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Author/s:

Chen, S;Duffield, C;Miramini, S;Nasim Khan Raja, B;Zhang, L

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# 1 **Life-cycle modelling of concrete cracking and reinforcement corrosion**

## 2 **in concrete bridges: A case study**

3 **Shilun Chen<sup>1</sup>, Colin Duffield<sup>2</sup>, Saeed Miramini<sup>3</sup>, Babar Nasim Khan Raja<sup>4</sup>, Lihai Zhang<sup>5,\*</sup>**

4 <sup>1</sup> Ph.D. student, Department of Infrastructure Engineering, B404, Engineering Block B, University of  
5 Melbourne, Victoria 3010, Australia. [shilunc@student.unimelb.edu.au](mailto:shilunc@student.unimelb.edu.au)

6  
7 <sup>2</sup> Professor., Department of Infrastructure Engineering, D212, Engineering Block D, University of Melbourne,  
8 Victoria 3010, Australia. [colinfid@unimelb.edu.au](mailto:colinfid@unimelb.edu.au)

9 <sup>3</sup> Dr.Eng./Lecturer, Department of Infrastructure Engineering, C501, Engineering Block C, University of  
10 Melbourne, Victoria 3010, Australia. [s.miramini@unimelb.edu.au](mailto:s.miramini@unimelb.edu.au)

11 <sup>4</sup> Ph.D. student, Department of Infrastructure Engineering, D402, Engineering Block D, University of  
12 Melbourne, Victoria 3010, Australia. [babarr@student.unimelb.edu.au](mailto:babarr@student.unimelb.edu.au)

13 <sup>5</sup>. \*Associate Professor., Department of Infrastructure Engineering, D210, Engineering Block D, University  
14 of Melbourne, Victoria 3010, Australia. [lih Zhang@unimelb.edu.au](mailto:lih Zhang@unimelb.edu.au) (Corresponding author)

15

### 16 **Abstract**

17 The development of effective life cycle management strategies for transport infrastructure assets  
18 is of importance for meeting the defined public policies and levels of service. In the last decades,  
19 much progress has been made in assessing the life-cycle performance of bridges using reliability-  
20 based approaches. However, the goal of developing a comprehensive life-cycle performance  
21 assessment framework for bridges has not been fully achieved. This is due to the uncertainties  
22 surrounding model parameters as well as the correlation between these parameters (e.g. the  
23 complex correlation between the reinforcement corrosion and the concrete cracking). It becomes  
24 more challenging due to the limited access to bridge inspection data by bridge research  
25 communities resulting from confidentiality issues. Using a typical highway concrete bridge as a  
26 case study, the present study systematically investigated the impact of concrete crack induced  
27 reinforcement corrosion on the serviceability of concrete bridges by developing an engineering  
28 reliability-based approach involving an auto-regressive crack propagation model and a steel  
29 corrosion prediction model. The model parameters were calibrated using the eight-year inspection

30 data of an operating bridge. The influence of different external environments in the reinforcement  
31 corrosion, ultimately the residual life of the bridges, was also investigated through conducting a  
32 series of parametric studies. Based on the collected bridge inspection data, the model results  
33 predict that, although the surface crack of a RC bridge is repairable through periodic maintenance,  
34 the corrosion of the steel bars in the bridge still continues over time with a corrosion rate which  
35 depends on different maintenance intervention cycle periods ( $T_{\text{cycle}}$ ). For example, reducing  $T_{\text{cycle}}$   
36 from 12 years to 4 years could potentially prolong the service life of the bridge by around 15 years.  
37 The developed model could assist bridge managers to estimate the optimal  $T_{\text{cycle}}$  to prolong the  
38 service life of bridges.

39 **Keywords**

40 Reinforced concrete bridges; concrete cracks; steel corrosion; engineering reliability analysis;  
41 first-order second-moment (FOSM); auto-regressive model

## 42 1. Introduction

43 As one of the fundamental transport infrastructures, bridges continuously support social and  
44 economic development of the world. The consistently operational performance of bridges plays a  
45 critical role for both public and private sectors. Bridge inspection, maintenance and repair become  
46 daily routines for maintaining the health of bridges [1]. However, the service life of many bridges  
47 in the world is expected to be less than their design life due to the continuous deterioration of  
48 bridges induced by daily traffic loading and environmental conditions [2, 3]. Therefore, it becomes  
49 increasingly important to maintain and repair the deteriorated bridges in a timely and cost-effective  
50 manner.

51 Concrete is one of the commonly used construction materials for bridges. Concrete cracking is  
52 generally used as one of the primary indicators for assessing the severity of reinforced concrete  
53 (RC) bridge deterioration [4]. Cracks in RC bridges can be classified into two main types, *i.e.*  
54 loading cracks and non-loading cracks [5]. While the non-loading cracks are mainly caused by  
55 environmental factors (*e.g.* cyclic changes in temperature and moisture), the loading cracks are  
56 caused by external loading imposed on the bridges (*e.g.* traffic loading, earthquake, etc.) [6, 7].  
57 The crack development in RC bridges over time could result in the corrosion of reinforcements in  
58 concrete, delamination, which affects the mechanical properties of concrete material, and  
59 ultimately reduce the service life of a bridge [8]. Most importantly, corrosion of steel bars in  
60 concrete induced by cracking may lead to the failure of the bridges. Concrete cracking and  
61 corrosion of reinforcing steel bars have a bilateral influence on each other [9]. During the corrosion  
62 process of steel bars, the hydrated ferric oxide (*i.e.* rust), which is a larger substance than the  
63 original ferrous hydroxide, causes the expansion of the internal space between concrete and  
64 reinforcing steel bars. This produces a higher inner pressure in concrete which results in the  
65 formation of cracks around the interface between concrete and steel bars [10]. As the corrosion  
66 proceeds, cracks can propagate from the inner surface to the external surface, *i.e.* so-called

67 corrosion-induced cracking. Although a high alkaline environment in concrete can hinder the  
68 corrosion of steel bars through the formation of a passive layer, cracks in concrete could provide  
69 a pathway for chloride ions, acidic ions, carbon dioxide and other substances to corrode the steel  
70 bars in concrete [11]. In addition, under sulfate attack, formation of ettringite could happen in  
71 concrete [12], ultimately lead to concrete cracking due to expansion of ettringite within the  
72 concrete [13]. Previous experimental studies have shown that the steel bar corrosion in concrete  
73 depends on crack characteristics [14], such as width, types and frequency, particularly at the initial  
74 stage of corrosion.

75 Several theoretical studies have been carried out to investigate the relationship between corrosion  
76 of steel bars and crack development in RC structure members [15-19]. Based on the relationship  
77 between residual load capacity, surface crack width and corrosion of steel bars of a RC structure,  
78 an empirical model is proposed to predict the structural performance of corroded RC structures  
79 [20]. In addition, a theoretical model for crack widths has been developed to assess the  
80 serviceability of the RC structures based on the concept of fracture energy under combined  
81 reinforcement corrosion and applied loading effects [21]. Further, a mathematical model was  
82 proposed to predict the service life of the RC bridge structures exposed to chloride environment  
83 based on Fick's second law [22]. Moreover, to account for the uncertainties of structural  
84 deterioration induced by the combined effects of progressive loading (corrosion and cracking) and  
85 extreme loading (earthquake and impact), a stochastic model was used to predict the time-  
86 dependent performance of infrastructures using structural reliability analysis with the aim of  
87 improving decision-making for maintenance and replacement of infrastructures [2]. Furthermore,  
88 because of the importance of failure probability assessment for structural systems which have  
89 different uncertain inputs [23], a probabilistic model regarding structural deterioration subject to  
90 corrosion was developed to predict the probability of failure (PoF) of reinforcing steels in a RC  
91 bridge deck in a marine environment throughout its 75 years' service life [24].

92 Although much research work has been done in last decades to develop the reliability-based life-  
93 cycle performance model for bridges [25-27], the accuracy of the realistic model forecasting of  
94 future bridge performance is much dependent on the accurate determination of the model  
95 parameters. This is rather challenging due to a range of uncertainties surrounding these  
96 parameters resulting from the limited measurement data as well as the correlation between these  
97 parameters, e.g. the corrosion rate of reinforcing steel bars is closely correlated to the crack  
98 propagation of concrete. Although autoregressive processes have been implemented to predict  
99 the long-term deterioration of concrete structures [28], the determination of the model coefficients  
100 depends on the collection of a large amount of historical data. In addition, even if several reliability-  
101 based mathematical model have been established to simulate the reinforcement corrosion of  
102 highway bridges [29], the chloride diffusion coefficient and corrosive rate of reinforcements are  
103 significantly affected by the crack propagation of concrete cover over time. Without historical crack  
104 measurement data, the long-term reinforcement corrosion behavior cannot be correctly modelled.  
105 Using the historical data collected from bridge inspection, the purpose of this study is to develop  
106 an engineering reliability analysis (ERA)-based framework to assess the life-cycle performance  
107 of RC bridges subject to reinforcement corrosion by concrete cracking. The current research  
108 represents the first step towards fundamental understanding of the concrete cracking induced  
109 reinforcement corrosion, which could potentially contribute to the development of effective bridge  
110 maintenance strategies.

