13th International Conference on Applications of Statistics and Probability in Civil Engineering, ICASP13
Seoul, South Korea, May 26-30, 2019

2-Dimensional Simulation of Deterioration Process for Life-cycle Performance Assessment of RC Structures in Marine Environment

Xinyi Ye
Graduate Student, Dept. of Civil Engineering, Tsinghua University, Beijing, China

Quanwang Li
Associate Professor, Dept. of Civil Engineering, Tsinghua University, Beijing, China

ABSTRACT: The reliability-based durability design approach doesn't account for neither the surface deterioration of structures over service lives, nor the possible life-cycle maintenance. The paper employs the 2-dimentional (2D) simulation technique based on random field theory and Monte Carlo simulation method, to analyze the life-cycle performance of reinforced concrete structures under chloride attack, which is illustrated through the surface deterioration modelling of immersed tube tunnel segment of Hong Kong-Zhuhai-Macao (HZM) sea-link project. Then, the paper compares the maintenance demands imposed to different durability design specifications with different life-cycle performance target. The results may provide useful information in future durability design and aid the decision making process.

KEYWORDS: Concrete structure, Durability design, Spatial variability, Structural reliability, Life-cycle performance

Chloride-induced corrosion cracking is one of the main deterioration patterns of reinforced concrete (RC) structures in marine environment, leading to the reduction of service life and the increase of economic burden on maintenance. Thus, the owners put great concern on the durability performance of structures with long service lives. However, existing durability design approaches fail in providing such information. For one thing, the existing prescriptive approaches in most codes (ACI, 2004; CEN, 2004; GB/T 50476, 2008) are lacking of quantified descriptions on deterioration states and incompetent for structures with long expected service life or under brutal environment. For another, although variability of structure dimensions and material properties are taken into account, existing quantitative approach based on reliability theory (DuraCrete, 2000; fib, 2006 & 2010) takes the onset of steel corrosion as the limit state and concerns the deterioration of concrete at a point rather than a surface; As consequences, it is unable to describe the surface's spatial "post-corrosion" deterioration behaviors, and the structure's life-cycle deterioration behaviors are not considered in choosing durability target of long-term structures. Therefore, it's necessary to link the life-cycle durability performance with the reliability level of corrosion initiation which dominates the durability design, to answer owners' concern on life-cycle deterioration and maintenance demands of structures.

The paper firstly presents the models that describe the deterioration process of RC structures under chloride attack, and then the simulation technique based on 2D random field modelling. Taking the immersed tunnel segment in HZM project as an example, the surface deterioration process considering the effects of maintenance actions are predicted and presented. Finally, the life-cycle performance under different durability design levels and different intervention levels are compared.

1. DETERIORATION MODELS

Deterioration of RC structures in aggressive chloride environment was often modelled as a three-phase process (Li, 2003). The first phase is from the completion of construction to the corrosion onset of rebar, during which the diffusion of chloride ions is the focus of the deterioration behaviors, and can be described by the Fick's Second Law (Collepardi et al., 1972; Tuutti, 1982). The model suggests that the onset of corrosion is due to depassivation of rebar, which is characterized as the accumulation of the chloride ions until the content exceeds a threshold value (C_{cr}: % of binders). A time-dependent apparent diffusion coefficient is used in the model (Bamforth, 1995). The time to the onset of steel corrosion is given by:

$$t_{ini}=\left\{\frac{1}{D_{a0}\left(t_0\right)^n}\left[\frac{c}{\sqrt{2}\Phi^{-1}\left(1-\frac{C_{cr}}{C_s}\right)}\right]^2\right\}^{\frac{1}{1-n}} \tag{1}$$

in which C_s is surface chloride content (% of binders), D_{a0} is the apparent diffusion coefficient (mm^2/s) at age t_0 (s), n is the aging factor, c is the cover thickness (mm) and Φ^{-1} denotes the inversed cumulative probability function of standard normal distribution.

