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Load-Carrying Capacity Service Life Prediction of Existing Reinforced Concrete Bridge Using Time-Dependent Reliability Analysis

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Abstract: Engineering experience has clearly shown that existing reinforced concrete (RC) bridges are reliable to damage from environmental attack such as freeze-thaw, alkali-silica reaction, and corrosion. To establish feasible and rational method, reliable prediction of the service life of deteriorating RC bridge is needed. To obtain an exact insight into this problem, time-dependent reliability methods have to be used. In this paper, the reliability of RC girder bridges under corrosion condition is studied using a time-dependent reliability analysis in which both load and resistance are time-dependent. 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 load-carrying capacity service lives of existing RC bridges or viaducts can be expressed as $t_t = t_c + t_p + t_{corr}$ Many mathematical models could be employed to calculate each value of t_c, t_p, and t_{corr}. The value of t_t may be directly predicted from the relationship between reliability index and time. The existing Wann-fwu bridge and Chung-ching viaduct in Taipei were offered as illustrative examples for the modeling technique and load-carrying capacity service life prediction. The results of t_t predicted from the relationship between reliability index and time were rationally compared with the results of t_t calculated from the sum of t_c,t_p, and t_{corr}. The results of this study were provided as a decision making for repair, strengthening, and demolition of existing RC bridges or viaducts.

Keywords: Bridge, Corrosion, Load-carrying capacity, Reliability analysis, Ser-

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vice life prediction, Viaduct

1 Introduction

Owing to suffering natural and deteriorated environmental attack such as carbon dioxide, chloride ions, sulfate, freeze-thaw, and alkali-silicate reaction, reinforced concrete (RC) structures will occur material aging and strength decay during in-service. Through long-term under external physical action and internal chemetry reaction, the effective cross-section of steel will decrease and the properties of structural components will decay after the corrosion of steel in concrete. Meanwhile, rust-expansion-crack of concrete cover will occur the decreasing of effective cross-section of concrete. This is also induces the deceasing of bond stress between concrete and steel of RC structure. Further, both load-carrying capacity and durability of RC structure reduce. As a result, the calculation of load-carrying capacity of RC structures with corrosion absolutely is an important problem not to be ignored for carrying out durability evaluation and service life prediction.

Tabsh and Nowak (1991) described the highway girder bridges using a moment curvature relationship. They measured the highway girder bridges in terms of reliability indexes and shown that reliability indices for the system are higher than for girders. Nowak et al.(1994) used a probabilistic model to predict the load-carrying capacity of concrete bridge girders. This model provide bias factors (mean-to-nominal ratios) and coefficients of variation for moment-carrying capacity and shear capacity. The obtained statistical parameters can be used as basis for the development of design and evaluation criteria for concrete bridge components. Enright and Frangopol (1998) investigated the time-variant reliability of RC highway girder bridges subjected to time-dependent load and resistance. This study serves as an initial base on which to develop improved service life prediction models for deteriorating bridges. Stewart and Rosowsky(1998) developed a time-dependent reliability analysis for estimating RC bridge deck and the consequent loss of structural and serviceability performance due to chloride-induced corrosion. This approach may be applied to bridge management systems. Liang et al. (2002) used a service life prediction model which consists initiation (diffusion) time ($t_i = t_c$) included depassivation time (t_p) and propagation (corrosion) time (t_{corr}) to predict the service life of Chung-shan bridge in Taipei. Melchers[2009] pointed out that structural reliability theory provides a very comprehensive approach to assessing risks for complex infrastructure systems. He used the structural reliability to assess the corrosion of structural steel in marine environments. The structural reliability may be predicted the service lives of existing infrastructure and provided optimal management of their maintenance. Sung et al. [2009] adopted a state-of-the art pushover analysis method to calculate the seismic resistance capacity of the Li-Kun bridge in Taiwan. This study shows the efficiency of employing the retrofitting measure to improve the seismic behavior in terms of the return period of the earthquake or anticipated service life.

To date, however, no studies have attempted to predict the service life of existing RC bridges directly from the relationship between reliability index and time compared with that of the sum of t_c, t_p, and t_{corr}. This is a notable shortcoming, because the use of reliability indexes in previous studies may have resulted in underestimation or overestimation of service life for existing RC bridges. The objective of the present study was to determine the service life of existing RC bridges using the time-dependent reliability analysis and to compare the results of those of the sum of t_c, t_p, and t_{corr}. The results reported here may be of importance in bridge management system for determining repair, strengthening or demolition of existing RC bridges or viaducts.

2 Service life model of RC structure

For the sake of predicting the service life of existing RC structures, the prediction model should be essentially introduced. Huey (1997) raised a service life model of RC structure as depicted in Fig.1. In the Fig.1, t_i is the time of CO_2 penetration from concrete surface into internal concrete until that steel in concrete is occurred corrosion, 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 appeared cracking. t_t is the time of load-carrying capacity service life. Fig.2 shows the deterioration process of RC structures undergone 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 propagation (corrosion) 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 depapassivation 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

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

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

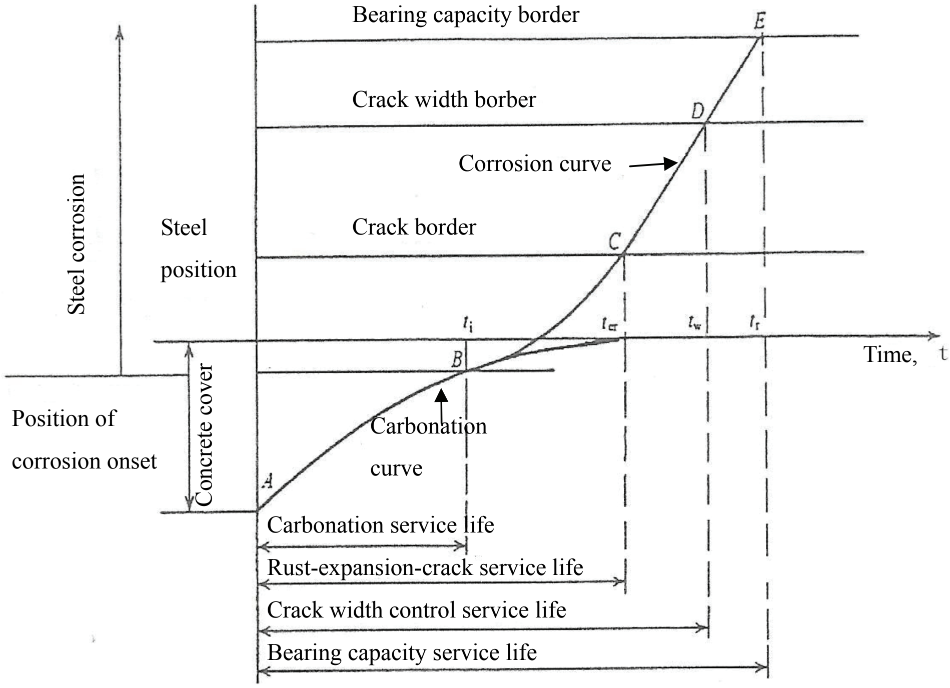


Figure 1: Schematic diagram of steel corrosion and service life of reinforced concrete structures.

Thus, the degree of deterioration is zero.

According to 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 by the each value of t_c, t_p, and t_{corr}.

