy in experimentally investigating concrete carbonation depth. They pointed out that the concrete carbonation region can be divided into three regions: carbonated, partially carbonated and noncarbonated (see Fig. 1). The amount of $CaCO_3$ decreases when the concrete depth increases. The amount of $Ca(OH)_2$ increases when the concrete depth decreases. In this carbonation situation, the pH value of the concrete gradually increases from surface to the concrete interior. The fully carbonated region is identified to have a pH < 9. The pH value in the partly carbonated region is 9 < pH < 11.5. The noncarbonated region has pH > 11.5. From the point of view of the carbonation influence on steel corrosion in concrete, the steel is located in a passive status and is not corroded when the steel is placed in the pH > 11.5 region. The steel is located in depassive status and corrosion occurs when the steel is placed in the pH < 11.5 region. The steel begins to corrode when the pH value of the concrete drops to the pH_0 = 11.5 [30] value. The result obtained from the field engineering survey indicates that the steel commences corrosion corresponding to the pH_0 value differs due to the differences in environmental conditions. At present, the study work to pH_0 value is developed a little. The determination of pH_0 value is very difficult to treat by using the traditional carbonation-steel corrosion mechanism. However, the concrete carbonation data and start of steel corrosion data requires a great deal of engineering measurement to obtain the interval length from the complete carbonation front and pH = pH_0. This interval length is defined as the carbonation remainder (see Fig. 2 and [23]). Establishing the onset of steel corrosion in concrete is determined by the carbonation remainder calculation model. The carbonation remainder value is restricted by the partially carbonated length and the rate of steel depassivation. The larger the length of partial carbonation, the greater the carbonation remainder is. At the beginning of carbonation, the value of the partially carbonated region is small and the gradient of variation in pH value is large. The partially carbonated region increases when the carbonation increases time. When the partially carbonated region length reaches a certain value, it will not increase. Because the concrete carbonation problem is dependent on the RH, the onset time of steel corrosion in concrete, the compressive strength and concrete cover thickness, the carbonation remainder, x_0, can be computed or predicted using an empirical formula through experimental study or field tests based on a great deal of engineering measurement data. Based on the engineering measurement data, Xu *et al.* [30] obtained the carbonation remainder calculation model

$$x_0(t) = \frac{166(-RH^2 + 1.4RH - 0.4)(c-5)}{f_{cu}}\sqrt{\frac{t}{t_0}},\ t \le t_0 \quad (6)$$

and

$$x_0 = \frac{166(-RH^2 + 1.4RH - 0.4)(c-5)}{f_{cu}},\ t > t_0 \quad (7)$$

where x_0 is the carbonation remainder (mm), c is the concrete cover (mm), to is the steel corrosion onset time (yr) when the concrete pH value drops to pH = pH_0 = 11.5, and t is the structure service life (yr). It is worthy to point out that the carbonation problem of concrete or RC structures under an environment of RH = 100% can be neglected. This means that the porosity in concrete is full of water when it is exposed in environment with RH = 100%.

Eqs. (6) and (7) indicate that the carbonation remainder at the beginning of steel corrosion in concrete is related to the concrete structure or component and environment (RH, CO_2 content), the compressive strength and concrete cover thickness. The larger RH environment value corresponds to the slow carbonation rate while early steel corrosion in concrete corresponds to a larger carbonation remainder value. The CO_2 and O_2 coefficients in dense concrete are slow while the carbonation remainder is large. The greater the concrete cover, the more the carbonation remainder increases. However, when the concrete cover is over a threshold, the carbonation remainder does not increase further. Based on the same engineering measurement data, as Xu *et al*. [31], Niu [23] used simulations to obtain a more practical formula. That is Eqs. (6) and (7) can be replaced by the following formula

$$x_0 = 4.86k_{mo}(-RH^2 + 1.5RH - 0.45)(c-5)(\ln f_{cu} - 2.30),$$
$$c > 5\ \text{mm} \tag{8}$$

where k_{mo} is an uncertainty coefficient due to the calculation model of carbonation remainder. It is noteworthy to point out that we take c = 50 mm when c > 50 mm.

