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Surface deterioration analysis for probabilistic durability design of RC structures in marine environment

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ABSTRACT

In probabilistic durability design of concrete structures in marine environment, the limit state is defined as the corrosion initiation of reinforced steel. However, the owner may more concern the cracking growth of concrete surface or the first repair time without maintenance, in that case, additional information on surface deterioration should be supplemented to eliminate owner's concern. This paper employs the two-dimensional simulation method to predict the surface deterioration process for reinforced concrete structures exposed to aggressive chloride environment. The spatial variability of concrete cover (c), chloride diffusion coefficient (D_{a0}) and other durability parameters are considered in the simulation, and the effect of batch-production of concrete is accounted for in the spatial variation modelling of c and D_{a0}. Taking the Hong Kong-Zhuhai-Macau sea-link project as an example, the paper evaluates the durability performance of the immerged tube tunnel over the expected service life, and compares a range of durability design specifications with different target reliabilities and combinations of c and D_{a0}, in terms of the probability & extent of surface damage. With three maintenance levels prescribed by surface damage grade at intervention, the paper compares the intervention time of different durability design specifications. The results provide a comparative tool to aid designers in selecting design specifications to achieve owner-concerned durability objective.

1. Introduction

Corrosion-induced deterioration is the main cause of damage in reinforced concrete (RC) structures, especially in marine environment, leading to the failure of serviceability and the reduction of structural safety. Thus, durability design has been recognized as an essential part of structural design to ensure the expected working life of concrete structures. Most codes on durability design, up to now, are prescriptive [7,1,14], which stipulate the requirements for material composition and structural details for given environmental conditions. They are good for benign environments and short to moderate service lives, but not adequate for extreme environments or long design lives. The model-based durability design provides another approach to achieve the target service lives [9,12–13]. They employ mathematical models to determine the material properties and structural dimensions for expected design lives. Especially, combined with reliability analysis, it can account for the uncertainties associates with the properties of concrete, structural dimensions and environmental actions, as well as the uncertainty of the empirical model, and provides greater level of reliability for service life achievement and reduces the risk of major maintenance. Reliability-based durability design has been implemented in some major infrastructure projects [28,20,21].

However, durability design approach at current state is far from perfect. The limit state for quantitative durability design of concrete structures is the onset of steel corrosion, which depends on the exposure condition, the cover thickness, and the property of concrete and steel, but not on the dimensions of members, so the durability design is at "material level" concerning the deterioration of concrete at a point, rather than a surface. On the contrary, the durability damage of existing structures at service, which is usually more concerned by the owner, is observed and evaluated through the "member level" inspections, such as rust stains, crack width/length or spalling area of concrete, etc., based on which, the decision on maintenance actions is made. The durability design, targeting at the same reliability level for corrosion initiation by different combinations of concrete cover and diffusivity, may result in different durability performance of structures over the whole service life, because the surface damage of RC member depends on many other factors, e.g., steel diameter, corrosion pattern, corrosion rate etc., which are not considered in durability design taking corrosion initiation as limit state. Therefore, it is meaningful to establish a method for linking the reliability level of corrosion initiation in probabilistic durability design to the probability and extent of surface

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damage of RC structure over the service life, in response to the owner's concern.

Corrosion damage is spatially varied due to the spatial variability of dimensions, material properties, exposure conditions, workmanship and other factors. Some recent work has been conducted on the spatial modeling of deterioration process of reinforced concrete members due to chloride-induced corrosion [23,36,38], and the effects of spatial variability of ductility parameters were investigated on the corrosion initiation, crack propagation and strength reduction. For example, Vu and Stewart [44] developed a two-dimensional spatial time-dependent reliability model to predict the extent and probability of corrosion-induced cracking. Val and Stewart [40] established a reliability assessment framework for aging reinforced concrete structures and applied it to analyzing the process of two-dimensional chloride-induced corrosion cracking; Stewart [37] conducted the assessment of corrosion damage, safety and structural repair decision of RC structure using the spatial time dependent modeling of deterioration process. Although spatial reliability analysis of corroding RC structures was well studied in past decade, it has never been applied in durability design of RC structures and its effect on durability design remains unaddressed.

This paper first presents models for corrosion initiation, concrete cracking and crack propagation, to track the evolution of concrete deterioration at a point. Then the models are combined with the two-dimensional (2D) random field modelling technique to simulate the cracking process of concrete surface. Employing the method, the durability design of the immerged tube tunnel of Hong Kong – Zhuhai – Macau (HZM) sea-link project is evaluated for surface damage over the expected service life, and a range of durability specifications with different target reliability indices and combinations of concrete cover – chloride diffusivity are compared in terms of the probability and extent of concrete surface cracking. Finally, three maintenance levels are defined by surface damage grade at intervention, and accordingly, the intervention times are estimated and compared among different durability design specifications.