The second phase starts at corrosion initiation and ends at the appearance of surface cracking. Models for describing this phase had been developed in previous studies (Liu and Weyers, 1998; El Maaddawy and Soudki, 2011). The model by Vidal et al. (2004) is adopted in this research, which is validated based on natural corrosion experiment results. By using Faraday's Law of Electrolysis, the duration of the second phase (t_{ci}: years) is given by:

$$t_{ci}=\frac{A_s}{0.0366d\cdot i_{cor1}}\left[1-\left[1-\frac{\alpha}{d}\left(7.53+9.32\frac{c}{d}\right)10^{-3}\right]^2\right] \tag{2}$$

in which i_{cor1} is the corrosion rate (μA/cm^2) used at this phase, A_s is the cross-section of steel bar (mm^2); d is the rebar's diameter (mm); α is pit-penetration factor, and takes the value of 4 to 8 for pitting corrosion pattern.

After surface cracking, the corrosion product causes the extension and expansion of surface cracks. Models relating crack widths with corrosion amounts of rebar were developed based on acceleration corrosion tests (Rodriguez et al., 1996; Mullard and Stewart, 2011) and natural corrosion experiments (Vidal et al., 2004; Zhang et al., 2010). Transition from pitting pattern to general pattern was observed in their experiments, and models for both patterns were suggested (Zhang et al., 2010). The model for general pattern is further modified by Khan et al. (2014). Recognizing this transition, using Faraday's Law of Electrolysis, the crack width (w: mm) at different ages (t: years) can be calculated by (Li and Ye, 2018):

$$w=\begin{cases}\text{Pitting corrosion:} & 0.002\,d\cdot i_{cor1}\cdot\left(t-t_{ini}-t_{ci}\right) & t\le t_{0.3}\\ & 0.002\,d\cdot\left[i_{cor1}\cdot\left(t_{0.3}-t_{ini}-t_{ci}\right)+i_{cor2}\left(t-t_{0.3}\right)^{0.85}\right] & t>t_{0.3}\\ \text{General corrosion:} & 0.007\frac{d^2}{c}i_{cor1}\left(t-t_{ini}-t_{ci}\right)+0.164 & t\le t_{0.3}\\ & 0.007\frac{d^2}{c}\left[i_{cor1}\left(t_{0.3}-t_{ini}-t_{ci}\right)+i_{cor2}\left(t-t_{0.3}\right)^{0.85}\right]+0.164 & t>t_{0.3}\end{cases} \tag{3}$$

where $t_{0.3}$ is the time to crack width reaching 0.3mm (years); i_{cor1} and i_{cor2} are both corrosion rates (μA/cm^2), which will be demonstrated later.

More details about Eq.(3) can be found elsewhere (Li and Ye, 2018), and two notes are important.

- The transition time from pitting corrosion to general corrosion remains unclear. Based on the mechanism of pitting corrosion (Wilkins and Sharp, 1990), continuous length of corroded rebar, in this paper, is used as the indicator determining the transition from pitting corrosion to general corrosion. For example, pitting corrosion is assumed to be dominant when the length is less than 40 mm, and general corrosion is dominant when the length is over 200 mm. Interpolation is used to determine the crack width when the length is between 40 mm and 200 mm.
- The surface crack width will significantly accelerate the rate of corrosion (Bentur et al., 1997), and 0.3-mm is adopted as the critical crack width in the research. The corrosion rate

assumed to be lognormally distributed with mean of 0.67 μA/cm^2 before cracking of 0.3mm and 7.54 μA/cm^2 after that and a COV of 0.58 based on existing studies (Nakagawa et al., 2004; Li and Pang, 2015).

2. SIMULATION TECHNIQUE

Due to the spatial variability of exposed condition, concrete properties and cover thickness, the surface damage observed on in-service structures shows significant spatial variability, which suggests that spatial variability should be considered when conducting analysis of surface deterioration. We employ the random field theory (Vanmarcke, 1983) to account for the spatial variability of concrete cover, diffusion coefficient, chloride content, and corrosion rate.

Monte Carlo simulation is used. The concrete surface is divided into k×n identical elements of size Δ. The random variables, constant within each element, are identically distributed and correlated in different elements according to the correlation function. The Gaussian correlation function is adopted in this research:

$$\rho(\tau) = \exp\left(-\frac{\tau^2}{\xi^2}\right) \tag{4}$$

in which τ is the distance between the elements; and ξ is the correlation length of the random field.

Based on inspection data collected (Li and Ye, 2018), the correlation lengths of concrete cover and diffusion coefficient are 130 mm and 250 mm, respectively. The correlation lengths of surface content of chloride, critical content of chloride and corrosion rates are taken as 1960 mm, 2000 mm and 2000 mm respectively according to existing literatures (Karimi, 2002; Vu and Stewart, 2005; O'Connor and Kenshel, 2013).