The onset time for steel corrosion in concrete is often defined as the time the necessary carbonation depth is reached at the steel surface. When the pH value reaches 11.5, i.e., the boundary between the partially carbonated and non-carbonated regions presents steel corrosion in concrete. Thus, the concrete carbonation life can be defined as life termination when the steel in concrete is at the beginning of corrosion. The criteria for concrete carbonation life [26] can be expressed as

$$\Omega_c = \{x(t) + x_0 - c \geq 0\} \tag{9}$$

Since the concrete strength grade is a random variant, the carbonation rate (see Eq. (5)) is also a random variant. Zhao [32] investigated and pointed out that the carbonation rate obeys a normal distribution. The one-dimensional probability density function of the carbonation rate is

$$f(k) = \frac{1}{\sqrt{2\pi}\sigma_k}\exp\left[-\frac{1}{2}\left(\frac{k-\mu_k}{\sigma_k}\right)^2\right] \tag{10}$$

where μ_k and σ_k are the mean and standard deviations of the carbonation rate, respectively.

According to Eq. (5), we may have

$$\mu_k = 2.56\mu_{k_{mc}} k_j k_{CO_2} k_p k_s \sqrt[4]{T}(1-RH)RH\left(\frac{57.94}{\mu_{f_{cu}}}m_c - 0.76\right) \tag{11}$$

and using error propagation formula, we may have

$$\sigma_k = \left[\left(\left.\frac{\partial k}{\partial k_{mc}}\right|_{\mu}\right)^2 \sigma_{k_{mc}}{}^2 + \left(\left.\frac{\partial k}{\partial f_{cu}}\right|_{\mu}\right)^2 \sigma^2 f_{cu}\right]^{\frac{1}{2}} \tag{12}$$

where $\mu_{k_{mc}}$ and $\sigma_{k_{mc}}$ are the mean and standard deviation of the uncertainty coefficient, respectively. μ means that the partial derivative takes the value at the carbonation rate mean. $\mu_{f_{cu}}$ and $\sigma_{f_{cu}}$ are the mean and standard deviation of the compressive strength of cube concrete, respectively. It is worthy to point out that both k_{m_c} and f_{cu} are considered as random variables due to uncertainty in Eq. (11) while both RH and m_c are not regarded as the random variables due to certainty in Eq. (11).

The concrete cover obeys a normal distribution [32]. Its probability density function is

$$f(c) = \frac{1}{\sqrt{2\pi}\sigma_c}\exp\left[-\frac{1}{2}\left(\frac{c-\mu_c}{\sigma_c}\right)^2\right] \tag{13}$$

where μ_c and σ_c are the mean and standard deviation for the concrete cover, respectively.

The carbonation remainder is suitably obeyed normal distribution [26]. The probability density function of carbonation remainder can be represented as

$$f(x_0) = \frac{1}{\sqrt{2\pi}\sigma_{x_0}}\exp\left[-\frac{1}{2}\left(\frac{x_0-\mu_{x_0}}{\sigma_{x_0}}\right)^2\right] \tag{14}$$

where μ_{x_0} and σ_{x_0} are the mean and standard normal distribution for the carbonation remainder, respectively.

According to Eq. (8) and Ref. [23] we may have

$$\mu_{x_0} = \mu_{k_{m_0}} 4.86(-RH^2 + 1.5RH - 0.45)(\mu_c - 5)(\ln \mu_{f_{cu}} - 2.3) \tag{15}$$

and employing error propagation formula, we obtain

$$\sigma_{x_0} = \left[\left(\left.\frac{\partial x_0}{\partial k_{m_0}}\right|_{\mu}\right)^2 \sigma_{k_{m_0}}{}^2 + \left(\left.\frac{\partial x_0}{\partial f_{cu}}\right|_{\mu}\right)^2 \sigma_{f_{cu}} + \left(\left.\frac{\partial x_0}{\partial c}\right|_{\mu}\right)^2 \sigma^2{}_c\right]^{\frac{1}{2}} \tag{16}$$

where $\mu_{k_{m0}}$ and $\sigma_{k_{m0}}$ are the mean and standard deviation for the uncertainty coefficient, k_{m0}, respectively. $\mu_{f_{cu}}$ and $\sigma_{f_{cu}}$ are the mean and standard deviation for cube concrete compressive strength, respectively.