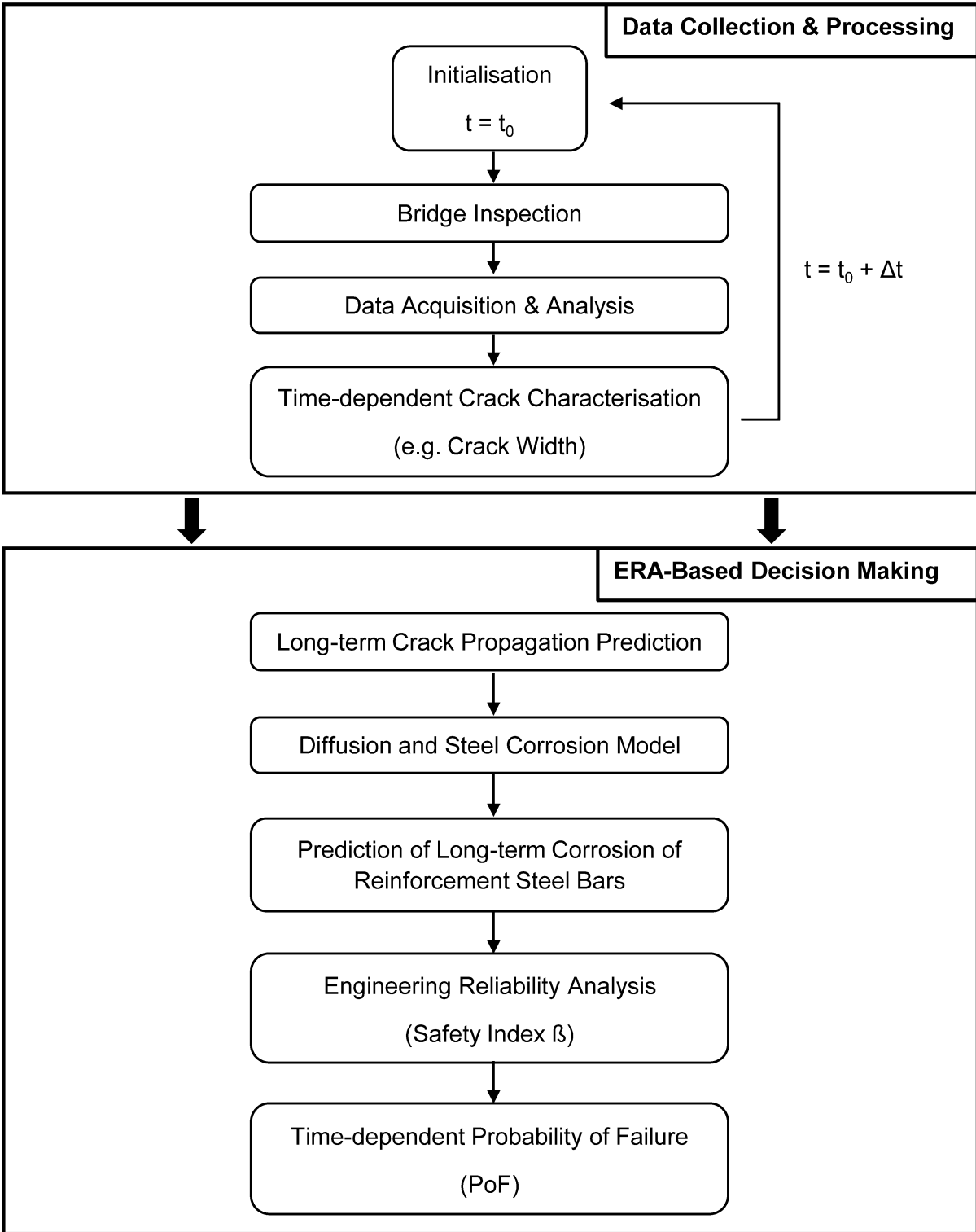
## 111 **2. Method**

### 112 **2.1 ERA-based framework for life-cycle condition assessment of bridges**

113 Figure 1 shows the details of developed ERA-based framework for assessing the life-cycle  
114 performance of the RC bridges which are gradually deteriorating resulting from steel bar corrosion  
115 due to the development of concrete cracking. First, the initial crack characteristics (e.g. crack  
116 width) at time  $t_0$  are quantified through data collection via bridge inspection. Then, the

117 development of crack characteristics over time is captured from next bridge inspection at time  
118  $t_0 + \Delta t$  ( $\Delta t$ : bridge inspection time interval). Based on collected historical data, the long-term crack  
119 propagation is predicted using an auto-regressive (AR) approach (Equation (1)) [30, 31]. The  
120 accuracy of autoregression model prediction could be improved with the collection of more crack  
121 measurement data over time. Using chloride diffusion model and steel corrosion model (Equation  
122 (8)-(14)), the long-term corrosion behaviour of steel bars can be estimated based on the time-  
123 dependent crack characteristics (see Figure 2). Finally, the change reliability index ( $\beta$ ) of the  
124 bridge structures can be predicted based on the codes of practice requirements of bridge design  
125 and construction (e.g. maximum allowable crack width,  $W_0$ ).

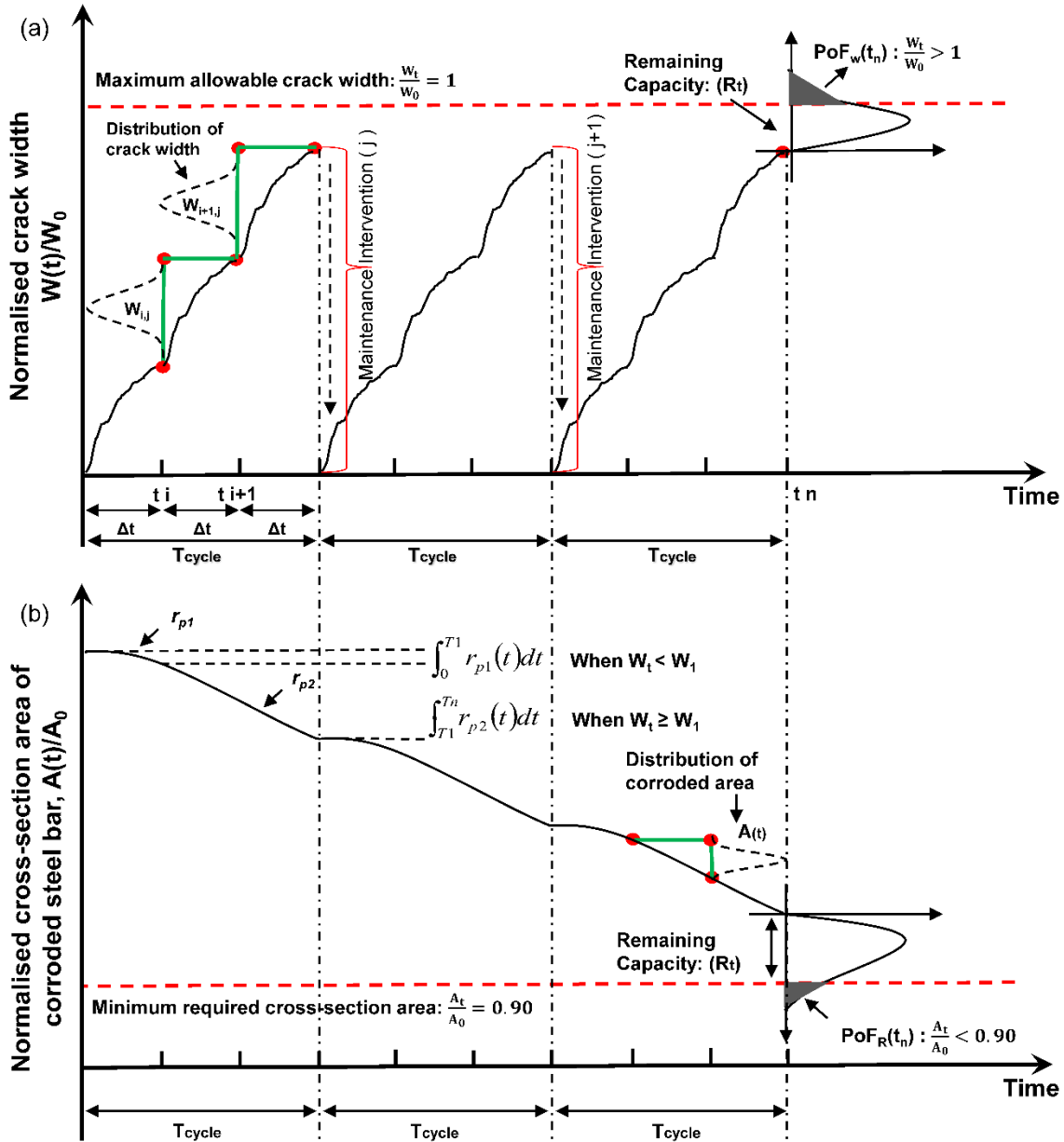
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127

128 Figure 1 – Proposed framework for life-cycle condition assessment of reinforced concrete  
 129 structures using engineering reliability analysis. ERA, engineering reliability analysis.

130 As shown in Figure 2, the developed framework could be implemented to assess the PoF of a  
131 bridge under different maintenance intervention cycle periods ( $T_{\text{cycle}}$ ). It shows the time-dependent  
132 crack propagation under different routine bridge inspection intervals ( $\Delta t$ ). When crack width  $W_t$   
133 approaches  $W_0$ , maintenance intervention is required for crack treatment under a certain  $T_{\text{cycle}}$ .  
134 As shown in Equations (12) - (14), it is assumed that the rate of the reinforcement corrosion  
135 depends on the crack width. Initially (e.g. short time after maintenance,  $t < T_1$ ), the rate of  
136 corrosion is relatively low when the concrete crack width is relatively small ( $W_t < W_1$ ). Over time,  
137 the rate of corrosion accelerates when the crack width is over a certain threshold (i.e.  $W_0$ ). The  
138 details are described in Figure 2.



139

140 Figure 2 – (a) An engineering reliability analysis (ERA)-based approach for assessing crack  
 141 propagation under maintenance intervention cycle period  $T_{\text{cycle}}$  ( $W_0$ : maximum allowable crack  
 142 width,  $W_1$ : crack width inducing higher diffusion coefficient and corrosion rate,  $\Delta t$ : inspection time  
 143 interval and  $T_{\text{cycle}}$ : maintenance intervention cycle period); (b) an ERA-based approach for  
 144 assessing reinforcement steel corrosion in concrete under crack maintenance intervention cycle

145 period  $T_{\text{cycle}}$  ( $r_{p1}$ : steel corrosion rate when  $W_t < W_1$ ,  $r_{p2}$ : steel corrosion rate when  $W_t \geq W_1$  &  $A_0$ :  
146 initial cross-section area of steel bar).