3. RESULT AND DISCUSSION

In this section, we analyze the life-cycle performance of an immerged tube tunnel segment of HZM project whose service life is 120 years. A 22.5m×4m rectangular surface, seen in Figure 1, is taken as an example. 250 reinforcement bars with diameter of 28mm are vertically placed and equally spaced in the first layer. Table 1 summarizes the statistics of the durability parameters, according to the research conducted by Li et al. (2015).

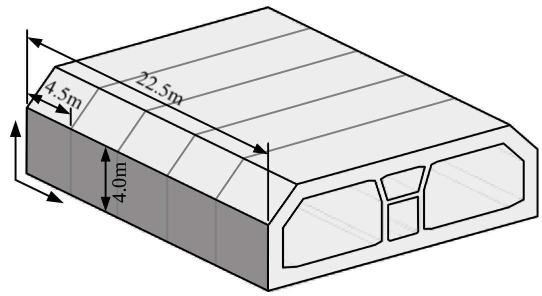

Figure 1: Standard segment of immersed tunnel tube in HZM project.

Table 1: Statistics for durability parameters.

Variable	Distribution
C_{cr} (% binder)	Beta (shape factors: 0.23, 033, bound:1.0%, 3.5%)
C_s (% binder)	Lognormal (mean=4.5%, deviation=0.68%)
c (mm)	Normal (mean=90, deviation=5.3)
D_{a0} ($10^{-12}m^2/s$)	Lognormal (mean=3.5, deviation=1.23)
n	Lognormal (mean=0.47, deviation=0.029)
i_{cor1} ($\mu A/cm^2$)	Lognormal (mean=0.67, deviation=0.39)
i_{cor2} ($\mu A/cm^2$)	Lognormal (mean=7.54, deviation=4.37)

3.1. Surface deterioration process without maintenance

The crack width is a most widely used measurement of surface deterioration for RC structures. Chinese code (JTJ 302-2006) grades the extent of deterioration into A, B, C and D by crack proportion and crack width, as seen in Table 2. In existing studies, the percentage of surface where crack width exceed a specific value is used to judge whether the damage state is reached or repair should be taken (Canisius and Waleed, 2004; Stewart and Mullard, 2007). In this study, we used the 1% crack proportion of specific crack width to classify the 4 grades of deterioration, also seen in Table 2.

Table 2: Durability damage grade for concrete structures.

Grade	*Item in JTJ-302*	*Our quantification*[1]
A	*Without any visible cracking*	$p_0 \le 1\%$
B	*A few corrosion-induced cracks with width less than 0.3mm*	$p_0 > 1\%$ *and* $p_{0.3} \le 1\%$
C	*Some continuous cracks along the reinforcement bar, with width between 0.3mm and 3.0mm*	$p_{0.3} > 1\%$ *and* $p_{3.0} \le 1\%$
D	*A large amount of continuous cracks along the reinforcement bar, some crack widths exceed 3.0mm*	$p_{3.0} > 1\%$

[1] p_0, $p_{0.3}$, and $p_{3.0}$ denote the proportion of cracking surface with the crack width exceeding 0mm, 0.3mm and 3.0mm, respectively.

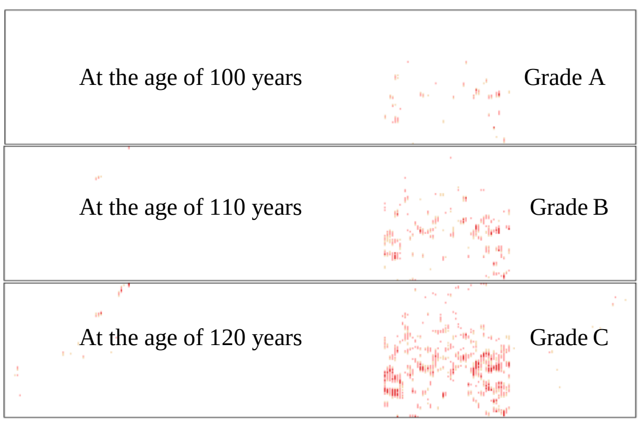

Figure 2: Crack distribution of a typical surface damage simulation.