The one-dimensional time-dependent probability density function for the concrete carbonation depth [26] can be expressed in terms of

$$f_x(x,t) = \frac{1}{\sqrt{2\pi}\sigma_x(t)} \exp\left[-\frac{1}{2}\left(\frac{x-\mu_x(t)}{\sigma_x(t)}\right)^2\right] \quad (17)$$

where t is the carbonation time (yr) estimated using Eq. (2). The carbonation depth for calculating the carbonation time concludes the fully and partially carbonated regions. $\mu_x(t)$ and $\sigma_x(t)$ are the mean and standard deviation functions of concrete carbonation depth, respectively. Based on Eq. (17), they can be represented as

$$\mu_x(t) = \mu_k \sqrt{t} \quad (18)$$

and

$$\sigma_x(t) = \sigma_k \sqrt{t} \quad (19)$$

The time-dependent durability failure of concrete carbonation, i.e., the limit state equation for the onset of steel corrosion in concrete, can be described by Eq. (2) as

$$z(t) = x(t) + x_0 - c = k\sqrt{t} + x_0 - c = 0 \quad (20)$$

From Eq. (20), the onset time for steel corrosion in concrete can be calculated using

$$t_0 = \left(\frac{c - x_0}{k}\right)^2 \quad (21)$$

where t_0 is a random variable.

The time-dependent probability for corrosion onset in concrete may be expressed as [23]

$$p_{fc}(t) = p\left\{k\sqrt{t} + x_0 - c < 0\right\} \quad (22)$$

The time-dependent probability that corrosion may not occur is described as [23, 24]

$$p_{DC}(t) = p\left\{k\sqrt{t} + x_0 - c > 0\right\} \quad (23)$$

The corresponding time-dependent reliability index can be expressed as

$$\beta_c = -\Phi^{-1}(p_{DC}(t)) \quad (24)$$

where $\Phi^{-1}(\cdot)$ is the inverse function of the standard normal distribution function.

Herein, we should define the degree of concrete carbonation durability as: Under normal use and concrete structure maintenance, the probability for structural durability failure within service is nil. The corresponding failure criterion of concrete carbonation durability is called concrete carbonation durability. Obviously, Eq. (22) is the degree of concrete carbonation durability.

Table 1. Overall structural conditions of Chorng-ching viaduct.

Item	Description
Date of completed construction	This viaduct was completed in October 1971.
Viaduct site	This viaduct locates the southern of Chorng-ching road, strides over the Ting-jou road, and connects the Jong-jeng bridge, which is stridden over the Shin-diann stream, to the Yeong-her city.
Simply structural introduction	This viaduct has 25 span. Besides Piers P3 and P4 have 30 m and stride over the Ting-jou road. The others have 15 m. Total length is 390 m. Net width is 14 m. The southern and northern guide passages have 61.4 m and 71.7 m, respectively. The superstructure is prestressed box beam. Piers P3 and P4 are structural type of cantilever girder. The others are simple supports. The substructure are the doorframe piers with two columns of steel-reinforced concrete. The foundation types with piles are located at abutment A1 and piers from P1 to P4. The others are direct foundations.

Table 2. Overall structural conditions of Wann-fwu bridge.

Item	Description
Date of completed construction Bridge site	This bridge was completed in May 1987. This bridge locates Shin-guang road, strides over the Jiing-meei stream road, and connects the Muh-juh road and the Shen-keng district.
Simply structural introduction	This bridge has 5 span. Total length is of 188 m. Net width is 25 m. The superstructure is the steel box beam of prestressed concrete. The structural type is closely spread over simple support. The substructure are the doorframe piers with two columns of steel-reinforced concrete. The abutment is cantilever. The foundations are reverse circulation concrete.

Table 3. Testing data of Chorng-ching viaduct.

Test poing No.	Testing points	Compressive strength (kgf/cm^2)	Design strength (kgf/cm^2)	Concrete cover (mm)	Carbonation depth (mm)
A	A1 (Abutment)	248	210	50	22
B	P23 (Left pier)	287	210	50	30
C	P3 (Right pier)	196	210	50	33
D	P24 (Right pier)	205	210	50	38
E	Retaining wall (terminal guide passage) (right)	241	210	50	22
F	P24 (Left pier)	143	210	50	30
G	Retaining wall (terminal guide passage) (left)	225	210	50	40
H	Retaining wall (guide passage) (right)	321	210	50	31
I	Retaining wall (guide passage) (left)	291	210	50	18
J	P23 (Right pier)	287	210	50	26
K*	G6S4 (Girder)	361	350	25	28
L	G3S4 (Girder)	415	350	25	15
M	G10S4 (Girder)	269	350	25	17
N	G14S24 (Girder)	622	350	50	11
O	G1S4 (Girder)	415	350	50	8
P	G7S4 (Girder)	472	350	25	15
Q	G11S4(2) (Girder)	562	350	25	0
R	G2S4 (Girder)	414	350	25	0
S	G14S23 (Girder)	474	350	50	38
T	G1S23 (Girder)	514	350	50	20
Average		350.4	280	42.5	22.1

Remark: 1. A: Abutment; G: Girder; P: Pier; S: Slab.
2. * The carbonation depth of cored sample has surpassed concrete cover.