3 Time-dependent reliability analysis for load-carrying capacity service life

The theory of load-carrying capacity of existing RC structure is considered that the load-carrying capacity of component reduces to a boundary value as a durability

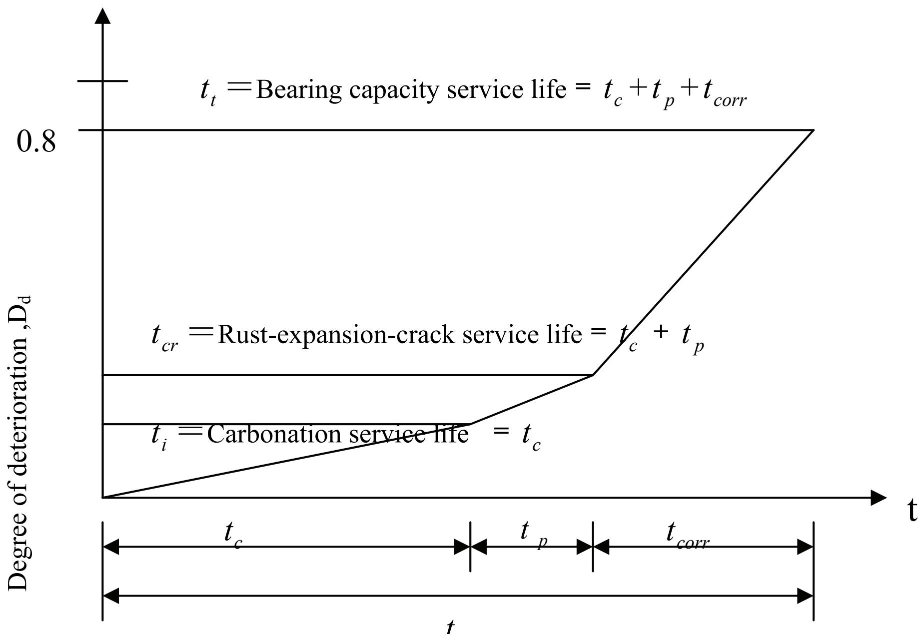


Figure 2: Schematic diagram of service life of existing reinforced concrete bridges.

limiting state when the corrosion of steel in concrete gives rise to resistance degradation. This means that the reduction of load-carrying capacity of RC structure which can not subject to loading action is referred to as an ending symbol of structural service life. Thus, the criterion of load-carrying capacity service life can be expressed as [Melchers (1999)]

$$\Omega_R(t) = \{\text{R(t) - S(t)} \geq 0\} \tag{4}$$

or

$$\Omega_{R_1}(t) = \{R(t_1,t) - S(t_1,t) \geq 0\} \tag{5}$$

where t is the time of structure in-service, $S(t)$ is the random process of action effect, R(t) is the random process of structural resistance, t_1 is the used time of structure, R(t_1,t) is the modified random process of structural resistance which is considered the structural state at time t_1, S(t_1,t) is the random process of action effect of structure which is considered the influence of suffering loading history, $\Omega_R(t)$ and $\Omega_{R_1}(t)$ are the criteria of load-carrying capacity of structure and are random processes. Eqs.(4) and (5) are the predictions of structural service life and structural remainder service life, respectively.

Owing to in the process of in-service of structures, they have the properties of random processes of both resistance and action. The analysis of load-carrying capacity of structure is substantially based on the service life prediction of structural dynamic reliability. According to the definition of reliability, the dynamic probability of existing RC bridge can be expressed as [Melchers (1999)]

$$P_s(t) = P\left\{\Omega_R(t)\right\} = P\left\{\text{R(t) - S(t)} \geq 0\right\}, \text{t} \in [0,T] \tag{6}$$

or

$$P_s\left(t_1,t\right) = P\left\{\Omega_{R_1}(t)\right\} = P\left\{R\left(t_1,t\right) \text{ - S}\left(t_1,t\right) \geq 0\right\}, t \in [0,T] \tag{7}$$

where T is the service life of structural design. The corresponding dynamic reliability indexes of Eqs. (6) and (7) can be written as [Melchers (1999)]

$$\beta(t) = -\Phi^{-1}\left[1 - P_s(t)\right] \tag{8}$$

or

$$\beta(t_1,t) = -\Phi^{-1}\left[1 - P_s(t_1,t)\right] \tag{9}$$

where Φ^{-1} is the inverse of standard normal distribution function.

Bending component is one of the major kinds of structural components. In this paper, the bridge deck bending component of existing RC bridge is referred to as investigated example. The calculation model of load-carrying capacity of corrosion beam (i.e. , bridge deck) [Chen and Duan (2000)] is

$$M_{su}(t) = \alpha_1 f_c' bx\left(h_0 - \frac{x}{2}\right) \tag{10}$$

$$x = \frac{A_{se}(t) f_y}{\alpha_1 f'_c b} \tag{11}$$

where $M_{su}(t)$ is the bending capacity of corrosion beam cross-section, f'_c is the compressive strength of concrete, α_1 is the ratio of stress taken from the rectangular stress diagram of concrete in compressive zone to the design value of compressive strength of concrete, b is the cross-sectional width, h_0 is the effective height of cross-section, f_y is the yielding strength of steel, and $A_{se}(t)$ is the equivalent cross-sectional area of tensile steel at time t.

Having a bearing on algorithm of $A_{se}(t)$, it needs synthetically to consider the influence of the loss of cross-sectional area of steel due to corrosion, the reduction of yielding strength of steel, and the reducing of bond stress between concrete and steel. It can be expressed in terms of

$$A_{se}(t) = \sum_{i=1}^{i=n} k_{si}\alpha_{si}A_{si}(t) \tag{12}$$

where $A_{si}(t)$ is the design value of cross-sectional area of the i-th tensile steel, k_{si} is the coordinated work coefficient of the i-th steel. This mainly considers that one is the performance reduction of bond stress induced the model change of RC beam subjected to action. Another is the yielding strength of steel which can not give full play to the influence of load-carrying capacity. α_{si} is the reduction coefficient of the i-th steel yielding strength. The value of α_{si} can be calculated by the following formula [Niu (2003)]

$$\alpha_{si} = \begin{cases} 1, & \eta_{si} \le 0.05 \\ 1 - 1.077\eta_{si}(t), & 0.05 < \eta_{si} \le 0.15 \end{cases} \tag{13}$$

where η_{si}(t) is the rate of loss of the i-th steel cross-section. The value of η_{si}(t) can be estimated from the depth of steel corrosion [Niu (2003)]

$$\eta_{si}(t) = \frac{4\delta_{ei}(t)}{d_i} + \frac{4\delta_{ei}^2(t)}{d_i^2} \approx \frac{4\delta_{ei}(t)}{d_i} \tag{14}$$

where d_i is the diameter of the i-th steel and $\delta_{ei}(t)$ is the corrosion depth of the i-th steel at time t.

As to the coordinated work coefficient of the i-th steel, it can be estimated by the following formula [Niu (2003)]

$$k_{si} = \begin{cases} 1, & \delta_{ei}(t) \le \delta_{cri}(t) \\ 1 - 0.85\left[\delta_{ei}(t) - \delta_{cri}\right], & \delta_{cri}(t) < \delta_{ei}(t) \le 0.3 \\ 0.745 + 0.7\delta_{cri}(t), & 0.3 < \delta_{ei}(t) \end{cases} \tag{15}$$

where $\delta_{cri}(t)$ is the corresponding corrosion depth of the i-th steel when the concrete cover is occurred cracking.