Figure 2 shows the crack distribution of a typical simulation of surface damage. Damage grades at the age of 100, 110 and 120 years are given in the figure.

Figure 3 plots the time-dependent probabilities of deteriorating to the four grades. It can be seen from the figure that the portions of grade B and C are relatively small compared with the other portions, which suggests that the development of cracking is quick. Although the probability of no cracking is as high as 60% at the end of 120 years, the probability of severe damage (grade D) reaches 20%, which should be treated seriously.

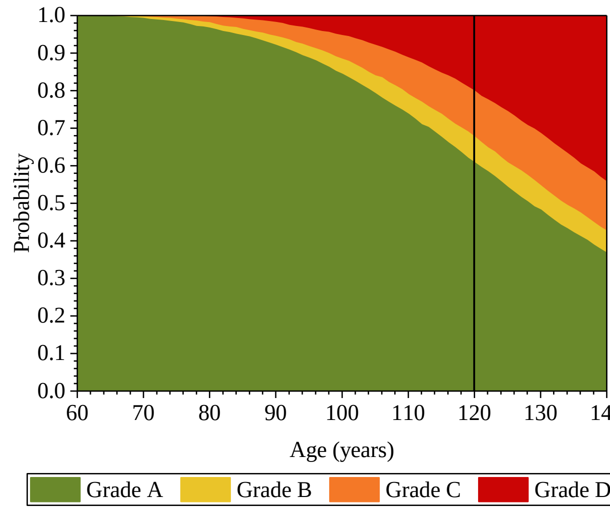

Figure 3: Probabilities of deteriorating to the four grades at different ages.

3.2. Possible maintenance demand

Maintenance serves as an essential part of life-cycle management, and is crucial to guarantee the structures' expected service life. The possible maintenance demand is a major concern of owners. The maintenance plan depends on not only the durability design level, but also the intervention level. In this study, three intervention levels for maintenance (preventive, necessary and mandatory) are considered. "Preventive" corresponds to the intervention threshold of grade B, while "necessary" and "mandatory" correspond to grades C and D, respectively (Li and Ye, 2018).

We assume that once the maintenance action is conducted on a member, all areas on surface with visible crack are repaired; and the repaired section will return to the initial newly-built state, which means the chloride content at steel surface drops back to zero while the C_s, D_{a0}, i_{cor1} and i_{cor2} remains unchanged after the repairs.

Figure 4 shows the distributions and mean values of time to first maintenance at three intervention levels, according to the 1000 times'

Monte Carlo simulation. The histogram shows the frequency of the time to first maintenance at preventive level, and the solid curve is the fitting curve of Lognormal distribution. The fitting curves for the other two levels are also shown in the figure, with mean values marked. It is found that, averagely there is no maintenance demand over the expected service life of 120 years, no matter which intervention level is selected; However, the probability of conducting a maintenance as early as 80 years is pretty high because of the high variability.

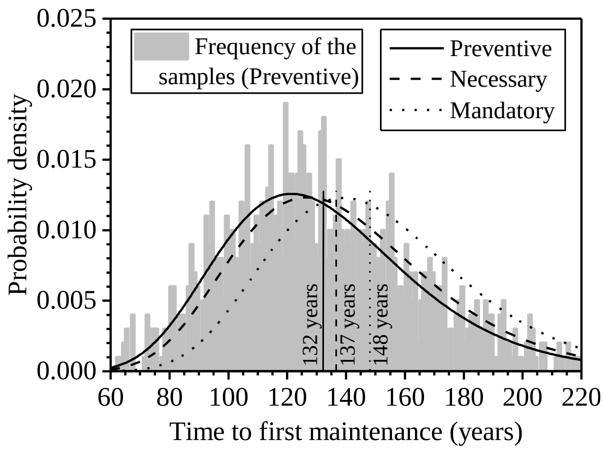

Figure 4: Distribution of time to first maintenance at three intervention levels.