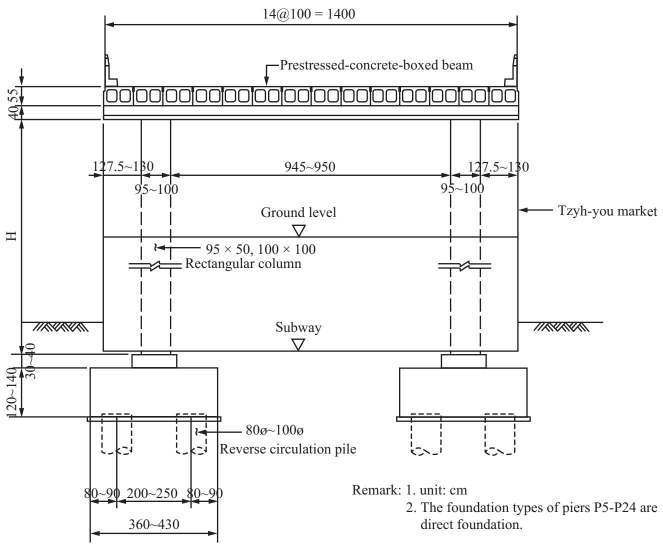

Fig. 3. Schematic diagram of cross-section of Chorng-ching viaduct [18].

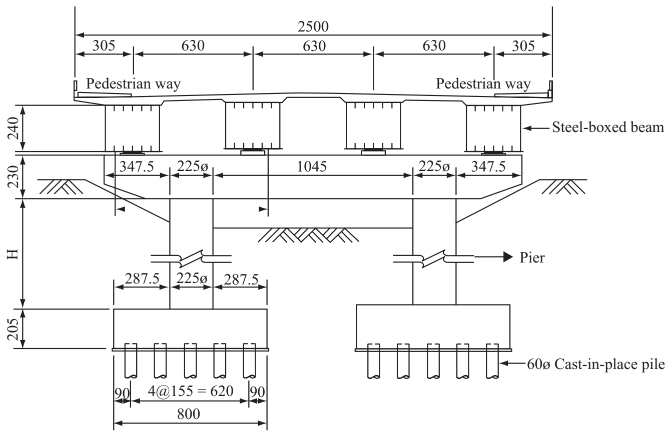

Fig. 4. Schematic diagram of cross-section of Wann-fwu bridge [19].

III. ILLUSTRATIVE EXAMPLES

The safety evaluation for an existing viaduct or bridge is important work for any infrastructure system. Rules and systems for safe evaluation have been established in many countries. In this paper, the method for carbonation life prediction will be employed to evaluate the carbonation life of the existing Chorng-ching viaduct and Wann-fwu bridge in Taipei. Tables l and 2 are the overall structural conditions of the existing Chorng-ching viaduct and Wann-fwu bridge, respectively. Figs. 3 [18] and 4 [19] are the partial cross-section of the existing Chorng-ching viaduct and Wann-fwu bridge, respectively. Tables 3 [18] and 4 [19] show the compressive strength [7], design strength, concrete cover and carbonation depth of the existing Chorng-ching viaduct and Wann-fwu bridge, respectively. According to Tables 3 and 4, and putting $\mu_{k_{mc}} = 0.996$, $k_j = 1.0$, $k_{CO_2} = 2.0$, $k_p = 1.2$, $k_s = 1.0$ [25, 26],

Table 4. Testing data of Wann-fwu bridge.