## 147 **2.2 Auto-regressive model for crack propagation prediction**

148 While recent advance in artificial intelligence provide an innovative way to model crack in concrete  
149 bridges [32], a large amount of training data and the selection of the optimal algorithm are required.  
150 Considering the limited amount of inspection data available in this study, the autoregressive  
151 model was implanted to predict the long-term crack propagation due to its capability of modelling  
152 time-varying random process based on historical data [33]. The AR model has the capability of  
153 univariately forecasting time-series models which demonstrate the relationship between a single  
154 variable and its corresponding past values [34]. As a first step, by assuming that there is little  
155 change in environmental and loading conditions in recent years, the long-term prediction of crack  
156 width in concrete can be defined as a stationary process. That is,

$$157 \quad W_t = \beta_1 W_{t-1} + \beta_2 W_{t-2} + \dots + \beta_p W_{t-p} + \varepsilon_t \quad (1)$$

158 where  $P$ , as the order of auto-regressive process (AR( $p$ )), represents the number of past values  
159 which will be included.  $P$  is defined by the highest time-lag order coefficient which is less than  
160 0.05. At a given time  $t$ ,  $\varepsilon_t$  is the random distribution of uncorrelated error with zero mean and  
161 variance, *i.e.*  $\{\varepsilon_t\} \sim (0, \sigma^2)$ .  $\beta$  is the AR coefficient with  $|\beta| < 1$  [31, 35].

162 Hence, the predicted concrete crack width at time  $t$  can be expressed as,

$$163 \quad W_t = \sum_{i=0}^p \beta_i W_{t-i} + \varepsilon_t \quad (2)$$

164 By applying the backward shift operator  $\pi(B)$ ,

$$165 \quad \pi(B) = \sum_{i=0}^p \beta_i B^i \quad (3)$$

166 Equation (2) can be written as,

167 
$$W_t = \pi(B)W_{t-i} + \varepsilon_t \quad (4)$$

168 With consideration of periodic crack maintenance intervention, we obtain,

169 
$$W_{t,i,j} = \begin{cases} \sum_{i=0,j=0}^p \beta_{i,j} W_{t-i,j} + \varepsilon_t & t_j < t < t_{j+1} \\ 0 & t = t_j \end{cases} \quad (5)$$

170 where  $j$  represents the  $j$ -th number of crack maintenance intervention after the  $i$ -th year of crack  
 171 inspection. Using the indicator function  $\mathbf{1}_{t_j}$  as shown below,

172 
$$\mathbf{1}_{t_j} = \begin{cases} 1 & t_j < t < t_{j+1} \\ 0 & t = t_j \end{cases} \quad (6)$$

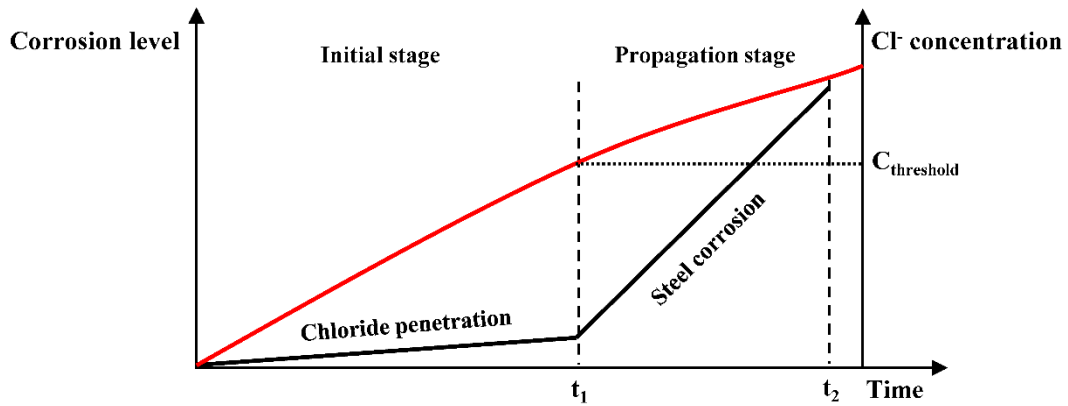
173 The crack width with time under different maintenance interventions can be expressed as,

174 
$$W_{t,i,j} = \sum_{i=0,j=0}^p (\beta_{i,j} W_{t-i,j} + \varepsilon_t) * \mathbf{1}_{t_j} \quad (7)$$

### 175 **2.3 Predicting corrosion of reinforcement steel bars in concrete**

176 As shown in Figure 3, throughout the service life, the corrosion process of RC structures can be  
 177 divided to the following two stages:

- 178 • Initial stage ( $0 < t < t_1$ ): At this stage, chloride ions gradually diffuse into concrete from  
 179 external environment until the concentration of chloride ions on the surface of steel bars  
 180 in concrete reaches to the maximum allowable threshold value ( $C_{\text{threshold}}$ ) defined in design  
 181 guidelines [11]. The thickness of concrete cover, concentration of chloride ions on  
 182 concrete surface and diffusion coefficient of chloride ions play an important role in  
 183 corrosion resistance at this stage.
- 184 • Propagation stage ( $t > t_1$ ): At this stage, significant chloride ions induced corrosion of steel  
 185 bars occurs and could cause the local damage of the RC structures [36, 37].



186

187 Figure 3 – Corrosion process of reinforcement steel bars in concrete [36, 37]

188 **Initial stage:**

189 *The transport of mass through diffusive means is generally modelled using the Fick's second law*  
 190 *[38, 39]. That is,*

191 
$$C(x, t) = kt \left\{ \left( 1 + \frac{x^2}{Dt} \right) \operatorname{erf} \left( \frac{x}{2\sqrt{Dt}} \right) - \left( \frac{x}{\sqrt{\pi Dt}} \right) e^{-\frac{x^2}{4Dt}} \right\} \quad (8)$$

192 In this study,  $C(x, t)$  is chloride concentration at depth  $x$  (mm) of concrete cover at time  $t$  (year).  $D$   
 193 is the diffusion coefficient ( $\text{mm}^2/\text{year}$ ) which is much dependent on the porosity of the concrete  
 194 and the volume fraction of aggregate in concrete [11, 12].

195 **Propagation stage:**

196 At this stage, the significant corrosion process of steel bars in concrete is triggered as the chloride  
 197 concentration reaches to the threshold value ( $C_{\text{threshold}} = 0.68\%$  [11, 40-42]). In this study, the mass  
 198 loss of steel bars in concrete was modelled using mathematical approach of Du et al. (2005) [9],  
 199 which divides the corrosion of steel bars in concrete into two stages, i.e. (1) chemical reaction  
 200 stage, and (2) the stage involving current flow with anode and cathode on surface of reinforcement

201 steel bars, leading to the transformation of iron into rust which sticks onto the surface of steel bars  
 202 [43]. The rate of mass loss of steel bar ( $Q_{\text{corr}}$ ) can be defined as,

$$203 \quad Q_{\text{corr}} = \frac{M_0 - M_1}{M_0} \quad (9)$$

204 where  $M_0$  is the initial weight of steel bar before corrosion started and  $M_1$  is the weight of corroded  
 205 steel bar. Using the empirical corrosion model proposed by [9], Equation (8) can be modified as,

$$206 \quad Q_{\text{corr}} = 0.046 * \frac{r_{\text{corr}}}{d} * t \quad (10)$$

207 where  $r_{\text{corr}}$  is the corrosion rate of steel bar in a RC structure under certain environmental  
 208 conditions ( $\mu\text{A}/\text{cm}^2$ ),  $d$  is the diameter of non-corroded steel bar (mm). Thus, the time-dependent  
 209 cross-section area of the corroded steel bars in concrete structure can be expressed as [9],

$$210 \quad A_{t_{i,j}} = A_0(1 - Q_{\text{corr}}) \quad (11)$$

211 where  $A_0$  is the initial cross-section area of non-corroded steel bar.

212 Previous studies have demonstrated that there is a positive correlation between the corrosion rate  
 213 of bars in concrete and the surface crack width as shown in Figure 2 [9]. Let  $\mathbf{1}_{(W_t)}$  be an indicator  
 214 function for corrosion rate of steel bar under different crack widths on a concrete surface. It is  
 215 assumed that the corrosion rate of a steel bar is  $r_1$  when crack width  $W_t$  is less than  $W_1$ , while the  
 216 corrosion rate of a steel bar is  $r_2$  when  $W_t$  is greater than  $W_1$ . The indicator function  $\mathbf{1}_{(W_t)}$  can be  
 217 written as,

$$218 \quad \mathbf{1}_{W_{t,1}} = \begin{cases} 1 & 0 < W_{t,i,j} < W_1 \\ 0 & W_{t,i,j} \geq W_1 \end{cases} \quad (12)$$

$$219 \quad \mathbf{1}_{W_{t,2}} = \begin{cases} 0 & 0 < W_{t,i,j} < W_1 \\ 1 & W_{t,i,j} \geq W_1 \end{cases} \quad (13)$$

220 Combining Equations (10) and (12), the time-dependent cross-section area of the corroded steel  
 221 bars in concrete structure due to the development of crack width can be expressed as,

$$222 \quad A_{t_{i,j}} = A_0 * \left\{ 1 - \int_0^{\infty} \left[ (0.046 * \frac{r_1}{d} * \mathbf{1}_{W_{t,1}}) + (0.046 * \frac{r_2}{d} * \mathbf{1}_{W_{t,2}}) \right] dt \right\} \quad (14)$$