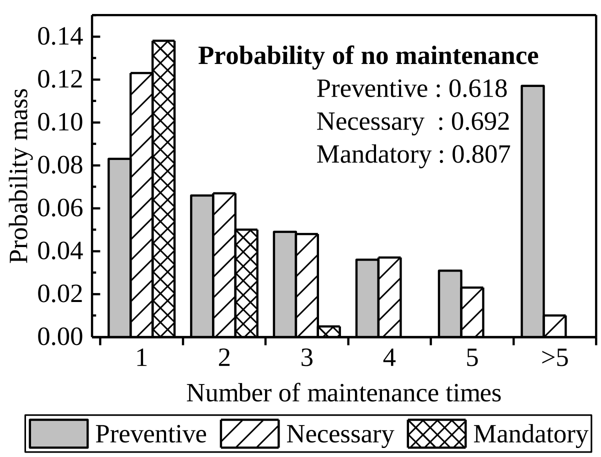

Figure 5: Distribution of maintenance times at three intervention levels over service life.

Figure 5 shows the probability mass of maintenance times at three intervention levels over the expected service life of 120 years. It's shown in the figure that the probabilities of at least 2 maintenances are significant for the three maintenance levels (e.g., 0.3 for preventive maintenance). Compared to mandatory intervention level, the preventive intervention level corresponds to shorter inspection intervals and more frequent maintenance actions. The figure also suggests that, although the probability of no maintenance is over 60%, the probability of more than 5 maintenances is about 0.12 at preventive level due to the high variabilities.

Figure 6 shows the mean values of crack proportion p_0 and $p_{0.3}$ when mandatory maintenance is conducted each time. It can be seen that the means of p_0 and $p_{0.3}$ increase at later maintenance. Although the threshold for mandatory maintenance is the same (when deteriorating to grade D), the area of surface needs to be repaired may increase in later maintenance, and the maintenance cost increases as well.

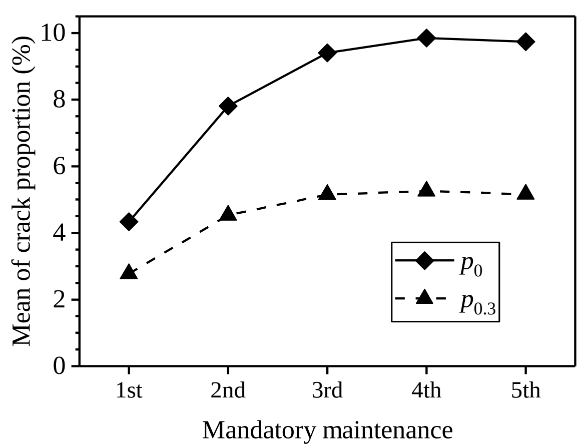

Figure 6: Crack proportion conducting mandatory maintenance.

3.3. Life-cycle performance under different durability design levels

Target reliability index (β_T) is used to represent the level of durability design. The value of 1.3 at 120 years is taken for the design of immersed tunnel. In this section, by varying durability design level (β_T), its effects on deterioration development and maintenance plan are discussed. Five durability design levels are considered, with β_T being 0.8, 1.1, 1.3, 1.6 and 1.8. The diffusion coefficient is fixed at the design value shown in

Table 1. The cover thickness varies to achieve different β_Ts, as listed in Table 3.

Table 3: Design values (mean) of cover thickness for five target reliability indices.

Specifications (β_T)	1 (0.8)	2 (1.1)	3 (1.3)	4 (1.6)	5 (1.8)
Design value of c (mm)	80	85	90	95	100

Table 4 shows the probabilities of the segment deteriorating to the four grade at the end of the service life without maintenance. The probability of deteriorating to grade C or D at the age of 120 years is more than 50% when β_T = 0.8 or 1.1 is selected, which implies that, to maintain the functionality and serviceability of the structure, intervention action is very likely to occur during the service life; While when β_T=1.8 is taken, the probability of remaining grade A is more than 85%, which suggests intervention action is less likely to occur during the service life.

Table 4: Probabilities of damage grades at the age of 120 years under five durability design level, without maintenance.

Grade	A	B	C	D
β_T=0.8	0.240	0.057	0.146	0.557
β_T=1.1	0.420	0.067	0.155	0.358
β_T=1.3	0.608	0.070	0.116	0.206
β_T=1.6	0.759	0.041	0.073	0.127
β_T=1.8	0.852	0.023	0.055	0.070

Table 5 shows the mean values of time to first maintenance during the 120 years' service life, under five specifications and three intervention levels. Compared with the specification with β_T=0.8, The specification with β_T=1.8 will delay the time to first intervention by 46 years at preventive level, 42 years at necessary level and 39 years at mandatory level. According to Table 5, if the owner's durability objective is "exemption from necessary maintenance during the service life averagely", β_T = 1.1 should be selected.