Test poing No.	Testing points	Compressive strength (kgf/cm^2)	Design strength (kgf/cm^2)	Concrete cover (mm)	Carbonation depth (mm)
A	A1-P1 (Bridge deck)	237	240	40	0
B	P1-P2 (Bridge deck)	484	240	40	0
C	P2-P3 (Bridge deck)	353	240	40	0
D	P3-P4 (Bridge deck)	310	240	40	20
E	P4-A2 (Bridge deck)	354	240	40	0
F*	A1-P1 (Bridge deck)	380	240	40	120
G*	P1-P2 (Bridge deck)	584	240	40	140
H	P2-P3 (Bridge deck)	660	240	40	34
I	P3-P4 (Bridge deck)	313	240	40	0
J	P4-A2 (Bridge deck)	310	240	40	0
K	S1-S1 (side)	246	240	25	24
L	P1 (Cap beam) (rear)	560	280	50	0
M	P1 (Cap beam)	374	280	50	4
N	P2 (Cap beam)	351	280	50	28
O	P2 (Cap beam)	307	280	50	8
P	P2 (Cap beam)	305	280	50	18
Q	P3 (right)	411	280	50	0
R*	P4 (Cap beam)	352	280	50	90
S	P4 (Left)	374	280	50	18
T	Guide Road (Retaining wall) (1)	450	280	50	11
U	Guide Road (Retaining wall) (2)	324	280	50	16
	Average	350.4	382.81	44.04	25.28

Remark: 1. A: Abutment; G: Girder; P: Pier; S: Slab.
2. * The Carbonation depth of cored sample has surpassed concrete cover.

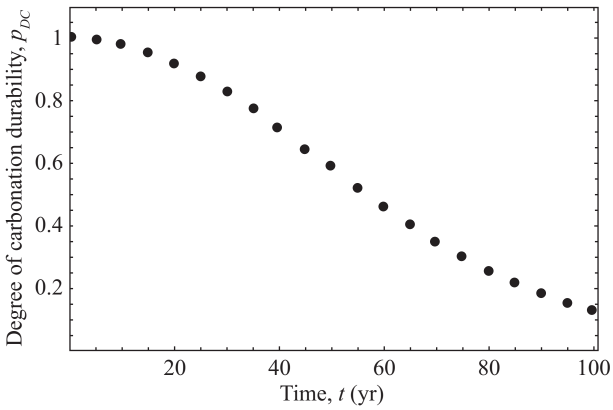

Fig. 5. Relationship between degree of carbonation durability and time of Chorng-ching viaduct.

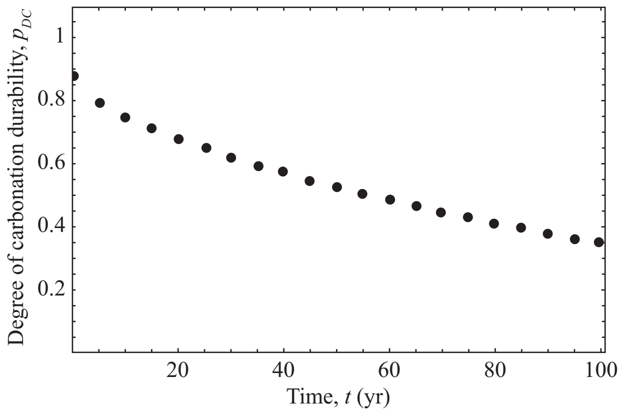

Fig. 6. Relationship between degree of carbonation durability and time of Wann-fwu bridge.

T = 21°C, RH = 70%, f_c' (*cylinder*) = 0.85 f_c' (*cube*) [12], and m_c = 1 into Eqs. (7), (8), and (23), we obtain the relationship between the degree of carbonation durability and time for the Chorng-ching viaduct and Wann-fwu bridge, as shown in Figs. 5 and 6, respectively. Using Eq. (24), Figs. 5 and 6, the relation can be changed to the relationship between the reli ability index *vs*. time for the Chorng-ching viaduct and Wann-fwu bridge, as shown in Figs. 7 and 8, respectively. Table 5 indicates the allowable probability for the steel commencing corrosion in concrete [23]. The engineering meanings of reliability index (β_c) (see Table 5) may be provided that the range of β_c for serviceability for different components of viaducts/bridges should be set at different values, like β_c = 0 for easily-replaceable components and β_c = 2.3 for hard-maintain structures. If β_c = 0, 0.5, and 1.25, then the carbonation service lives of the Chorng-ching viaduct and Wann-fwu bridge from Figs. 4 and 5 are 55 and 55, 40 and 17, and 18 and 0 years, respectively. On the carbonation service