2. Models of concrete deterioration process

Chloride-induced concrete deterioration is commonly modelled as a three-phase process including chloride-ingress, crack initiation and cracking propagation [19], as showed in Fig. 1. The first phase is the time period from the completion of the structure construction to the corrosion initiation in the structure, and was the research focus in the area of reinforcement corrosion in the past several decades [4,3,18]. The second phase is from the initiation of corrosion to the onset of concrete cracking, and the third phase is characterized with the crack propagation. The duration of the second and third phases depends

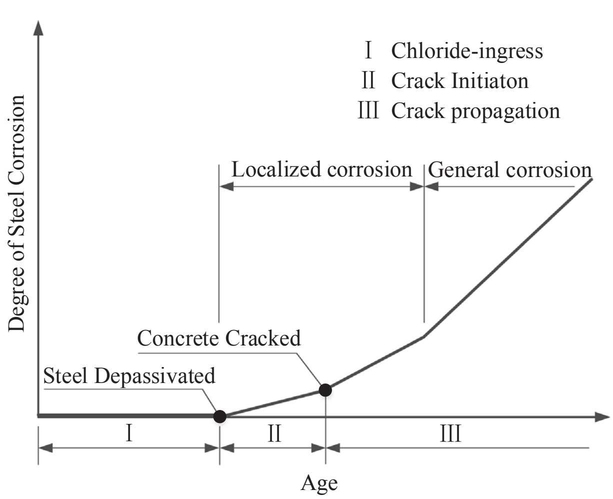

Fig. 1. Schematic three-phase deterioration process of RC structures.

principally on the amount of steel corrosion [25,10,29]. Experimental results show that the localized (pitting) corrosion pattern dominates the crack occurring and the early stage of crack propagation, while the general corrosion pattern becomes dominant in the later stage of cracking propagation [48]. The transition from localized corrosion to general corrosion is due to the interconnection and widening of longitudinal corrosion cracks as corrosion develops [47].

The models for the three-phase deterioration process are summarized in the following.

2.1. Chloride-ingress

In this phase, chloride ions from outer environment ingresses through the porous or micro-fracture within concrete cover, until the passivation layer of embedded steel is destroyed indicating the onset of steel corrosion. This phase can be described by the Fick's Second Law [8]. Under the assumption of time-invariant surface chloride concentration (C_s: mass percentage of binders) and diffusion coefficient (D: mm^2/s), the chloride concentration (C: mass percentage of binders) at certain depth (x: mm) and certain time (t: s) can be expressed as:

$$C(x,t) = C_s\left[1-erf\left(\frac{x}{2\sqrt{Dt}}\right)\right] \tag{1}$$

where *erf* is the mathematical error function. To deal with the fact that chloride diffusion coefficient is time-variant [27], an apparent diffusion coefficient is adopted in the model [3], and a power law is recommended for its ageing behavior [26].

$$D_a(t) = D_{a0}\left(\frac{t_0}{t}\right)^n \tag{2}$$

where n is the age factor and D_{a0} is the apparent diffusion coefficient at time t_0.

It is assumed that the embedded steel bar is depassivated and the corrosion begins when the chloride concentration at its surface reaches a threshold value (C_{cr}: mass percentage of binders), the time to corrosion initiation (t_{ini}: s) is then given by:

$$t_{ini} = \left\{\frac{1}{D_{a0}(t_0)^n}\left[\frac{c}{\sqrt{2}\,\Phi^{-1}\left(1-\frac{C_{cr}}{C_s}\right)}\right]^2\right\}^{\frac{1}{1-n}} \tag{3}$$

where c is the concrete cover thickness (mm), and ϕ^{-1} is the inversed cumulative function of standard normal distribution.

2.2. Crack initiation

When the reinforcing steel corrodes in concrete, its products fill the concrete pore firstly, and then exert pressure to the surrounding concrete, resulting in the cracking at the surface of concrete member. A number of prediction models have been developed, including the cracking time models [4,25,10] and the corrosion amount models for cracking [35,2]. They considered the mechanical behaviors of the surrounding concrete as the result of steel corrosion and expansion, and were calibrated with accelerated corrosion tests. However, the accelerated laboratory tests have been criticized for not being representable of the natural corrosion process and hence the model obtained from such tests may not be applicable to real structures [48]. Based on long-term in-situ corrosion experiments, in which full-size beams with compression strength (f'_c) of 45 MPa were put in an aggressive chloride environment for nearly 20 years, Vidal et al. [42] validated the model given by Alonso et al. [2], and suggested the steel cross-section loss (ΔA_{s0}: mm^2) for concrete cracking:

$$\Delta A_{s0} = A_s\left[1-\left[1-\frac{\alpha}{d}\left(7.53+9.32\frac{c}{d}\right)10^{-3}\right]^2\right] \tag{4}$$