Table 5: The time to first intervention under five specifications.

Durability design	Preventive	Necessary	Mandatory
β_T = 0.8	103 years	110 years	120 years
β_T = 1.1	117 years	123 years	132 years
β_T = 1.3	129 years	135 years	143 years
β_T = 1.6	140 years	144 years	152 years
β_T = 1.8	149 years	152 years	159 years

Table 6: Maintenance intervals and the numbers of maintenance under five specifications over service life.

Durability design	Preventive	Necessary	Mandatory
β_T = 0.8	5.1 years 5.46 times	9.6 years 2.25 times	16.9 years 0.86 times
β_T = 1.1	5.4 years 3.20 times	9.8 years 1.36 times	17.4 years 0.50 times
β_T = 1.3	5.7 years 1.85 times	10.0 years 0.77 times	18.0 years 0.27 times
β_T = 1.6	5.7 years 0.95 times	10.0 years 0.41 times	18.4 years 0.14 times
β_T = 1.8	5.8 years 0.50 times	10.0 years 0.20 times	18.8 years 0.07 times

Table 6 shows the average number of maintenance and the mean intervention intervals during the service life of 120 years. Averagely, compared with the specification with β_T=0.8, the specification with β_T=1.8 will lessen the intervention number by 5 at preventive level, 2 at necessary level and 0.8 at mandatory level. As the intervention level changes from preventive to mandatory, the mean maintenance interval increases from around 5.5 years to around 18 years. But changing the durability design level makes nearly no difference on the intervals. According to the table, if the owner durability objective is "at most 1 averagely for preventive maintenance", β_T = 1.6 should be selected.

4. CONCLUSION

2D random field modelling was employed to examine the life-cycle performance of immerged tunnel of HZM project. The long-term deterioration process was simulated. Possible maintenance demand was estimated and discussed.

Comparisons were made among different specifications for reliability-based durability design. Conclusions are listed in the following.

- The 2D random field simulation, combined with calculation models for deterioration behaviors, is able to analyze the long-term durability performance.
- Analysis on surface deterioration of the segment in HZM project suggests that, maintenance is required to achieve the expected service life, and the probability of deteriorating to grade D is over 20% at the end of service life without maintenance.
- Different intervention levels lead to significant differences in maintenance frequency. The cost of later maintenance usually increases compared with the earlier maintenance although the same intervention level is adopted.
- Long-term deterioration process and possible maintenance demand are estimated under different durability design levels. Significant differences on time to first maintenance and maintenance frequency are observed, while the differences on maintenance interval are slight.

5. REFERENCE

American Concrete Institute (ACI). (2004). “Building code requirements for structural concrete & commentary.” *ACI 318-05* & *ACI 318R-05*, Farmington Hills, Michigan.

Bamforth, P. B. (1999). “The derivation of input data for modelling chloride ingress from eight-year UK coastal exposure trials.” *Magazine of Concrete Research*, 51(2), 87-96.

Bentur, A., Diamond, S., and Berke, N. (1997). *Steel corrosion in concrete: Fundamentals and civil engineering practice*, E & FN Spon, London.

Canisius, T. D. G., and Waleed, N. (2004). “Concrete patch repairs under propped and unpropped implementation.” *Proceedings of the Institution of Civil Engineers-Structures and Buildings*, 157(2), 149-156.

Collepardi, M., Marcialis, A., and Turriziani, R. (1972). “Penetration of chloride ions into cement pastes and concretes.” *Journal of the American Ceramic Society*, 55(10), 534-535.

DuraCrete. (2000). “Probabilistic performance based durability of concrete structures: General guidelines for durability design and redesign.” *Vol. 15. Report No. BE95-1347*, The European Union - Brite EuRam III, Lyngby.

El Maaddawy, T., and Soudki, K. (2007). “A model for prediction of time from corrosion initiation to corrosion cracking.” *Cement and Concrete Composites*, 29(3), 168-175.

European Committee for Standardization (CEN). (2004). “EuroCode2: Design of Concrete Structures”, *EN 1992-1-1*, Brussels.