## 223 **2.4 Life-cycle performance assessment of the bridges using ERA**

224 The time dependent PoF of the bridges due to steel bar corrosion can be predicted using ERA.  $\beta$   
 225 can be calculated using first-order second-moment (FOSM) [44] with consideration of the first two  
 226 moments which are mean and standard deviation of the random variables. In FOSM, the non-  
 227 linear performance function is required to be linearised [45-47]. Considering  $(X_1, X_2, \dots, X_n)$  as  
 228 random variables, the non-linear function  $Z$  can be expressed as,

$$229 \quad Z = f(X) = f(X_1, X_2, \dots, X_n) \quad (15)$$

230 Taylor series expansion with second-order terms leads to,

$$231 \quad Z = f(\mu_X) + \sum_{i=1}^n \frac{\partial f}{\partial X_i} + \frac{1}{2} \sum_{i=1}^n \sum_{j=1}^n \frac{\partial^2 f}{\partial X_i \partial X_j} (X_i - \mu_{X_i})(X_j - \mu_{X_j}) + \dots \quad (16)$$

232 where  $\mu_X$  are mean of these random variables  $(\mu_{X_1}, \mu_{X_2}, \dots, \mu_{X_n})$ . The second-order approximated  
 233 mean of  $Z$ , *i.e.*  $E(Z)$  can be obtained,

$$234 \quad E(Z) = f(\mu_{X_1}, \mu_{X_2}, \dots, \mu_{X_n}) + \frac{1}{2} \sum_{i=1}^n \sum_{j=1}^n \frac{\partial^2 f}{\partial X_i \partial X_j} \text{Cov}(X_i, X_j) \quad (17)$$

235 where  $\text{Cov}(X_i, X_j)$  is the covariance of  $X_i$  and  $X_j$ .

$$236 \quad \text{Cov}(X_i, X_j) = \rho_{X_i, X_j} * \sigma_{X_i} * \sigma_{X_j} \quad (18)$$

237 where  $\sigma_X$  is the standard deviation of these random variables  $(\sigma_{X_1}, \sigma_{X_2}, \dots, \sigma_{X_n})$ ,  $\rho_{X_i, X_j}$  is  
 238 coefficient of correlation between  $X_i$  and  $X_j$ .

239 In the FOSM analysis, the function Z is expanded by Taylor series expansion with first-order terms.

240 That is,

$$241 \quad Z = f(X_1, X_2, \dots, X_n) + \sum_{i=1}^n \frac{\partial f}{\partial X_i} (X_i - \mu_{X_i}) + \dots \quad (19)$$

242 By applying the first-order approximation, E(Z) can be defined as,

$$243 \quad E(Z) \approx f(\mu_{X_1}, \mu_{X_2}, \dots, \mu_{X_n}) \quad (20)$$

244 The variance of function Var(Z) with second-order approximation is,

$$245 \quad \text{Var}(Z) \approx \sum_{i=1}^n \left( \frac{\partial f}{\partial X_i} \right)^2 \sigma_{X_i}^2 + \sum_{i=1}^n \sum_{j=1}^n \frac{\partial^2 f}{\partial X_i \partial X_j} \text{Cov}(X_i, X_j) \quad (21)$$

246 As variance of function Z equals the square of standard deviation of Z, we obtain,

$$247 \quad \sigma_Z^2 = \text{Var}(Z) \quad (22)$$

248 Using the first-order approximation, the variance of Z is defined as,

$$249 \quad \sigma_Z^2 = \sum_{i=1}^n \left( \frac{\partial f}{\partial X_i} \right)^2 \sigma_{X_i}^2 \quad (23)$$

250 In present study, it is assumed that the  $W_0$  of a concrete crack in the RC bridge is defined

251 according to Bridge Design Standard (AS 5100.5 – Bridge Design, 2017). The time-dependent

252 concrete crack width is defined as  $W_{t_{i,j}}$  with mean and standard deviation under normal

253 distribution. Therefore, the cracking assessment can be analysed based on the difference

254 between  $W_{t_{i,j}}$  and  $W_0$ . The study of Du et al. (2007) [48] indicates that the reduction in ductility of

255 a RC beam could be significantly affected when the amount of reinforcement corrosion exceeds

256 10%. Thus, 10% loss of cross-section area of reinforcements is used as the maximum allowable

257 corrosion limit in this study. Hence, the “remaining concrete surface crack width capacity” ( $R_{W_{t_{i,j}}}$ )

258 and “remaining cross-section area of the corroded steel bars” ( $R_{A_{t_{i,j}}}$ ) can be expressed as,

259 
$$R_{W_{t_{i,j}}} = W_0 - W_{t_{i,j}} = f(X_1, X_2, \dots, X_n)$$

260 (24)

261 
$$R_{A_{t_{i,j}}} = A_0 - A_{t_{i,j}} = f(X_1, X_2, \dots, X_n)$$

262 Substituting Equations (6), (11) and (12) into Equation (23) leads to,

263 
$$R_{W_{t_{i,j}}} = W_0 - \sum_{i=0, j=0}^p (\beta_{i,j} W_{t-i,j} + \varepsilon_t) * \mathbf{1}_{t_j}$$

264 (25)

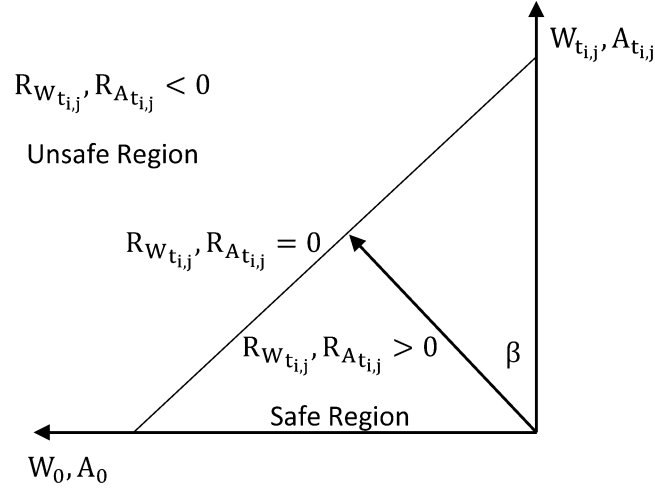
265 
$$R_{A_{t_{i,j}}} = A_0 - A_0 * \left\{ 1 - \int_0^{\infty} \left[ (0.046 * \frac{r_1}{d} * \mathbf{1}_{W_{t,1}}) + (0.046 * \frac{r_2}{d} * \mathbf{1}_{W_{t,2}}) \right] dt \right\}$$

266 where  $(X_1, X_2, \dots, X_n)$  are random variables, such as uncertainties of cracking width occurred  
 267 during data acquisition stage [49]. The change in value of  $R_{W_{t_{i,j}}}$  depends on the maintenance  
 268 intervention cycle period ( $T_{cycle}$ ). As shown in Figure 2, the probability of the width of the surface  
 269 crack ( $W_{t_{i,j}}$ ) over its allowable value ( $W_0$ ) as well as the cross-section area of a steel bar ( $A_{t_{i,j}}$ )  
 270 over its allowable value ( $A_0$ ) can be defined as [50],

271 
$$PoF_W = P(W_0 - W_{t_{i,j}} < 0) = P(R_{W_{t_{i,j}}} < 0)$$

272 (26)

273 
$$PoF_A = P(A_0 - A_{t_{i,j}} < 0.90A_0) = P(R_{A_{t_{i,j}}} < 0)$$



274

275

Figure 4 – Schematic diagram of reliability index.

276 As shown in Figure 4, the minimum distance from origin to the failure line is  $\beta$  which represents

277 the maximum value of PoF [51]. The point  $(R_{W_{t,i,j}}, R_{A_{t,i,j}} = 0)$  represents the design point. Thus,

278

$$\mu_{R_{W_{t,i,j}}} = \mu_{W_0} - \mu_{W_{t,i,j}}$$

279

$$\mu_{R_{W_{t,i,j}}} = \mu_{W_0 - \sum_{i=0,j=0}^p (\beta_{i,j} W_{t-i,j} + \varepsilon_t) * 1_{t_j}} \quad (27)$$

280

$$\sigma_{R_{W_{t,i,j}}} = \sqrt{\sigma_{W_0}^2 + \sigma_{W_{t,i,j}}^2}$$

281

282

$$\mu_{R_{A_{t,i,j}}} = \mu_{A_0} - \mu_{A_{t,i,j}}$$

283

$$\mu_{R_{A_{t,i,j}}} = \mu_{A_0 - A_0 * \left\{ 1 - \int_0^\infty \left[ (0.046 * \frac{r_1}{d} * 1_{W_{t,1}}) + (0.046 * \frac{r_2}{d} * 1_{W_{t,2}}) \right] dt \right\}} \quad (28)$$

284

$$\sigma_{R_{A_{t,i,j}}} = \sqrt{\sigma_{A_0}^2 + \sigma_{A_{t,i,j}}^2}$$

285 where  $\mu_{W_{t,i,j}}$  and  $\sigma_{R_{W_{t,i,j}}}$  are the mean and standard deviation of surface crack width, respectively.

286  $\mu_{R_{A_{t,i,j}}}$  and  $\sigma_{R_{A_{t,i,j}}}$  are the mean and standard deviation of cross-section area of the corroded steel

287 bars, respectively. Hence, engineering  $\beta$  of crack width and cross-section area of the corroded  
288 steel bars can be determined as,

$$\beta_{W_{t_{i,j}}} = \frac{\mu_{R_{W_{t_{i,j}}}}}{\sigma_{R_{W_{t_{i,j}}}}}$$

290 (29)

$$\beta_{A_{t_{i,j}}} = \frac{\mu_{R_{A_{t_{i,j}}}}}{\sigma_{R_{A_{t_{i,j}}}}}$$

291

292 In addition, the PoF of crack width and cross-section area of the corroded steel bars can be  
293 defined as,

$$PoF_W = 1 - \Phi(\beta_{W_{t_{i,j}}})$$

295 (30)

$$PoF_A = 1 - \Phi(\beta_{A_{t_{i,j}}})$$

296

297 where  $\Phi()$  is the cumulative distribution function of a standard normal random variable. The First  
298 Order Second Moment (FOSM) is widely used to analyze the load and resistance of the bridges  
299 [52]. As shown in Equation (28), the reliability index ( $\beta$ ) is defined based on the allowable crack  
300 width and reduction in the cross-sectional area of a steel bar (AS 5100.5 – Bridge Design, 2017).  
301 As the first estimate, the relationship between the loading imposed on the bridge and the concrete  
302 cracking is ignored in this study.