Now consider the deterioration of structural resistance parameter followed time increasing. This means that consider the alleviation model of resistance of structural component. Substituting Eq. (11) into Eq. (10) we obtain the calculation resistance of corroded beam subjected to bending action

$$R_p(t) = M_{su}(t) = F_y(t)\left(h_0 - \frac{1}{2}\frac{F_y(t)}{\alpha_1 f'_c b}\right) \tag{16}$$

where $F_y(t)$ is the yielding tension force of corroded steel. The value of $F_y(t)$ can be written as

$$F_y(t) = f_y A_{se}(t) \tag{17}$$

Based on Eq. (16), the mean and standard deviation of $R_p(t)$ can be respectively expressed in terms of

$$\mu_{R_p}(t) = \mu_{F_y}(t)\mu_{h_0}\left(1-\frac{1}{2}\Gamma(t)\right) \tag{18}$$

and

$$\sigma_{R_p}(t) = \left\{\mu_{F_y}^2(t)\mu_{h_o}^2\left[(1-\Gamma(t))^2\,\delta_{F_y}^2(t)+\frac{1}{4}\Gamma^2(t)\left(\delta_b^2+\delta_{fc'}^2\right)+\delta_{h_0}^2\right]\right\}^{\frac{1}{2}} \tag{19}$$

where

$$\Gamma(t) = \frac{\mu_{F_y}(t)}{\alpha_1\mu_b\mu_{fc'}\mu_{h_0}} \tag{20}$$

in which $\mu_{fc'}$ and $\delta_{fc'}$ are respectively the mean and coefficient of variation of fc', μ_b and δ_b are respectively the mean and coefficient of variation of b, μ_{h_0} and δ_{h_0} are respectively the mean and coefficient of variation of h_0, and $\mu_{F_y}(t)$ and $\delta_{F_y}(t)$ are respectively the mean and coefficient of variation of $F_y(t)$ and can be written as

$$\mu_{F_y}(t) = \mu_{f_y}\mu_{A_{se}}(t) \tag{21}$$

and

$$\delta_{F_y}(t) = \left[\delta_{f_y}^2+\delta_{A_{se}}^2(t)\right]^{\frac{1}{2}} \tag{22}$$

where μ_{f_y} and δ_{f_y} are respectively the mean and coefficient of variation of f_y and $\mu_{A_{se}}(t)$ and $\delta_{A_{se}}(t)$ are respectively the mean and coefficient of variation of $A_{se}(t)$.

According to Eq. (12), we obtain

$$\mu_{A_{se}}(t) = \sum_{i=1}^{i=n}\mu_{k_{si}}(t)\mu_{\alpha_{si}}(t)\mu_{A_{se}}(t) \tag{23}$$

$$\sigma_{A_{se}}(t) = \left[\sum_{i=1}^{i=n}\left(\frac{\partial A_{se}}{\partial k_{si}}|_\mu\right)^2\sigma_{k_{Si}}^2(t)+\sum_{i=1}^{i=n}\left(\frac{\partial A_{se}}{\partial \alpha_{si}}|_\mu\right)^2\sigma_{\alpha_{Si}}^2(t)+\sum_{i=1}^{i=n}\left(\frac{\partial A_{se}}{\partial A_{si}}|_\mu\right)^2\sigma_{A_{si}}^2(t)\right]^{\frac{1}{2}} \tag{24}$$

$$\delta_{A_{se}}(t) = \frac{\sigma_{A_{se}}(t)}{\mu_{A_{se}}(t)} \tag{25}$$

where $\mu_{A_{Si}}(t)$ and $\sigma_{A_{si}}(t)$ are respectively the mean and standard deviation of $A_{si}(t)$, and $\mu_{\alpha_{si}}(t)$ and $\sigma_{\alpha_{si}}(t)$ are respectively the mean and standard deviation function of $\alpha_{si}(t)$ and can be found from Eq.(13)

$$\mu_{\alpha_{si}}(t) = \begin{cases} 1, & \mu_{\eta_{si}}(t) \leq 0.05 \\ 1 - 1.077\mu_{\eta_{si}}(t), & 0.05 < \mu_{\eta_{si}}(t) \leq 0.15 \end{cases} \tag{26}$$

and

$$\sigma_{\alpha_{si}}(t) = 1.077\sigma_{\eta_{si}}(t) \tag{27}$$

Both $\mu_{k_{si}}(t)$ and $\sigma_{k_{si}}(t)$ in Eqs. (23)-(25) are the mean and standard deviation functions of $k_{si}(t)$ and can be found from Eq. (15)

$$\mu_{k_{si}}(t) = \begin{cases} 1, & \mu_{\delta_{ei}}(t) \leq \mu_{\delta_{cri}} \\ 1 - 0.85\left[\mu_{\delta_{ei}}(t) - \mu_{\delta_{cri}}\right], & \mu_{\delta_{cri}} < \mu_{\delta_{ei}}(t) \leq 0.3 \\ 0.745 + 0.7\mu_{\delta_{cri}}, & 0.3 < \mu_{\delta_{ei}}(t) \end{cases} \tag{28}$$

and

$$\sigma_{K_{Si}}(t) = \left[\left(\frac{\partial k_{si}}{\partial \delta_{ei}}|_{\mu}\right)^2 \sigma^2_{\delta_{ei}}(t) + \left(\frac{\partial k_{si}}{\partial \delta_{cri}}|_{\mu}\right)^2 \sigma^2_{\delta_{cri}}\right]^{\frac{1}{2}} \tag{29}$$

where $\mu_{\delta_{cri}}$ and $\sigma_{\delta_{cri}}$ are respectively the mean and standard deviation of δ_{cri} and can be found from the following formulas [Niu (2003)]

$$\mu_{\delta_{cri}} = \begin{cases} \mu_{K_{mcr}} k_{crs}(0.012\frac{\mu_c}{\mu_d} + 0.00084\mu_{f_{cu}} + 0.02 & \text{(bare steel)} \\ \mu_{K_{mcr}} k_{crs}(0.008\frac{\mu_c}{\mu_d} + 0.00055\mu_{f_{cu}} + 0.022) & \text{(deformed steel)} \\ \mu_{K_{mcr}}(0.026\frac{\mu_c}{\mu_d} + 0.0025\mu_{f_{cu}} + 0.068) & \text{(stirrup and network} \\ & \text{distributed steel)} \end{cases} \tag{30}$$

and

$$\sigma_{\delta_{cri}}(t) = \left[(\frac{\partial \delta_{cr}}{\partial K_{mcr}}|_{\mu})^2 \sigma^2_{k_{mcr}} + (\frac{\partial \delta_{cr}}{\partial c}|_{\mu})^2 \sigma^2_c + (\frac{\partial \delta_{cr}}{\partial d}\bigg|_{\mu})^2 \sigma d^2 + (\frac{\partial \delta_{cr}}{\partial f_{cu}}|_{\mu})^2 \sigma_{f_{cu}}\right]^{\frac{1}{2}} \tag{31}$$

where $\mu_{K_{mcr}}$ and $\sigma_{K_{mcr}}$are respectively the mean and standard deviation of the coefficient of uncertainty (K_{mcr}) of calculation model of corrosion depth of steel at the rust-expansion-crack of concrete cover, and k_{crs} is the influence coefficient of steel position. $k_{crs} = 1.0$ for steel at corner and $k_{crs} = 1.35$ for steel at noncorner. μ in Eq. (31) means that the partial derivative takes the value at the mean.