Fédération Internationale du Béton (fib). (2006). “Model code for service life design.” *Bulletin 34*, Lausanne.

Fédération Internationale du Béton (fib). (2010). “Model Code 2010–First complete draft.” *Bulletin 55/56*, Lausanne.

GB/T 50476. (2008). “Code for durability design of concrete structures.” *GB/T50476-2008*, Ministry of Housing and Urban-Rural Development of the People’s Republic of China, Beijing, China (in Chinese).

JTJ 302. (2007). “Technical specification for detection and assessment of harbour and marine structures.” *JTJ 302-2006*, Ministry of Transport of the People’s Republic of China, Beijing, China (in Chinese).

Karimi, A. R. (2002). “Probabilistic assessment of deterioration and strength of concrete bridge beams and slabs.” Doctoral dissertation, University of London, London.

Khan, I., François, R., and Castel, A. (2014). “Prediction of reinforcement corrosion using corrosion induced cracks width in corroded reinforced concrete beams.” *Cement and Concrete Research*, 56, 84-96.

Li, C. Q. (2003). “Life-cycle modeling of corrosion-affected concrete structures: Propagation.” *Journal of Structural Engineering*, 129(6), 753-761.

Li, Q., Li, K., Zhou, X., Zhang, Q., and Fan, Z. (2015). “Model-based durability design of concrete structures in Hong Kong–Zhuhai–Macau sea link project.” *Structural Safety*, 53, 1-12.

Li, Q., and Pang, L. (2015). “Survey on durability performance of high-pile wharf structures in South China.” *Report No.2013-328-49A-090*. (in Chinese)

Li, Q., and Ye, X. (2018). “Surface deterioration analysis for probabilistic durability design of RC structures in marine environment.” *Structural Safety*, 75, 13-23.

Liu, Y. P., and Weyers, R. E. (1998). “Modeling the time-to-corrosion cracking in chloride contaminated reinforced concrete structures.” *ACI Materials Journal*, 95(6), 675-681.

Mullard, J. A., and Stewart, M. G. (2011). “Corrosion-induced cover cracking: New test data and predictive models.” *ACI Structural Journal*, 108(1), 71-79.

Nakagawa, T., Seshimo, Y., Onitsuka, S., and Tsutsumi, T. (2004). “Assessment of corrosion speed of RC structure under the chloride deterioration environment.” *Proceedings of JCI symposium on the analysis model supporting the verification of longterm performance of concrete structure in design.* Japan Concrete Institute, 325-330. (in Japanese)

O’Connor,A.J., and Kenshel,O. (2013). “Experimental Evaluation of the Scale of Fluctuation for Spatial Variability Modeling of Chloride-Induced Reinforced Concrete Corrosion.” *Journal of Bridge Engineering*, 18(1), 3-14.

Rodriguez. J, Ortega. L.M., Casal. J,and Diez. J.M. (1996). “Corrosion of reinforcement and service life of concrete structures.” *Durab. Build. Mater.Compon.*, 7(1),117-126.

Stewart, M. G., and Mullard, J. A. (2007). “Spatial time-dependent reliability analysis of corrosion damage and the timing of first repair for RC structures.” *Engineering Structures*, 29(7), 1457-1464.

Tuutii, K. (1982). “Corrosion of Steel in Concrete.” *No. 4*, Swedish Cement and Concrete Research Institute, Stockholm.

Vanmarcke, E. (1983). *Random Fields: Analysis and Synthesis*, MIT Press.

Vidal, T., Castel, A., and François, R. (2004). “Analyzing crack width to predict corrosion in reinforced concrete.” *Cement and Concrete Research*, 34(1), 165-174.

Vu, K. A. T., and Stewart, M. G. (2005). “Predicting the likelihood and extent of reinforced concrete corrosion-induced cracking.” *Journal of Structural Engineering*, 131(11), 1681-1689.

Wilkins, N. J. M., and Sharp, J. V. (1990). “Localised corrosion of reinforcement in marine concrete.” *Proceeding of 3rd International Symposium on Corrosion of Reinforcement Concrete Construction*, Wishaw, Warwickshire, UK, 3-18.

Zhang, R., Castel, A., and François, R. (2010). “Concrete cover cracking with reinforcement corrosion of RC beam during chloride-induced corrosion process.” *Cement and Concrete Research*, 40(3), 415-425.