### 303 **3. Problem description**

304 The developed ERA-based framework was implemented to conduct life-cycle condition  
305 assessment of a RC highway bridge (Long Feng Xi Bridge, Chongqing, China) based on the  
306 historical inspection data of concrete surface crack width (Figure 5a). The present study mainly

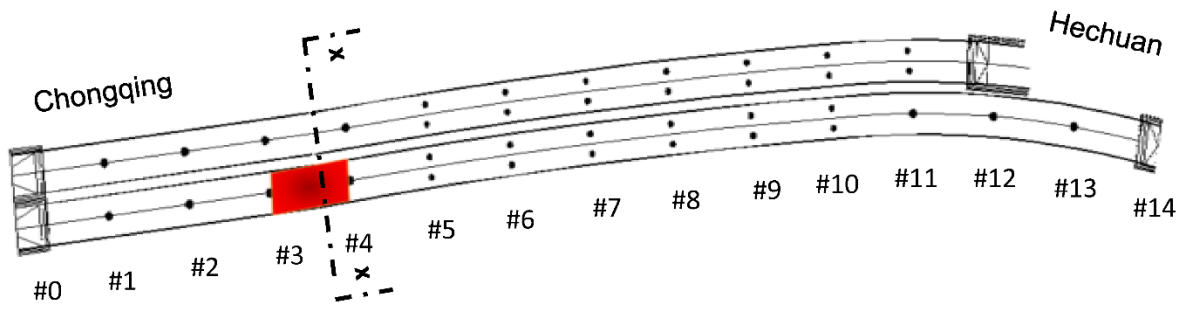
307 focuses on the No. 4 span of the bridge as shown in Figure 5 b-c. The developments of the 3  
308 cracks have been regularly monitored, and the image of the cracks are shown in Figure 5 d and  
309 e. The historical crack width data collected from 2011 to 2018 by Chongqing Expressway Group  
310 Company Ltd. were statistically analysed, and the long-term crack development was predicted  
311 using the AR model based on  $W_0 = 0.25$  mm (AS 5100.5 – Bridge Design, 2017). In this study,  
312 the relationship between the chloride penetration profile, corrosion rate of reinforcement and crack  
313 width was established based on Fick's 2nd law and mathematical model which ignores the  
314 variation of environment condition. To cater for the effects of different environmental conditions,  
315 a series of parametric studies were carried out to investigate the impact of reinforcement corrosion  
316 rate on the residual life of the bridge under normal and costal environments, respectively. The  
317 experimental studies of [53] revealed that the corrosion rate of steel bars is dependent on the  
318 experimental conditions, *i.e.* (1) the value of steel corrosion rate is  $r_1$  under non-exposed-to-air  
319 conditions ( $W_t \leq W_1$ ) and (2) the value of steel corrosion rate is  $r_2$  under exposed-to-air conditions  
320 ( $W_t > W_1$ ). Although day-night cycles, seasonal cycles, rain periods and extreme temperature  
321 cycles could affect the corrosion rate of steel bars in practice [54], it is assumed that the  
322 experimental conditions remain constant to simplify the complex corrosion processes in this study.  
323 Based on AS 5100.5 – Bridge Design (2017) and previous studies [53, 54], the  $W_0$  (AS 5100.5 –  
324 Bridge Design 2017), diffusion coefficient and corrosion rates [55-58] are shown in Table 1.



325

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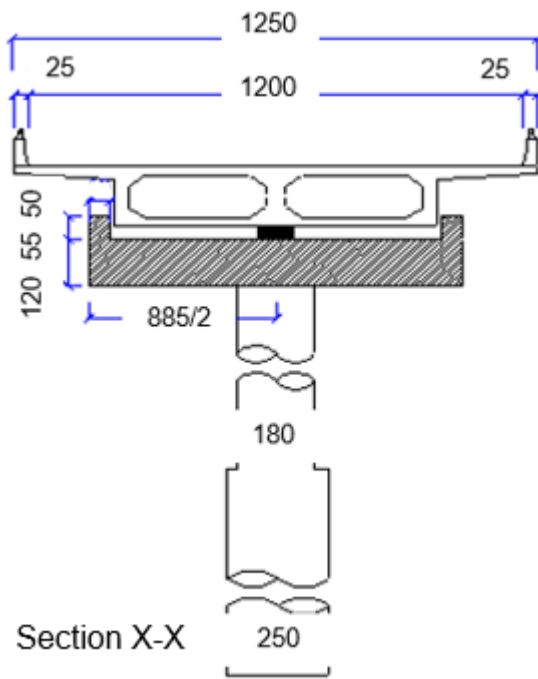
(a)



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(b)



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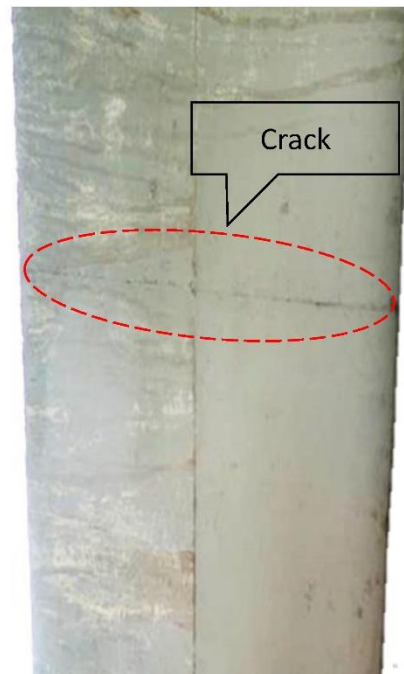
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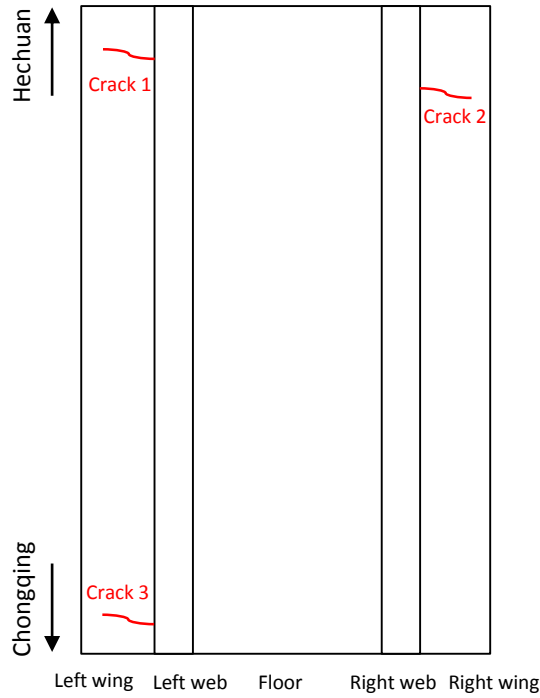
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(c)



(d)

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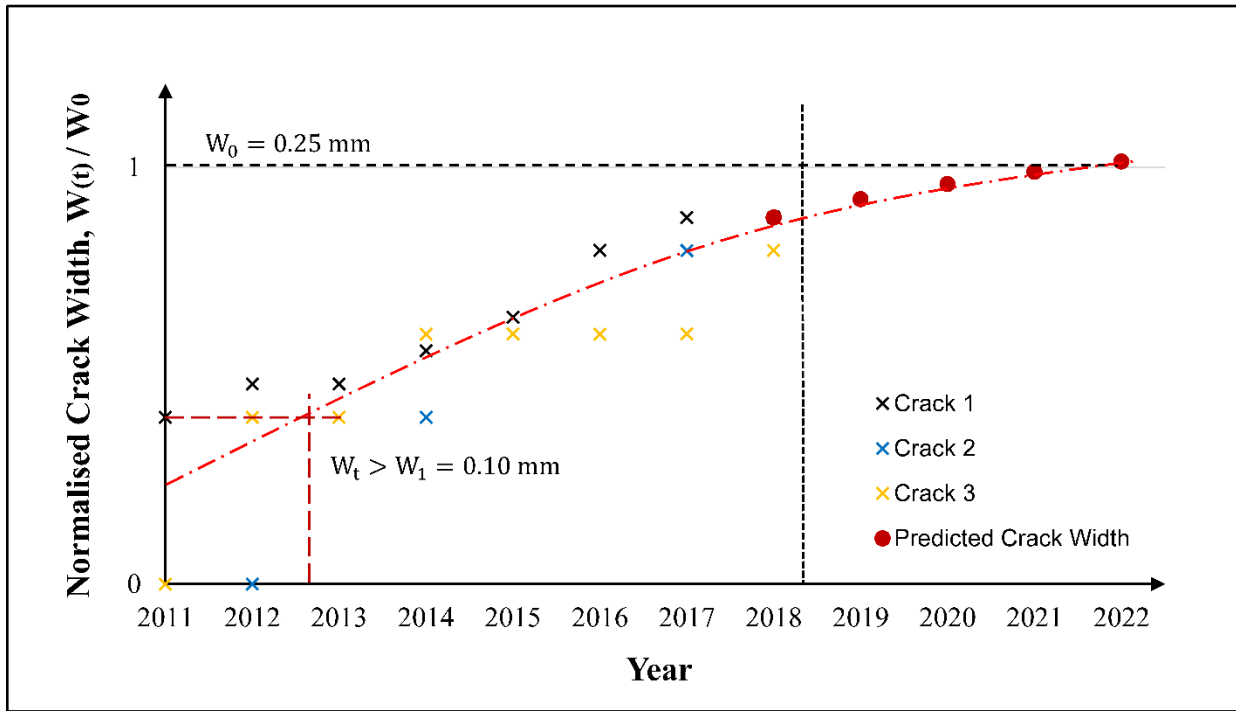
(e)

345 Figure 5 – Case study of Long Feng Xi Bridge, Chongqing, China: (a) bridge appearance, (b)  
346 section of the bridge with cracking investigated in this study, (c) the cross-section X-X, (d) cracking  
347 1 detail and (e) cracks' locations on No.4 span of the bridge.