$\mu_{\delta_{ei}}(t)$ and $\sigma_{\delta_{ei}}(t)$ in Eqs. (28) and (29) are respectively the mean and standard deviation functions of $\delta_{ei}(t)$. If consider the mean and standard deviation functions of steel corrosion depth before the rust-expansion-crack of concrete cover, then the values of $\mu_{\delta_{eli}}(t)$, $\sigma_{\delta_{el}}(t)$, $\mu_{\lambda_{el}}$ and $\sigma_{\lambda_{el}}$ can be found from the following formulas [Niu (2003)]

$$\mu_{\delta_{eli}}(t) = \mu_{k_{mel}}\mu_{\lambda_{el}}(t-t_i) \tag{32}$$

$$\mu_{\lambda_{el}} = 46k_{cr}k_{ce}e^{0.04T}(\text{RH - 0.45})^{\frac{2}{3}}\mu_c^{-1.36}\mu_{f_{cu}}^{-1.83} \tag{33}$$

$$\sigma_{\delta_{el}}(t) = \left[(\frac{\partial \delta_{el}}{\partial k_{mel}}|_{\mu})^2\sigma_{k_{mel}}^2 + (\frac{\partial \delta_{el}}{\partial \lambda_{el}}|_{\mu})^2\sigma_{\lambda_{el}}^2 \right]^{\frac{1}{2}} \tag{34}$$

$$\sigma_{\lambda_{el}} = \left[(\frac{\partial \lambda_{el}}{\partial c}|_{\mu})^2\sigma_c^2 + (\frac{\partial \lambda_{el}}{\partial f_{cu}}|_{\mu})^2\sigma_{f_{cu}}^2 \right]^{\frac{1}{2}} \tag{35}$$

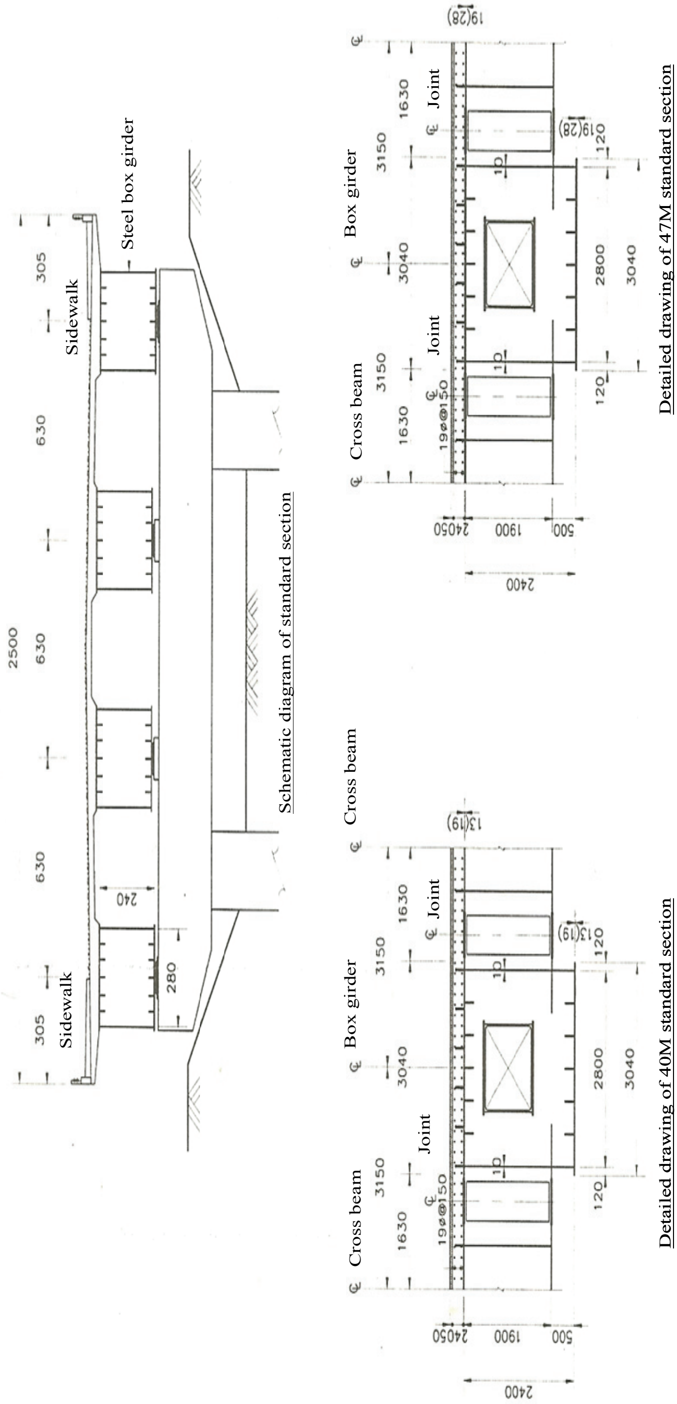
where $\mu_{\delta_{el}}(t)$ and $\sigma_{\delta_{el}}(t)$ are respectively the mean and standard deviation functions of the steel rust volume before rust-expansion-crack $\delta_{el}(t)$, $\mu_{k_{mel}}$ and $\sigma_{k_{mel}}$ are respectively the mean and standard deviation of the coefficient of uncertainty of calculation model before the rust-expansion-crack of concrete cover k_{mel}, $\mu_{\lambda_{el}}$ and $\sigma_{\lambda_{el}}$ are respectively the mean and standard deviation of the rate of corrosion of steel before rust-expansion-crack λ_{el}, μ_c and σ_c are respectively the mean and standard deviation of concrete cover c, $\mu_{f_{cu}}$ and $\sigma_{f_{cu}}$ are respectively the mean and standard deviation of the cube compressive strength of concrete $f_{cu}(fc' = 0.85f_{cu})$ [Hassoun (2003)]. μ in Eqs. (34) and (35) mean that the partial derivatives take the values at the mean.

If consider the mean and standard deviation of steel corrosion depth after the rust-expansion-crack of concrete cover, the values of $\mu_{\delta_{ei}}(t)$ and $\sigma_{\delta_{ei}}(t)$ can be found from the following formulas [Niu (2003)]

$$\mu_{\delta_{ei}}(t) = \begin{cases} \mu_{\delta_{cr}} + \mu_{K_{mei}}2\mu_{\lambda_{el}}(t-t_{cr}), & \lambda_{el} > 0.003 \\ \mu_{\delta_{cr}} + \mu_{K_{mei}}(3.5\mu_{\lambda_{el}} - 500\mu_{\lambda_{el}}^2)(t-t_{cr}), & \lambda_{el} \le 0.003 \end{cases} \tag{36}$$

and

$$\sigma_{\delta_{ei}}(t) = \left[(\frac{\partial \delta_{ei}}{\partial \delta_{cr}}|_{\mu})^2\sigma_{\delta_{cr}}^2 + (\frac{\partial \delta_{ei}}{\partial k_{mei}}|_{\mu})^2\sigma_{k_{mei}}^2 + (\frac{\partial \delta_{ei}}{\partial \lambda_{el}}|_{\mu})^2\sigma_{\lambda_{el}}^2 \right]^{\frac{1}{2}} \tag{37}$$



Explanation: The numeral in parentheses is the plate thickness of steel beam under negative moment

Figure 3: Steel beam section diagram of Wann-fwu bridge.