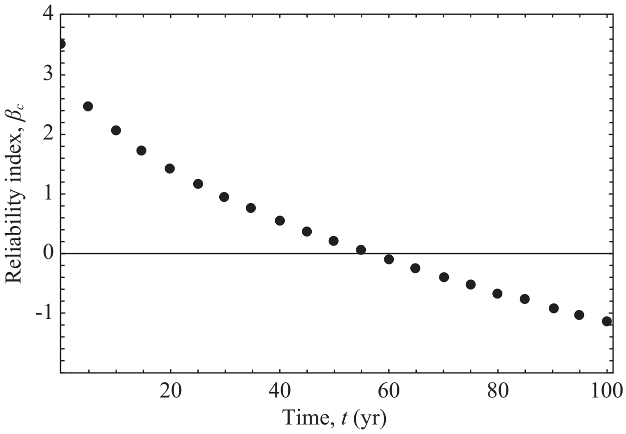

Fig. 7. Relationship between reliability index and time of Chorng-ching viaduct.

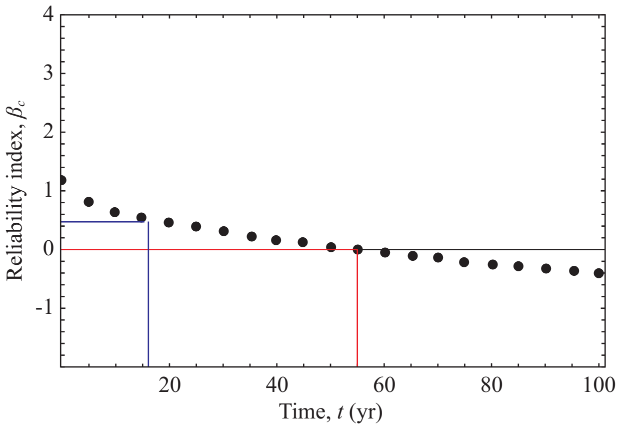

Fig. 8. Relationship between reliability index and time of Wann-fwu bridge.

life prediction of these examples, a flow chart showing the steps and required equations has been provided in Fig. 9.

IV. DISCUSSION

In the previous section, the results of Figs. 5 and 6 show that the degree of carbonation durability, Probability with binomial distribution, is a sigmoid regression curve [2]. These regression curves are unlikely to be linear, because the scale of the proportion, probability, is limited by the values of 0 and 1 and changes in any relevant explanatory variable (see Eq. (23)) at the extrerue ends of its scale are unlikely to produce much change in the probability. An appropriate transformation, i.e., a linearizing transformation, may convert this to a linear relationship. The transformation is termed the link function, since it provides the link between the linear function and the random variable. The linear function is called the linear predictor and the distribution random variable is the error distribution. Since the linear predictor covers an unlimited range, the link function should transform, the binormial probability from the range 0 to 1 to -∞ to ∞. Both the probity [3] and logit [2] transformation are used to achieve this. Based on this concept, Figs. 7 and 8 are respectively transformed from Figs. 5 and 6 and show that the reliability index has positive and negative values. The positive value of reliability index is usually to be considered in RC structures.

The deterioration process of RC structures subjected to corrosive media ingress is expressed in Fig. 10 [29]. The corrosion process in Fig. 10 can be divided into three stages, initiation time (t_c), depassivation time (t_p), and corrosion (or

Table 5. Allowable probability of steel commencing corrosion in concrete [23].

Classification		p_{fc} (%)	β_c
Prestressed concrete structure		10	1.25
Concrete structure	important structure	30	0.5
	General structure	50	0

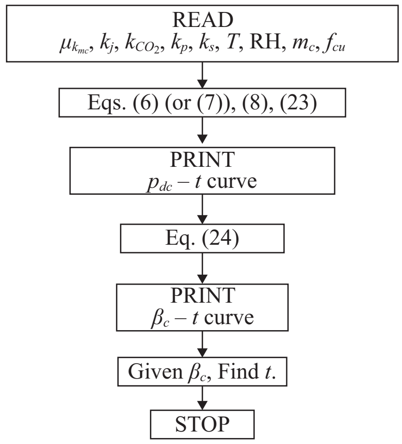

Fig. 9. Flow chart for carbonation service life prediction of existing concrete viaduct/bridge.