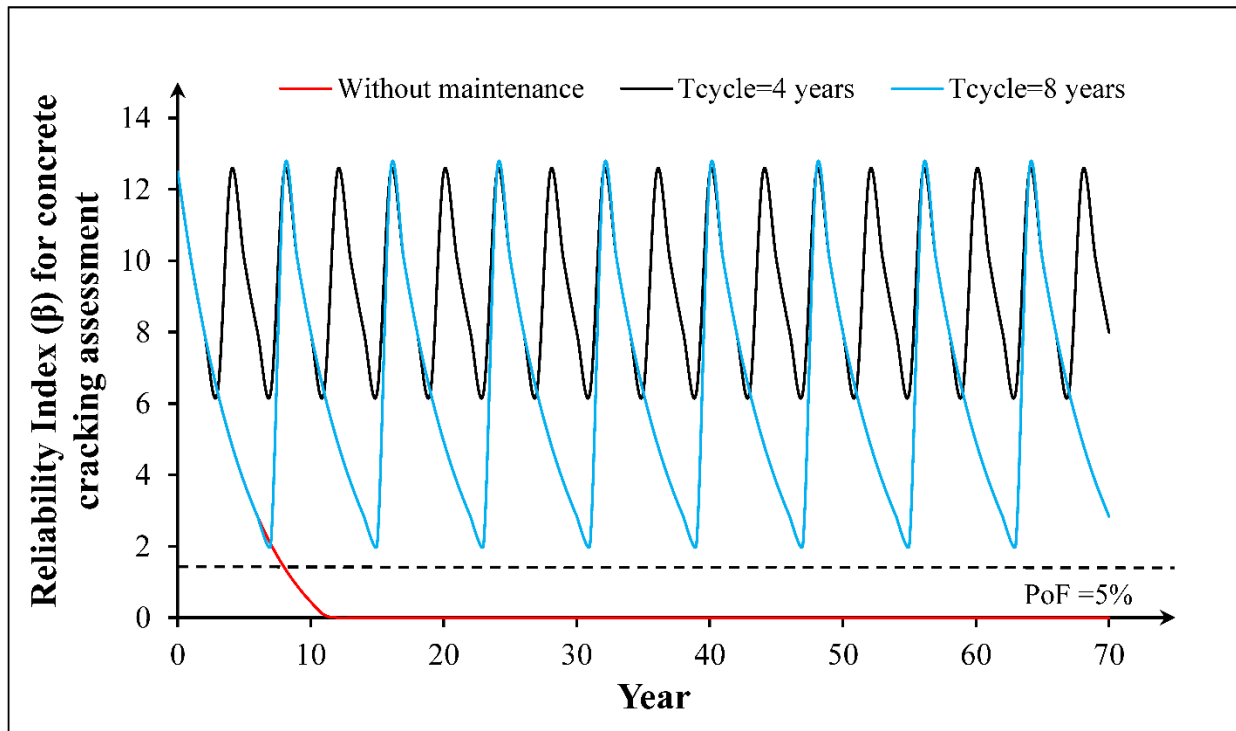
348 Table 1 – Value of crack width  $W_1$  causing steel bars exposed to air, different diffusion coefficients  
349 of chloride ions in concrete subject to different environmental conditions [55-58] and corrosion  
350 rates  $r_1$  and  $r_2$  of a reinforcement steel bar under normal and coastal environmental conditions in  
351 this study [53, 54].

Classification of assessment conditions	$W_1$ (mm)	$D_1: W_t < W_1$ ( $\text{mm}^2/\text{year}$ )	$D_2: W_t \geq W_1$ ( $\text{mm}^2/\text{year}$ )	$r_1: W_t < W_1$ ( $\mu\text{A}/\text{cm}^2$ )	$r_2: W_t \geq W_1$ ( $\mu\text{A}/\text{cm}^2$ )
Normal environmental conditions	0.1	63.08	189.24	0.5	2
Coastal environmental conditions	0.1	167.78	503.31	1	10

352



354 (a)



355 (b)

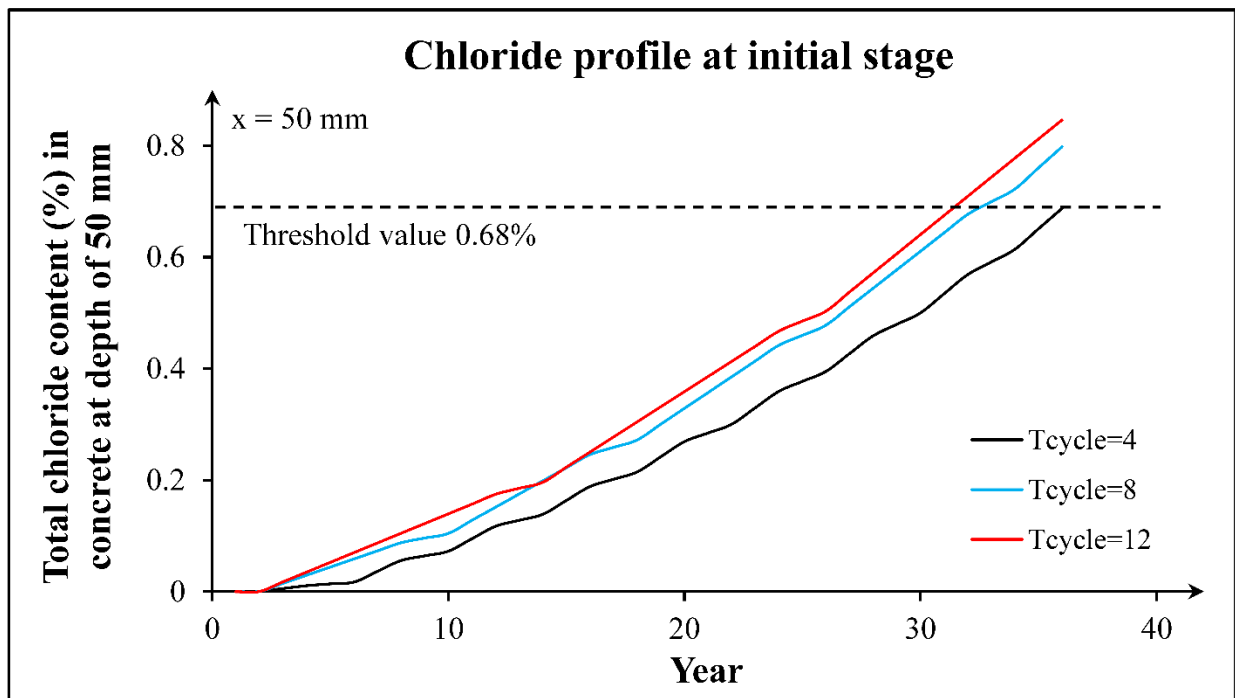
356

357 Figure 6 – (a) Time-dependent normalised crack width prediction using autoregression with 3  
358 cracks data on the same span of bridge,  $W_0$ : maximum allowable crack width (AS 5100.5 – Bridge  
359 Design, 2017); (b) time-dependent reliability index ( $\beta$ ) for concrete cracking assessment under  
360 different maintenance intervention cycle periods ( $T_{\text{cycle}}$ ). PoF, probability of failure.

361 As shown in Figure 6 (a), the long-term crack width is predicted using an autoregression approach  
362 based on the available eight-year crack width measurements of 3 different cracks obtained from  
363 routine bridge inspection. The model coefficients were determined based on the measurement  
364 data using a well-established methodology proposed by Hagan et al (1987) [31]. The linear crack  
365 width evolution trend results from the limited historical measurement data. With more crack data  
366 available over the time, the prediction of autoregression model will become more accurate. The  
367 model predicted that, without any crack repair, the crack width will reach its threshold (i.e.  $W_0 =$   
368 0.25mm, AS 5100.5 - Bridge Design, 2017) between 2021 and 2022. Figure 6 (b) shows the  
369 effects of different maintenance intervention cycle periods ( $T_{\text{cycle}}$ ) on the time-dependent  $\beta$ .  
370 Assuming  $\text{PoF} \leq 5\%$  of  $W_0$  is the objective of the road authority, it indicates that the service life of  
371 the bridge, in terms of allowable crack width ( $W_0$ ), is very sensitive to  $T_{\text{cycle}}$ . For example, the  
372 reduction of  $T_{\text{cycle}}$  by 50% (i.e. reduce  $T_{\text{cycle}}$  from 8 years to 4 years) could lead to three-fold  
373 increase of  $\beta$ , and therefore prolong the time for crack width to reach its allowable value. However,  
374 without a maintenance plan, the time for the crack width to reach  $W_0$  would be significantly  
375 shortened (i.e. less than 10 years).