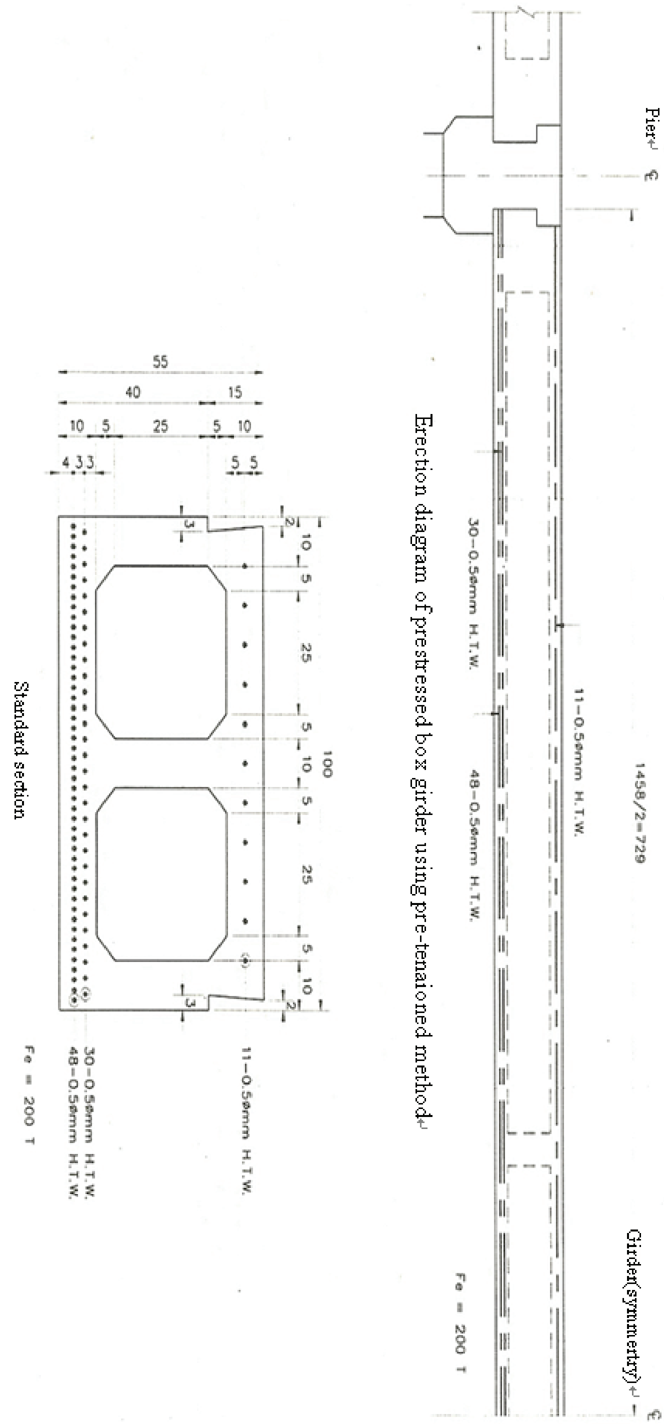


Figure 4: Section diagram of 15M standard span prestressed beam of Chorng-ching viaduct.

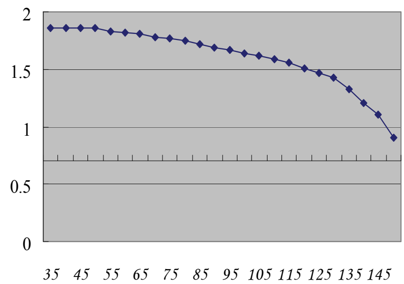


Figure 5: Relationship between durability degree of load-carrying capacity service life and time for Wann-fwu bridge.

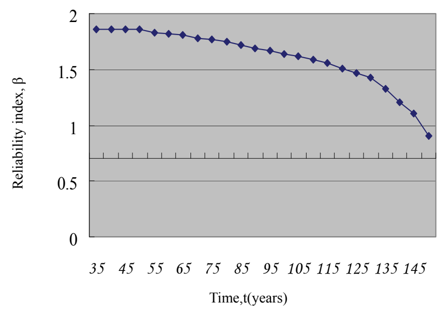


Figure 6: Relationship between reliability index of load-carrying capacity service-life and time for Wann-fwu bridge.

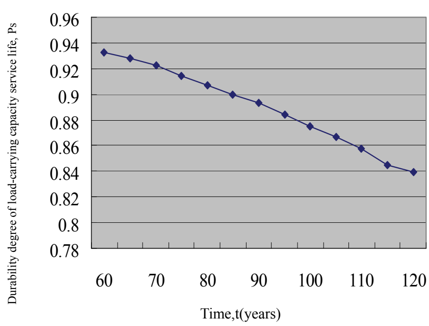


Figure 7: Relationship between durability degree of load-carrying capacity service life and time for Chorng-ching viaduct.

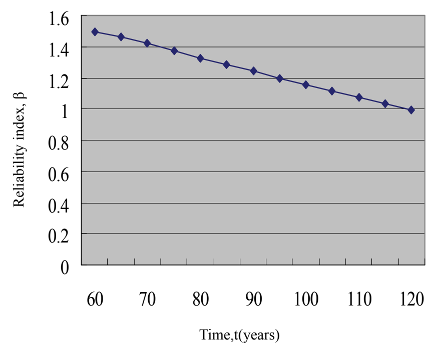


Figure 8: Relationship between reliability index of load-carrying capacity service life and time for Chorng-ching viaduct.

where $\mu_{\delta_{cr}}$ and $\sigma_{\delta_{cr}}$ are respectively the mean and standard deviation of the steel corrosion depth when concrete cover occurs crack due to rust expansion of steel in concrete δ_{cr}, $\mu_{k_{mei}}$ and $\sigma_{k_{mei}}$ are respectively the mean and standard deviation of the coefficient of uncertainty of calculation model after the rust-expansion-crack of concrete cover k_{mei}, μ in Eq. (37) means that the partial derivative takes the value at the mean.

If substituting Eqs. (30)-(37) into the following equations

$$\delta_{R_p}(t) = \frac{\sigma_{R_p}(t)}{\mu_{R_p}(t)} \tag{38}$$

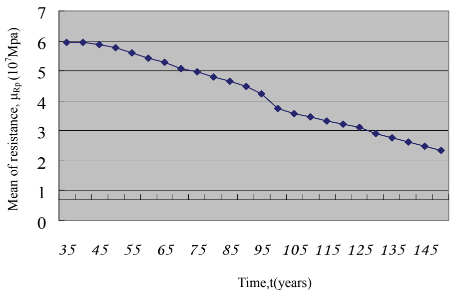


Figure 9: Relationship between mean of resistance and time for Wann-fwu bridge.

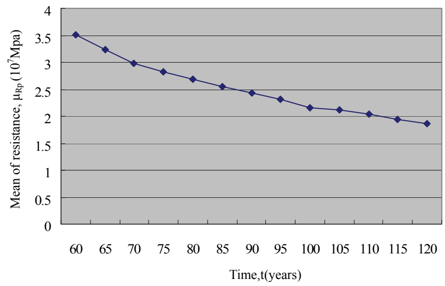


Figure 10: Relationship between mean of resistance and time for Chorng-ching viaduct.

$$\delta_R(t) = \left[\delta_{K_p}^2 + \delta_{R_p}^2(t)\right]^{\frac{1}{2}} \tag{39}$$

$$\mu_R(t) = \mu_{K_p}\mu_{R_p}(t) \tag{40}$$

$$\sigma_R(t) = \delta_R(t)\mu_R(t) \tag{41}$$

then the mean and standard deviation functions of resistance of corroded beam component subjected to bending action of existing RC bridge/viaduct can be found.

Let $t = t_1$ be the in-service criterion period of existing RC bridge/viaduct. Then the mean, $\mu_{R_{min}}$ and standard deviation $\sigma_{R_{min}}$ of the minimum value R_{min} of the resistance of existing RC bridge/viaduct can be obtained. Using Eqs. (6)-(9), the dynamic probability and reliability index of the load-carrying capacity of corroded beam (i.e., bridge deck) of existing RC bridge/viaduct can be found.

4 Illustrative examples

For the sake of examining the serviceability of the theory of load-carrying capacity stated above, the existing Wann-fwu bridge and Chorng-ching viaduct in Taipei are used to evaluate the load-carrying capacity service life. At present, it is needed to point out that the corresponding rust-expansion-crack service lives are 33 and 57 years at reliability index $\beta_{cr} = 1.0$ [Chang, 2008].