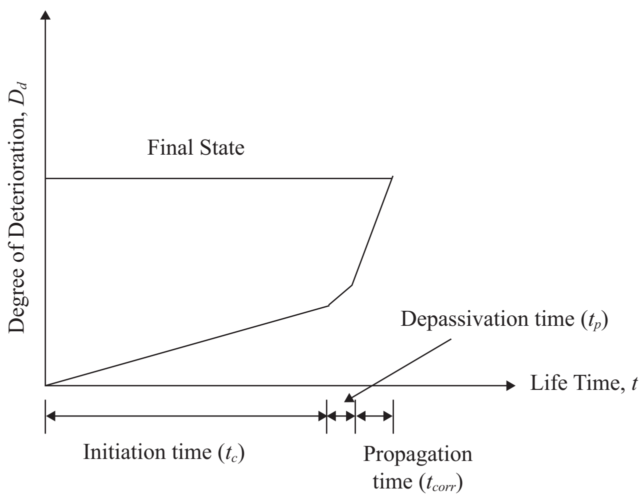

Fig. 10. Deterioration process of reinforced concrete structures due to corrosion.

Table 6. Prediction method for carbonation life for existing viaduct.

Prediction method	Formula	Remark	Reference
Fick second law of diffusion	$C(x,t) = C_0\, erfc \dfrac{x}{\sqrt{4D_{CO_2}t_c}}$	x = concrete cover C_0 = CO_2 concentration on concrete surface D = Coefficient of diffusion	Crank [8]
Guirguis	$t_c = \dfrac{L}{\lambda D_{CO_2}}$	L = Concrete cover λ = Concrete cover	Guirguis [11]
Hookham	$t_c = K_c K_e x^2 + K_a x$	K_c = Concrete quality factor (7.59) K_e = Environmental factor (0.85) K_a = Depassive corrosion factor (4.0)	Hookham [13]
AJMF	$C(x,t) = kt\left\{\left(1+\dfrac{x^2}{2D_{CO_2}t}\right)erfc\left(\dfrac{x}{2\sqrt{D_{CO_2}t}}\right)-\left(\dfrac{x}{\sqrt{\pi D_{CO_2}t}}\right)e^{\frac{x^2}{4D_{CO_2}t}}\right\}$	$k = 0.1$	Amey *et al.* [1]
	$C(x,t) = k\sqrt{t}\left\{e^{\frac{x^2}{4D_{CO_2}t}}-\left[\dfrac{x\sqrt{\pi}}{2\sqrt{D_{CO_2}t}}erfc\left(\dfrac{x}{2\sqrt{D_{CO_2}t}}\right)\right]\right\}$	$k = 0.545$	

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 to be normally provided the alkaline hydrated cement matrix for the steel 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 attaches onto the reinforcement becomes sufficiently severe to impair the load-carrying capacity. The degree of deterioration, D_d, in Fig. 10 can be defined as

$$D_d = 1 - \frac{i}{100} \tag{25}$$

where i is the integrity factor of the RC structure. The integrity factor is a measure of how integrative the RC structure is on a scale of 100, with 100 being the most integrative. A integrity factor of 100 indicates that no corrosion damage occurred over the test period. For instance, if RC structure is free of corrosion damage then the value of i is hundred. Thus, the degree of deterioration is zero. Thus, the total service life of RC structures may be represented as

$$t = t_c + t_p + t_{corr} \tag{26}$$

From Eq. (26) we know that the service life of RC structures can be calculated employing the t_c, t_p, and t_{corr} values. However, the service life of PC structures is only calculated the sum of both t_c and t_p values. The prediction methods for t_c are listed in Table 6. Using Table 6 and some parameters D_{CO_2} = 8.3 × 10^{-9} m_2/s, $C(x, t)$ = 1.0 kg/m^3, C_0 = 1.0 kg/m^3 and λ = 1.0 × 10^{-2}, we have the carbonation service life predictions for the Chorng-ching viaduct and Wann-fwu bridge viaducts listed in Tables 7 and 8, respectively. It is recessary to point out that the carbonation service lives obtained by the Fick second law of diffusion and AJMF are carried out using a commercial compuer pakage named "Mathematica" [30]. Neglecting the Hookham method [13], we obtain the average carbonation service life for the Chorng-ching viaduct and Wann-fwu bridge at 43 and 45 years, respectively. Comparing the carbonation service life predictions obtained from the proposed method with Fick's second law [8], Guirguis [11] and AJMF [1], we know that the prediction results are very reasonable. Ignoring the Hookham method [13], a reasonable illustration may be the calculation result was obtained using this approach because its' parameters are based on a chloride ion environment.