376 Although the surface crack of the bridge can be fixed through regular maintenance, the chloride  
377 penetration and corrosion of the steel bars in concrete will continue over time with certain diffusion  
378 coefficient and corrosion rate which are dependent on different maintenance intervention cycle  
379 periods ( $T_{\text{cycle}}$ ). Figure 7a shows that chloride content at 50 mm depth in concrete increases with  
380 time because of the chloride penetration even the crack is sealed. Meanwhile, because the  
381 diffusion coefficient  $D$  is larger when concrete crack width  $W_t$  is greater than  $W_1$  (0.1mm), the

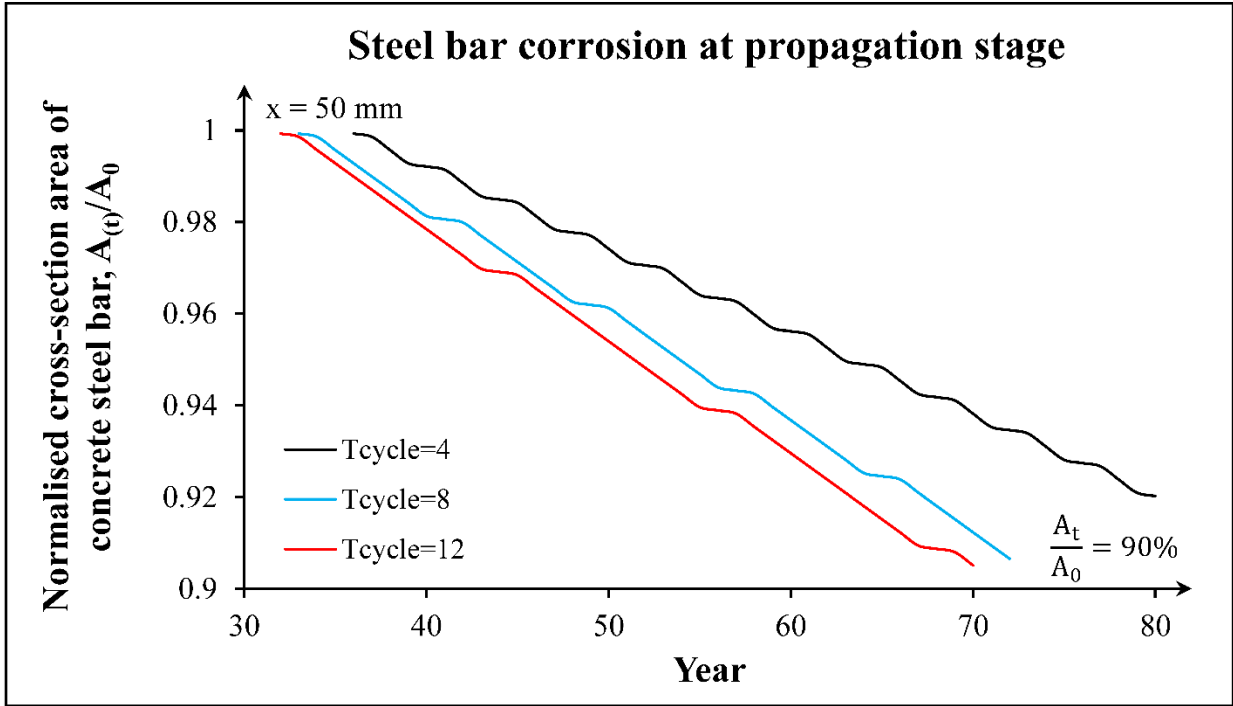
382 figure 7a shows different chloride profiles under different maintenance intervention cycles. In  
 383 figure 7b, assuming the allowable loss of the cross-section of a steel bar is 10% (i.e.  $A_i/A_0 \geq 90\%$ ),  
 384 the time-dependent normalised cross-section area of a steel bar ( $A_i/A_0 \geq 90\%$ ) under different  
 385  $T_{\text{cycle}}$  is shown, while the time-dependent value of  $\beta$  under different  $T_{\text{cycle}}$  can be found in Figure  
 386 7c. It demonstrates that reducing  $T_{\text{cycle}}$  from 12 years to 4 years could prolong the service life of  
 387 the bridge by around 15 years. In addition, assuming  $\text{PoF} \leq 5\%$  loss of cross-section area of a  
 388 steel bar is the objective of the road authority, the maintenance intervention cycle period of ( $T_{\text{cycle}}$ )  
 389 should be 4 years to prolong the service life to 70 years from a conservative perspective.



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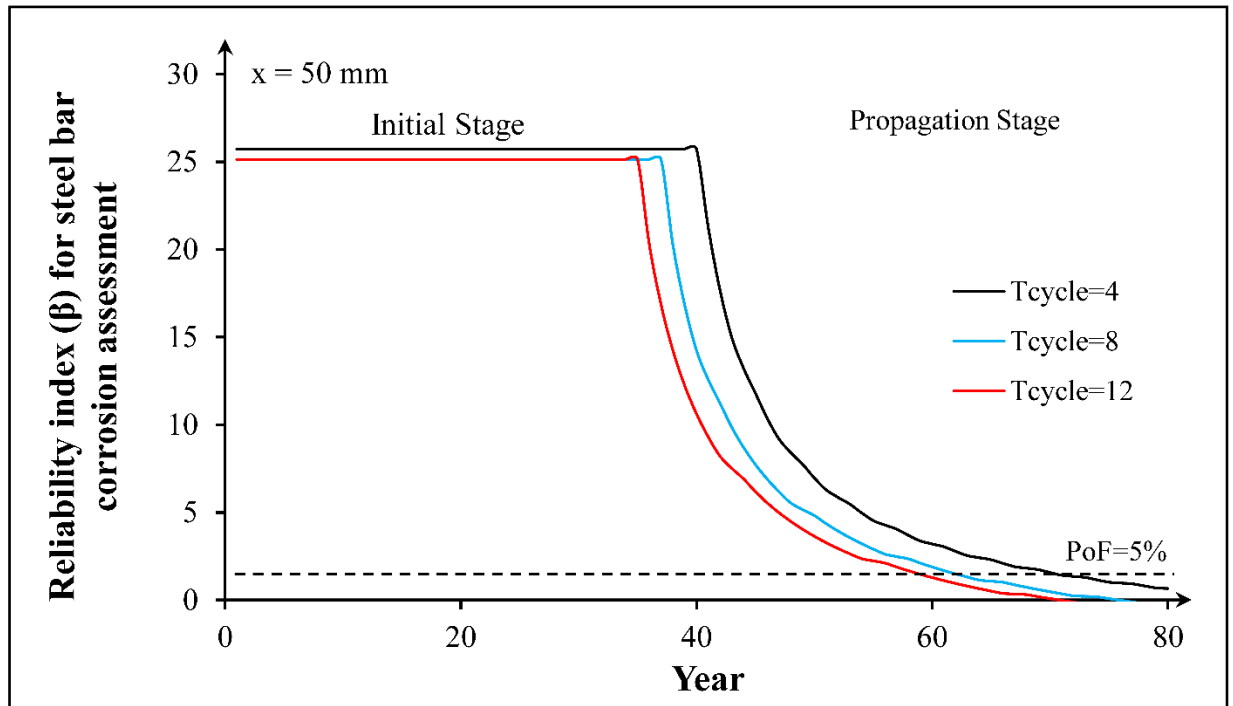
(a)



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(b)

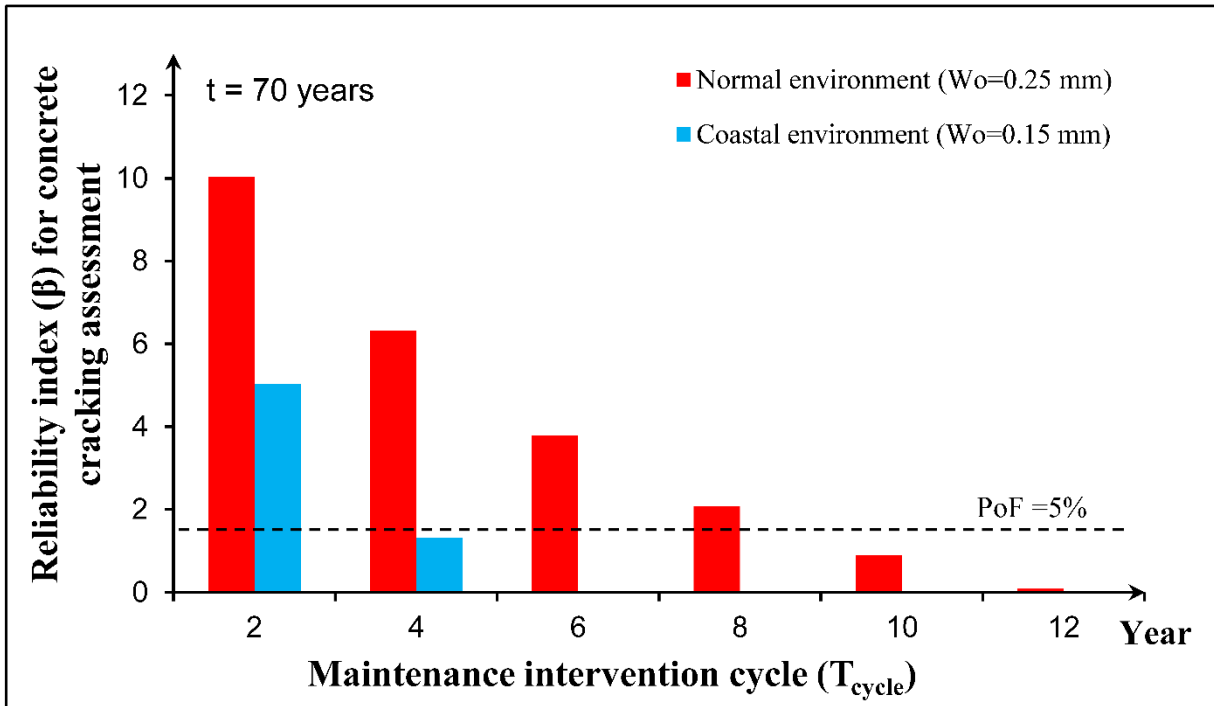


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(c)