Figs.3 and 4 show the section diagrams of bridge decks of the existing Wann-fwu bridge and Chorng-ching viaduct, respectively. Tables 1 and 2 indicate the mechanical properties of structural components of corresponding bridge and viaduct. Tables 3 and 4 denote the cross-sectional sizes of corresponding bridge and viaduct. To employ the theory mentioned early to estimate the service life of the existing Wann-fwu bridge and Chorng-ching viaduct in Taipei, many parameters should

Table 1: Mechanical properties of structural components of Wann-fwu bridge

Test point No.	Testing points	Design compressive strength $f'_c\,(kgf/cm^2)$	Compressive strength $f'_c\,(MPa)$	Tensile strength $f'_t\,(MPa)$	Elastic modulus $E_c\,(GPa)$	Effective elastic modulus $E_{ef}\,(GPa)$
A	A1-P1 Slab(left)	240	23.52	2.35	22.77	7.59
B	P1-P2 Slab(left)	240	23.52	2.35	22.77	7.59
C	P2-P3 Slab(left)	240	23.52	2.35	22.77	7.59
D	P3-P4 Slab(left)	240	23.52	2.35	22.77	7.59
E	P4-A2 Slab(left)	240	23.52	2.35	22.77	7.59
F*	A1-P1 Slab(right)	240	23.52	2.35	22.77	7.59
G*	P1-P2 Slab(right)	240	23.52	2.35	22.77	7.59
H	P2-P3 Slab(right)	240	23.52	2.35	22.77	7.59
I	P3-P4 Slab(right)	240	23.52	2.35	22.77	7.59
J	P4-A2 Slab(right)	240	23.52	2.35	22.77	7.59
K	S1-S1(Side)	240	23.52	2.35	22.77	7.59
L	P1 Right capbeam(rear)	280	27.44	2.74	24.60	8.20
M	P1 Left capbeam(rear)	280	27.44	2.74	24.60	8.20
N	P2 Right capbeam(rear)	280	27.44	2.74	24.60	8.20
O	P2 Left capbeam(rear)	280	27.44	2.74	24.60	8.20
P	P2 Middle capbeam(rear)	280	27.44	2.74	24.60	8.20
Q	P3 Right pier	280	27.44	2.74	24.60	8.20
R*	P4 Capbeam	280	27.44	2.74	24.60	8.20
S	P4 Left pier	280	27.44	2.74	24.60	8.20
T	Retaining wall(guide passage)(1)	280	27.44	2.74	24.60	8.20
U	Retaining wall(guide passage)(2)	280	27.44	2.74	24.60	8.20
	Average	259.05	25.28	2.53	23.59	7.86

*The carbonation of cored sample has surpassed concrete cover.

Remark: 1. A:Abutment; G:Girder; P:Pier; S:Slab(bridge deck)

2. f't=0.1f'c

3. $E_c = 15000\sqrt{f'c} \times 9.8 \times 10^{-6}$ (GPa)

4. $E_{ef} = \dfrac{E_c}{1+\phi_{cr}}$, ϕ_{cr} = Creep coefficient of concrete=2

Table 2: Mechanical properties of structural components of Chorng-ching viaduct

Test point No.	Testing points	Design compressive strength $f'_c\,(kgf/cm^2)$	Compressive strength $f'_c\,(MPa)$	Tensile strength $f'_t\,(MPa)$	Elastic modulus $E_c\,(GPa)$	Effective elastic modulus $E_{ef}\,(GPa)$
A	A1 Abutment	210	20.58	2.06	21.30	7.10
B	P23 Left pier	210	20.58	2.06	21.30	7.10
C	P3 Right pier	210	20.58	2.06	21.30	7.10
D	P24 Right pier	210	20.58	2.06	21.30	7.10
E	Retaining wall(terminal guide passage)(right)	210	20.58	2.06	21.30	7.10
F	P24 Left pier	210	20.58	2.06	21.30	7.10
G	Retaining wall(guide passage)(left)	210	20.58	2.06	21.30	7.10
H	Retaining wall(guide passage)(right)	210	20.58	2.06	21.30	7.10
I	Retaining wall(guide passage)(left)	210	20.58	2.06	21.30	7.10
J	P23 Right pier	210	20.58	2.06	21.30	7.10
K*	G6S4(Girder)	350	34.3	3.43	27.50	9.17
L	G3S4(Girder)	350	34.3	3.43	27.50	9.17
M	G10S4(Girder)	350	34.3	3.43	27.50	9.17
N	G14S24(Girder)	350	34.3	3.43	27.50	9.17
O	G1S4(Side)	350	34.3	3.43	27.50	9.17
P	G7S4(Girder)	350	34.3	3.43	27.50	9.17
Q	G11S4(2)(Girder)	350	34.3	3.43	27.50	9.17
R	G2S4(Girder)	350	34.3	3.43	27.50	9.17
S	G14S23(Side)	350	34.3	3.43	27.50	9.17
T	G1S23(Side)	350	34.3	3.43	27.50	9.17
	Average	280	27.44	2.74	24.40	8.13

*The carbonation of cored sample has surpassed concrete cover.

Remark:1. A:Abutment; G:Girder; P:Pier; S:Slab(bridge deck)

2. f't=0.1f'c

3. $E_c = 15000\sqrt{f'c} \times 9.8 \times 10^{-6}$ (GPa)

4. $E_{ef} = \dfrac{E_c}{1+\phi_{cr}}, \phi_{cr} =$ Creep coefficient of concrete=2

Table 3: Cross-sectional size of Wann-fwu bridge

Test point No.	Steel diameter of bridge deck (mm)	Cross-sectional concrete cover (mm)	Cross-sectional width (mm)	Cross-sectional effective height (mm)
A	14.23	20	900	650
B	14.23	20	900	650
C	14.23	20	900	650
D	14.23	20	900	650
E	14.23	20	900	650
F*	18.5	25	860	800
G*	18.5	25	860	800
H	18.5	25	860	800
I	18.5	25	860	800
J	18.5	25	860	800
K	16.26	30	850	750
L	16.26	30	850	750
M	16.26	30	850	750
N	16.26	30	850	750
O	16.26	30	850	750
P	15.31	35	900	700
Q	15.31	35	900	700
R*	15.31	35	900	700
S	15.31	35	900	700
T	15.31	35	900	700
U	15.31	35	900	700
Average	16.04	27.86	878.57	723.81

be well known. However, besides many parameters were offered in the Tables 1-4, other parameters were needed as follows: $\alpha = 0.85$, $\mu_{K_p} = 1.0$, $\delta_{K_p} = 0.04$, $\mu_{k_{mel}} = 0.996$, $k_{cr} = 1.6$, $k_{ce} = 3.5$, $K_{crs} = 1.35$[Niu (2003)], T=21°, and RH=70%. Substituting these well know parameters into Eqs. (10)-(37) and passing a series of calculation, we can finally obtain the durability degree vs. time and reliability index vs. time from Eqs. (6) and (8) for the existing bridge and viaduct as shown in Figs. 5 and 6 and Figs. 7 and 8, respectively. Based on Figs. 7 and 8, we gain that the load-carrying capacity service lives t_t at reliability index β=1.0 are 147 and 116 years for existing Wann-fwu bridge and Chorng-ching viaduct, respectively.