It is noteworthy to point out that the parameters considered in the existing methods are limited. The carbonation service lives predicted by the Hookham method [13] listed in Tables 7 and 8 are not reasonable. The proposed method considers many parameters based on the general atmospheric environment. It is obvious that the advantage of the proposed method is that it is objective, reasonable, reliable, and accurate.

V. CONCLUSIONS

A carbonation service life prediction theory for existing concrete viaduct/bridge has been described in this paper. The carbonation service life is equivalent to the initiation time, t_c. Based on the probability and reliability index analytical method, the predicted results for carbonation service life for

Table 7. Carbonation service life prediction for Chorng-ching viaduct.

No.	Concrete cover (mm)	t_c (yrs)				
		Fick's 2nd law	Guirguis	Hookham	AJMF	
					$k = 0.1$	$k = 0.545$
A	50	50.76	38.16	181.28	56.46	55.47
B	50	50.76	38.16	181.28	56.46	55.47
C	50	50.76	38.16	181.28	56.46	55.47
D	50	50.76	38.16	181.28	56.46	55.47
E	50	50.76	38.16	181.28	56.46	55.47
F	50	50.76	38.16	181.28	56.46	55.47
G	50	50.76	38.16	181.28	56.46	55.47
H	50	50.76	38.16	181.28	56.46	55.47
I	50	50.76	38.16	181.28	56.46	55.47
J	50	50.76	38.16	181.28	56.46	55.47
K*	25	12.76	19.08	50.32	31.19	25.11
L	25	12.76	19.08	50.32	31.19	25.11
M	25	12.76	19.08	50.32	31.19	25.11
N	50	50.76	38.16	181.28	56.46	55.47
O	50	50.76	38.16	181.28	56.46	55.47
P	25	12.76	19.08	50.32	31.19	25.11
Q	25	12.76	19.08	50.32	31.19	25.11
R	25	12.76	19.08	50.32	31.19	25.11
S	50	50.76	38.16	181.28	56.46	55.47
T	50	50.76	38.16	181.28	56.46	55.47
Average	42.5	39.36	32.43	141.99	48.87	46.36

* The carbonation depth of cored sample has surpassed concrete cover.

Table 8. Carbonation service life prediction for Wann-fwu bridge.

No.	Concrete cover (mm)	t_c (yrs)				
		Fick's 2nd law	Guirguis	Hookham	AJMF	
					$k = 0.1$	$k = 0.545$
A	40	32.77	30.52	119.32	45.26	43.11
B	40	32.77	30.52	119.32	45.26	43.11
C	40	32.77	30.52	119.32	45.26	43.11
D	40	32.77	30.52	119.32	45.26	43.11
E	40	32.77	30.52	119.32	45.26	43.11
F*	40	32.77	30.52	119.32	45.26	43.11
G*	40	32.77	30.52	119.32	45.26	43.11
H	40	32.77	30.52	119.32	45.26	43.11
I	40	32.77	30.52	119.32	45.26	43.11
J	40	32.77	30.52	119.32	45.26	43.11
K	25	12.76	19.08	50.32	31.19	25.11
L	50	50.76	38.16	181.28	56.46	55.47
M	50	50.76	38.16	181.28	56.46	55.47
N	50	50.76	38.16	181.28	56.46	55.47
O	50	50.76	38.16	181.28	56.46	55.47
P	50	50.76	38.16	181.28	56.46	55.47
Q	50	50.76	38.16	181.28	56.46	55.47
R*	50	50.76	38.16	181.28	56.46	55.47
S	50	50.76	38.16	181.28	56.46	55.47
T	50	50.76	38.16	181.28	56.46	55.47
U	50	50.76	38.16	181.28	56.46	55.47
Average	44.04	40.38	33.61	145.49	49.92	48.13

* The carbonation depth of cored sample has surpassed concrete cover.

the existing Chorng-ching viaduct and Wann-fwu bridge were 55 and 55, 40 and 17, and 18 and 0 years at the $\beta_c = 0$, 0.5, and 1.25 reliability indices, respectively. The results of this study may provide a basis for repair, strengthening and demolition of existing concrete bridges or viaducts. The prediction method proposed in this paper can be extended to applications for other existing concrete bridges or viaducts. In order to secure an accurate prediction result, we recommend that the parameter employed be used in an experimental model first.

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