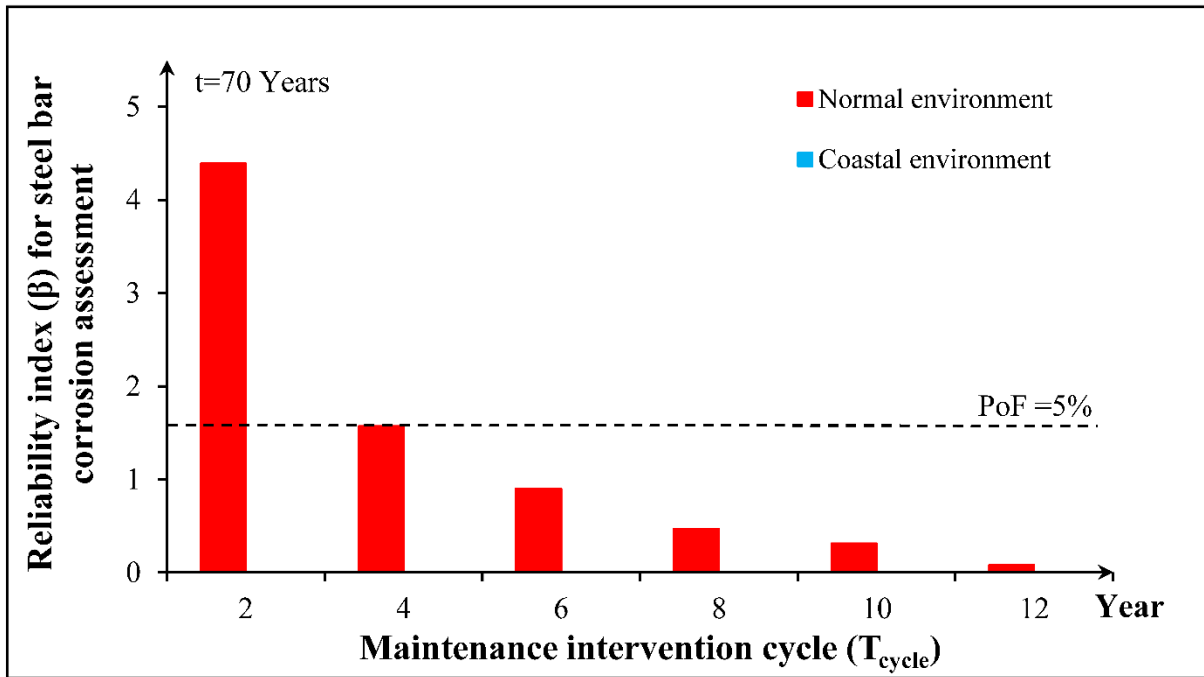
396 Figure 7 - (a) Chloride profile of total chloride content in concrete at depth 50 mm at initial stage;  
 397 (b) Time-dependent normalised cross-section area of concrete steel bar prediction using steel  
 398 corrosion model at propagation stage; (c) time-dependent reliability index ( $\beta$ ) for steel bar  
 399 corrosion assessment under different maintenance intervention cycle periods ( $T_{\text{cycle}}$ ). PoF,  
 400 probability of failure.



401

402

(a)



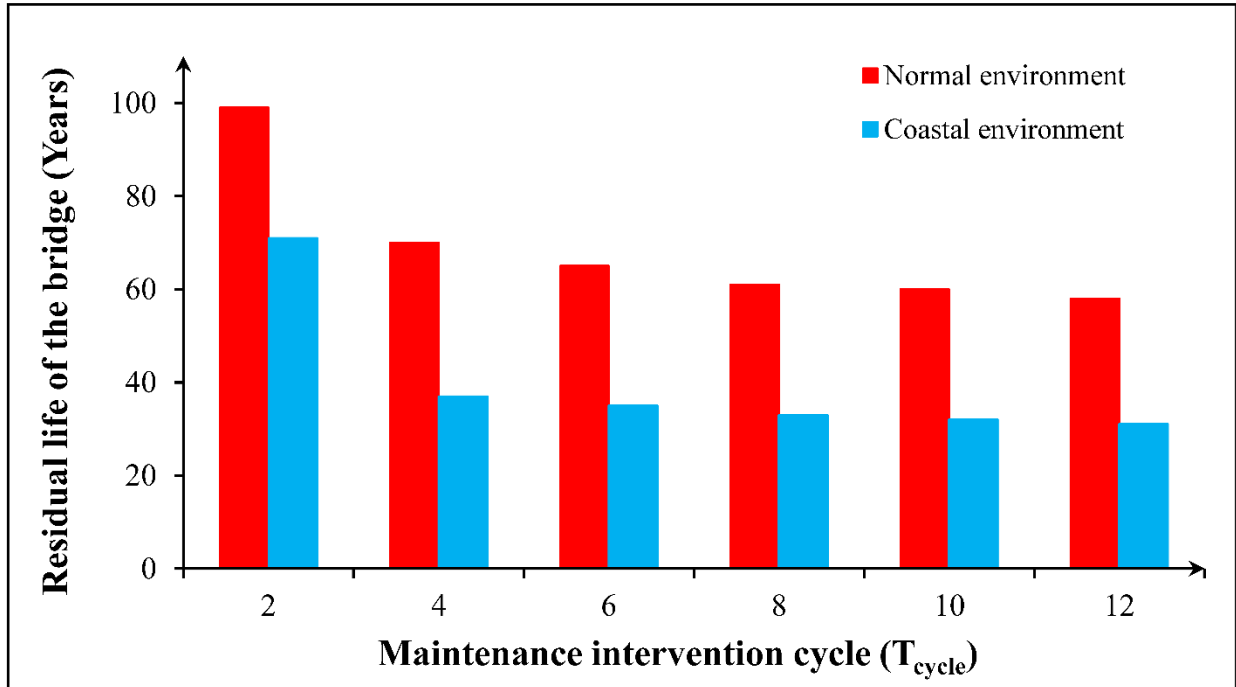
403

404

(b)

405 Figure 8 – (a) Reliability index ( $\beta$ ) for concrete cracking assessment at  $t = 70$  years under different  
 406 maintenance intervention cycle periods ( $T_{cycle}$ ) and different environmental conditions (normal  
 407 environment:  $W_0 = 0.25$  mm; coastal environment:  $W_0 = 0.15$  mm); (b) reliability index ( $\beta$ ) for steel  
 408 bar corrosion assessment at  $t = 70$  years under different maintenance intervention periods  $T_{cycle}$   
 409 and different environmental conditions (normal environment:  $W_0 = 0.25$  mm; coastal environment:  
 410  $W_0 = 0.15$  mm). PoF, probability of failure.

411 Figure 8 shows ERA assessment results of concrete cracking and steel bar corrosion of Long  
 412 Feng Xi Bridge at  $t = 70$  years under normal and coastal environmental conditions. It can be seen  
 413 from Figure 8a that for concrete crack assessment,  $T_{cycle}$  should be 8 years and 2 years for normal  
 414 and coastal environmental conditions, respectively. In addition, for steel bar corrosion assessment,  
 415  $T_{cycle}$  should be 2 years for normal environmental conditions from a conservative perspective.  
 416 However, in coastal environmental condition the corrosion causes serious rate of damage of  
 417 structure even the  $T_{cycle}$  is 2 years. Since the reliability index is below the target value.



418

419 Figure 9 – Estimated residual life of Long Feng Xi Bridge, Chongqing, China, with 5% probability  
 420 of failure under different maintenance intervention cycle periods ( $T_{\text{cycle}}$ ) and under two different  
 421 environmental conditions.

422 Based on the historical cracking data, the ERA-based framework can be implemented to predict  
 423 the residual life of a bridge. As shown in Figure 9, the residual life of a bridge is sensitive to the  
 424 external environmental conditions. Under along maintenance intervention cycle period (e.g.  $T_{\text{cycle}}$   
 425 = 6 years), the residual life of the bridge under normal environmental conditions is 2 times longer  
 426 than that under coastal environmental conditions. However, the difference of the residual life of  
 427 the bridge under normal and coastal environmental conditions can be significantly reduced with  
 428 a short intervention cycle period (e.g.  $T_{\text{cycle}}$  = 2 years).

429 **5. Conclusion**

430 The present study investigated the concrete crack induced corrosion of steel bars by developing  
431 ERA-based model using the eight-year inspection data from 3 different cracks of an operating  
432 bridge. The following are some major conclusions:

- 433 • The residual service life of a RC bridge based on the allowable crack width ( $W_0$ ) is dependent  
434 on the maintenance intervention cycle period ( $T_{\text{cycle}}$ ). To make sure the PoF of surface crack  
435 width is less than 5%, the  $T_{\text{cycle}}$  should be reduced by 65% (i.e. change  $T_{\text{cycle}}$  from 12 years  
436 to 4 years) which could lead to three-fold increase of  $\beta$ , and therefore prolong the service life  
437 of a bridge.
- 438 • Even if the surface crack of a RC bridge is repairable through periodic maintenance, the  
439 corrosion of the steel bars in the bridge could continue over time with a corrosion rate which  
440 depends on different  $T_{\text{cycle}}$ . It shows that reducing  $T_{\text{cycle}}$  from 12 years to 4 years could prolong  
441 the service life of the bridge by around 15 years.
- 442 • To make sure the probability of failure of the cross-section area of a steel bar is less than 5%,  
443  $T_{\text{cycle}}$  should be less than 4 years to prolong the service life to 70 years.
- 444 •  $T_{\text{cycle}}$  of a RC bridge is very sensitive to its external environment. To fulfil the allowable surface  
445 crack requirement,  $T_{\text{cycle}}$  should be 8 years and 2 years for bridges under normal and coastal  
446 environmental conditions, respectively. To fulfil the allowable steel bar corrosion requirement,  
447  $T_{\text{cycle}}$  should be at least 2 years for bridges under both normal and coastal environmental  
448 conditions.
- 449 • Under coastal environmental conditions, the maintenance intervention cycle period needs to  
450 be significantly shortened to prolong the service life to 70 years (e.g.  $T_{\text{cycle}} = 2$  years).

451 It should be mentioned that the reliability-based model developed in this study is completely  
452 general and are not in any way particular to a specific type of bridges. Sustainability assessment  
453 of concrete bridges under extreme environmental conditions is of critical importance all around  
454 the world [59]. However, much work is still to be done to accurately establish the correlation

455 between the reinforcement corrosion and crack development in concrete. Using a RC bridge in  
456 Chongqing, China as a case study, the present study could contribute to the accurate prediction  
457 of reinforcement corrosion induced by concrete cracking.

#### 458 **Limitations**

459 It should be mentioned that the relationship between the diffusion coefficient, corrosion rate of  
460 reinforcement and crack width was established based on previous literatures which ignores the  
461 variation of real environment condition. In addition, the traffic load induced concrete cracking was  
462 not considered in this study. Our future research will address these limitations and focus on model  
463 validation by collecting more bridge inspection data.

464

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468

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