According to Eqs. (38)-(41), we obtain the relationship between mean of resistance

Table 4: Cross-sectional size of Chorng-ching viaduct

Test point No.	Steel diameter of bridge deck (mm)	Cross-sectional concrete cover (mm)	Cross-sectional width (mm)	Cross-sectional effective height (mm)
A	13.5	10	1000	550
B	13.5	10	1000	550
C	13.5	10	960	700
D	13.5	10	950	700
E	13.5	10	950	700
F	12.6	10	900	700
G	12.6	14	900	700
H	12.6	14	1000	700
I	12.6	14	1000	700
J	12.6	14	1000	700
K*	13.2	12	1000	640
L	13.2	12	1000	640
M	12.4	12	1000	640
N	12.4	12	950	640
O	12.4	20	950	640
P	13.6	20	950	640
Q	13.6	20	950	550
R	13.6	20	950	550
S	13.6	14	950	550
T	13.6	14	950	550
Average	13.11	13.6	965.5	637

vs. time for the corresponding bridge and viaduct as shown in Figs. 9 and 10. It is very obvious that the decay ranking is the Chorng-ching viaduct and Wann-fwu bridge.

5 Discussion

According to Figs.1 and 2 and Eq. (3), we know that the load-carrying capacity of existing RC structure can be expressed in terms of $t_t = t_c + t_p + t_{corr}$. Table 5 lists the methods for calculating the values of t_c, t_p, and t_{corr}. The related data for calculating the values of t_c and t_p can be found in Fang [2007] and Chang [2008]. To calculate the value of t_{corr}, besides Tables 1-4, we need assume the following parameters: α=0.575, $\rho_{cor} = 3600kg/m^3 = 3.6g/cm^3$, D=19mm, s=10cm, $do =$

Table 5: Prediction method for initiation, depassivation, and propagation time due to corrosion

Time	Prediction method	Formula	Remark	Reference
t_c	Fick's second law of diffusion	$C(x,t)=C_0 erfc\frac{x}{\sqrt{4D_c t_c}}$	$x=$ constant cover $C_o=CO_2$ concentration on concrete surface $D_c=$ diffusion coefficient of CO_2	Crank[1975]
	Guirguis	$t_c=\frac{L}{\lambda D_c}$	$L=$ constant cover $\lambda=$ constant	Guirguis[1987]
	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) $K_a=$ active corrosion factor(4.0)	Hookham[1992]
	AJMF	$C(x,t)=kt\left\{\left(1+\frac{x^2}{2D_c t}\right)erfc\left(\frac{x}{2\sqrt{D_c t}}\right)-\left(\frac{x}{\sqrt{\pi D_c t}}\right)e^{-\frac{x^2}{4D_c t}}\right\}$	K=0.1	Amey et al.[1998]
		$C(x,t)=k\sqrt{t}\left\{e^{-\frac{x^2}{4D_c t}}-\left[\frac{x\sqrt{\pi}}{2\sqrt{D_c t}}erfc\left(\frac{x}{2\sqrt{D_c t}}\right)\right]\right\}$	K=0.545	
t_p	Parabolic curve	$t_p=\frac{1}{12D_c}\left(\frac{L}{1-\sqrt{\frac{C^*}{C_0}}}\right)^2$	$D_c=$ Diffusion coefficient of Cl^- $(m^2/year)$ $L=$ Concrete cover(m) $C^*=$ Threshold value of Cl^- concentration $C_o=$ Cl^- concentration on concrete surface	Bazant[1979]
	Declined straight line	$t_p=\frac{1}{4D_c}\left(\frac{L}{1-\sqrt{\frac{C^*}{C_0}}}\right)^2$	Same above	Fang [2007] Liang et al.[2009]
	Parabolic curve +declined straight line	$t_p=\frac{7}{384D_c}[\frac{L}{1-\sqrt{\frac{C^*}{C_0}}}]^2+\frac{1}{64D_c}[\frac{L}{1-\frac{C^*}{C_0}}]^2$	Same above	Chang [2008] Liang et al.[2010]

Table 5: (Continued) Prediction method for initiation, depassivation, and propagation time due to corrosion

	Bazant	$t_{corr} = \rho_{cor} \frac{D}{S} \frac{\Delta D}{j_r}$ $\Delta D = 2 f'_t \frac{L}{D} \delta_{pp}$	D = Steel diameter(m) δ_{pp} = Radical compliance of steel hole f'_t = Tensile strength of concrete (kgf/m^2) L=Concrete cover(m) s=Steel space(m) ρ_{cor} = Density of corrosion product (kg/m^3) j_r = Rate of rust product per unit area of place (g/m^2s)	Bazant[1979]
	Modified Bazant	$t_{corr} = \rho_{cor} \frac{D}{S} \frac{\Delta D^*}{j_r}$ $\Delta D^* = f'_t \left[2\left(\frac{L}{D} + 1 \right) \right] \delta_{pp}$	Same above	Liang et al.[2002]
t_{corr}	LW	$t_{corr} = \frac{W_{crit}^2}{2k_p}$ $W_{crit} = \rho_{cor} \{ \pi [\frac{Lf'_t}{E_{ef}} (\frac{a^2+b^2}{b^2-a^2} + \nu_c) + d_o] D + \frac{W_{st}}{\rho_{st}} \},$ $a = \frac{D+2d_o}{2}, b = L + a$ $k_p = 0.098 \frac{1}{\alpha} \pi D i_{corr}$ $\alpha = 0.57$	a=Inner radius b=Outer radius d_0 =Void thickness of concrete (m) ν_c =Psisson's ratio of concrete(0.18) E_{ef} =Effective elastic modulus(MPa) ρ_{cor} = Density of corrosion product k_p = rate of corrosion i_{corr} = Corrosion current density (μA/cm^2) W_{crit} =Critical amount of corrosion products W_{st} =Mass of corroded steel ρ_{st} =Steel density	Liu and Weyers[1998]

Table 5: (Continued) Prediction method for initiation, depassivation, and propagation time due to corrosion

ME	$t_{corr} = \frac{\delta \rho_{st} ZF}{A i_{corr}}$ $\delta = \frac{A i_{corr} t}{\gamma Z F}$	A = Atomic weightof iron（56g） Z = Ionic valance F = Farady constant ρ_{st} = Steel density γ = Material density（g/cm^3） i_{corr} = Corrosion current density（A/cm^2）	Mangat and Elgarf[1999]
CW	$t_{corr} = 2 \sim 5 years$	The average value of t_{corr} based on the CW method was adopted in this study, i.e., t_{corr}=3.5year	Cady and Weyers[1984]

$12.5 \times 10^{-3} mm$, $\nu_c = 0.18$ and $\varphi_{cr} = 2.0$[Bazant (1979)]. The results of calculation are listed in Tables 6 and 7. Based on Eq. (3), the maximum and minimum service live for each structural component of the existing Wann-fwu bridge and Chorng-ching viaduct are also respectively listed in Tables 6 and 7, i.e., $t_{max} = t_{c,max} + t_{p,max} + t_{corr,max}$ and $t_{min} = t_{c,min} + t_{p,min} + t_{corr,min}$. Based on Eq. (3), Figs. 6 and 8, and Tables 6 and 7, the results obtained from the service life prediction for the Wann-fwu bridge and Chorng-ching viaduct of 147 and 116 years in Taipei are acceptable. This means that the service life prediction for the corresponding bridge or viaduct coincide with $t_{min} \leq t \leq t_{max}$.

Since the prediction methods use different parameters for predicting the values of t_{corr}, the predicted results certainly have a big difference. The values of t_{corr} predicted by the LW method (see Table 5) for these existing bridge or viaduct are maximum. The value of t_{corr} predicted by the CW method (see Table 5) for the Wann-fwu bridge is minimum while the value of t_{corr} predicted by the Bazant method for the Chorng-ching viaduct is minimum. Generally speaking, the predicted values of t_{corr} obtained by the LW and ME methods (see Table 5) are very approached for these two bridge/viaduct. It is remarkable from Table 5 that both the LW and ME methods consider more parameters than the other methods. The values of t_{corr} calculated by the LW and ME methods are larger 3∼4 times than those results predicted by the other methods. If take the average of t_{corr} values predicted by the five prediction methods, the average t_{corr} values of the Wann-fwu bridge and Chorng-ching viaduct are 10.15 and 7.5 years, respectively.

Table 6: Service life prediction for Wann-fwu bridge

Test point No.	Corrosion current density i_{corr} ($\mu A/cm^2$)	t_c (yrs)					t_p (yrs)				t_{corr} (yrs)				Service life prediction (yrs)	
		Fick's law	Guirguis	Hookham	AJMF k=0.1	AJMF K=0.545	Bazant	Fang (s)	Chang (p+s)	Bazant	Modified Bazant	LW	ME	CW	t_{max}	t_{min}
A	0.45	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	4.4	5.8	9.98	8.27	3.5	156.74	36.73
B	0.16	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	4.4	5.8	28.08	23.26	3.5	174.84	36.73
C	0.18	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	4.4	5.8	24.96	20.68	3.5	171.72	36.73
D	0.52	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	4.4	5.8	8.64	7.16	3.5	155.4	36.73
E	0.36	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	4.4	5.8	12.48	10.34	3.5	159.24	36.73
F	0.23	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	3.78	5.22	19.54	16.18	3.5	166.3	36.73
G	0.17	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	3.78	5.22	26.43	21.9	3.5	173.19	36.73
H	0.31	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	3.78	5.22	14.49	12.01	3.5	161.25	36.73
I	0.23	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	3.78	5.22	19.54	16.18	3.5	166.3	36.73
J	0.36	32.77	30.52	119.22	45.26	43.11	9.18	27.54	2.71	3.78	5.22	12.48	10.34	3.5	159.24	36.73
K	0.48	12.76	19.08	50.32	31.19	25.11	3.59	10.76	1.06	5.42	6.77	9.36	7.75	3.5	70.44	23.64
L	0.15	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.85	7.31	29.95	24.81	3.5	254.26	45.9
M	0.11	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.85	7.31	40.85	33.84	3.5	265.16	45.9
N	0.17	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.85	7.31	26.43	21.9	3.5	250.74	45.9
O	0.21	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.85	7.31	21.4	17.72	3.5	245.71	45.9
P	0.18	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.44	6.92	24.96	20.68	3.5	249.27	45.9
Q	0.14	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.44	6.92	32.09	26.59	3.5	256.4	45.9
R	0.27	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.44	6.92	16.64	13.79	3.5	240.95	45.9
S	0.19	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.44	6.92	23.65	19.59	3.5	247.96	45.9
T	0.68	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.44	6.92	6.61	5.47	3.5	231.23	45.9
U	0.79	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	5.44	6.92	5.69	4.71	3.5	231.23	45.9
Average	**0.31**	**40.38**	**33.61**	**145.49**	**49.92**	**48.13**	**11.37**	**34.12**	**3.36**	**4.87**	**6.32**	**19.73**	**16.34**	**3.5**	**192.01**	**40.47**

Table 7: Service life prediction for Chorng-ching viaduct

Test point No.	Corrosion current density i_{corr} ($\mu A/cm^2$)	t_c (yrs)					t_p (yrs)				t_{corr} (yrs)				Service life prediction (yrs)	
		Fick's law	Guirguis	Hookham	AJMF		Bazant	Fang (s)	Chang (p+s)	Bazant	Modified Bazant	LW	ME	CW	$t_{\max}$	$t_{\min}$
					k=0.1	K=0.545										
A	0.401	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	1.87	3.65	11.2	9.28	3.5	235.51	44.27
B	0.396	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	1.87	3.65	11.35	9.4	3.5	235.66	44.27
C	0.328	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	1.87	3.65	13.7	11.35	3.5	238.01	44.27
D	0.354	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	1.87	3.65	12.69	10.51	3.5	237	44.27
E	0.319	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	1.87	3.65	14.08	11.67	3.5	238.39	44.27
F	0.325	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	1.87	3.65	13.82	11.45	3.5	238.13	44.27
G	0.261	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	2.3	3.86	17.21	14.26	3.5	241.52	44.7
H	0.286	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	2.3	3.86	15.71	13.02	3.5	240.02	44.7
I	0.317	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	2.3	3.86	14.17	11.74	3.5	238.48	44.7
J	0.295	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	2.3	3.86	15.23	12.62	3.5	239.54	44.7
K	0.362	12.76	19.08	50.32	31.19	25.11	3.59	10.76	1.06	2.69	4.82	12.41	10.28	3.5	73.49	16.51
L	0.266	12.76	19.08	50.32	31.19	25.11	3.59	10.76	1.06	2.69	4.82	16.89	14	3.5	77.97	16.51
M	0.282	12.76	19.08	50.32	31.19	25.11	3.59	10.76	1.06	2.69	4.82	15.93	14	3.5	77.01	16.51
N	0.294	50.76	38.16	181.28	56.46	55.47	14.34	43.09	4.24	2.69	4.82	15.28	12.66	3.5	239.65	45.09
O	0.263	50.76	38.16	181.28	56.46	55.47	14.34	43.09	4.24	3.83	5.64	17.08	14.15	3.5	241.45	45.9
P	0.314	12.76	19.08	50.32	31.19	25.11	3.59	10.76	1.06	3.83	5.64	14.31	11.86	3.5	75.39	17.32
Q	0.267	12.76	19.08	50.32	31.19	25.11	3.59	10.76	1.06	3.83	5.64	16.83	13.94	3.5	77.91	17.32
R	0.298	12.76	19.08	50.32	31.19	25.11	3.59	10.76	1.06	3.83	5.64	15.08	12.49	3.5	76.16	17.32
S	0.276	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	2.96	4.98	16.28	13.49	3.5	240.59	45.36
T	0.321	50.76	38.16	181.28	56.46	55.47	14.34	43.03	4.24	2.96	4.98	14	11.6	3.5	238.31	45.36
Average	**0.311**	**39.36**	**32.43**	**141.99**	**48.87**	**46.36**	**11.12**	**33.36**	**3.29**	**2.62**	**4.46**	**14.66**	**12.19**	**3.5**	**190.01**	**36.38**

6 Conclusions

The two service life models of RC structure and the analytical theory of load-carrying capacity service life have been stated in this paper. The service life models of existing RC structures consists of three phases, initiation (diffusion or carbonation) time, t_c, depassivation time, t_p, and propagation (corrosion) time, t_{corr}. The load-carrying capacity service life, $t_t = t_c + t_p + t_{corr}$, is major issue in this paper. The value of t_t predicted fro the relationship between reliability index and time at $\beta_{cr} = 1$ for the existing Wann-fwu bridge and Chorng-ching viaduct are 147 and 116 years, respectively. The $t_{t,min} = t_{c,min} + t_{p,min} + t_{corr,min}$ and $t_{t,max} = t_{c,max} + t_{p,max} + t_{corr,max}$ of the existing Wann-fwu bridge and Chorng-ching viaduct are 40.47 and 36.38 and 192.01 and 190.01 years, respectively. It is worthy to point out that the values of t_t predicted from the β_{cr} vs t are all lain within the range of $t_{t,min}$ and $t_{t,max}$. The results of this investigation may offer a basis for repair, strengthening, and demolition of existing RC bridges or viaducts. The prediction method proposed in this paper can be extended the application to other existing RC bridge or viaducts. Finally, we recommend that the crack width control service life is worth for future work.

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