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Service life prediction of RC structures in marine environment using long term chloride ingress data: Comparison between exposure trials and real structure surveys

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HIGHLIGHTS

- Real wharf structure surveys show time dependency of C_s and D at the splash zone.
- Height-dependency of C_s indicates the existence of a "semi-splash" zone.
- Adding BFS in concrete mix improves significantly its resistance to chloride ingress.
- Consistent decreasing trend of D for both OPC concrete and BFS concrete is observed.

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ABSTRACT

The service life of reinforced concrete (RC) structures in marine environments is mainly influenced by the chloride-induced corrosion of reinforcement, and the development of chloride penetration model is essential for its assessment. The empirical Fick's second law of diffusion provides a simple way to predict the chloride penetration in practical situations. However, the derivation of parameters of this model is mainly based on the results of laboratory experiments or field exposure trials, and these parameters need to be calibrated with long-term field results of real structures. The filed investigations of seventeen high-pile wharf structures located at the south coast of China were carried out, and the results were compared with the long-term exposure test results in terms of chloride ingress profile. The probability models for surface chloride content and chloride diffusion coefficient were derived according to these results; and finally the effects of different models of chloride ingress parameters, based on exposure trails or based on real structure surveys, on the expected service life of marine structures were investigated.

1. Introduction

The chloride-induced corrosion of steel reinforcement has been a critical issue for the durability of RC (Reinforced concrete) structures in marine environment, and considerable research efforts have been dedicated to the mechanisms and the modelling of the chloride ingress, as well as the subsequent steel corrosion processes [1–4]. In marine exposure environment, the chloride ions can penetrate into concrete through multiple mechanisms including diffusion, adsorption, permeation and surface deposit of airborne salts [5,6]. Among them the main transport mechanisms of chloride ions is diffusion [7], which, in engineering practice, can be modelled by the empirical Fick's diffusion model. The model uses the analytical solution of Fick's second law with constant boundary conditions to represent the chloride ingress [8]. This model presents a very simplified image of the real transport processes of chloride ingress, and envelops actually all processes into the well-known "apparent" chloride diffusion coefficient [9]; and moreover, long-term monitoring of apparent chloride diffusion coefficient in real structures and the standardization of its determination through chloride profiling method [10,11], make the empirical Fick's model apt for durability assessment and service life prediction of RC structures, as long as the uncertainties associated with the model can be quantified [12,13].

The influences on the durability of RC structures come from different factors, including: (i) material qualities, (ii) workmanship during construction and (iii) exposure conditions. Extensive studies have been dedicated to the effects of material types and qualities on the ingress process of chloride ions, however, the knowledge about the exposure conditions may be little, because the local marine environment may differ greatly at different loca-

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tions due to the change in climatic condition and the salinity [14]. In practice, to design or assess the durability of RC structures in a given location, the influence from the environmental actions has to be quantified with chloride ingress data measured in concrete exposed at the same location [15]. However, usually these kinds of data come from specimens exposed for relatively short period, thus the time-dependent behavior is difficult to model, e.g., the decreasing of chloride diffusion coefficient and the building up of surface chloride content over a long period. Moreover, in existing research, the derivation of chloride ingress parameters is mainly based on the results of laboratory experiments or field exposure tests [3,16–21], while the long-term filed results of real RC structures are rare [22,23]. These parameters need to be calibrated with long-term survey results of real marine structures. Finally, recognizing the great variability associated with the chloride diffusion data, the durability assessment or the service life prediction of RC structures should be performed based on probabilistic approach.

In this paper, as part of a durability survey of high-pile wharfs located on the south coast of China, the chloride ingress data were measured for 17 high-pile wharf structures constructed between 1970s to 2000s. Firstly these data are compared with the chloride ingress data from RC exposure trials located at the similar location in terms of surface chloride content and chloride diffusion coefficient; and then the data from real structures and exposure trails are combined to generate the probability models for chloride ingress parameters, as well as their time-dependent behaviors. The models are finally employed to predict the service life of high-pile wharf structures on the south cost of China, and a comparison in expected service life is made between different concrete mixes and between the real wharf structures and the exposure trails.

2. Structure and marine environment

2.1. Structure and material

Seventeen high-pile wharf structures located on the south coast of China were selected for field investigation. Their geographical locations are shown in the map in Fig. 1. These structures are representative of the wharf structures of that area, whose vertical view is shown typically in Fig. 2, and some characteristics of the structures are described in Table 1. The chloride ingress profiles of cross beams were measured and recorded in the field investigations. These beams are about 2 m above the sea level and subjected to the marine exposure due to wave splashing

Historical records were collected and studied prior to the field inspections, however, for most wharf structures constructed before 1990, the important design files such as the mix proportion of concrete and curing procedure were not documented, and the amount of information available regarding the concrete composition and the water-to-cement ratio (w/c) was quite low. Therefore, for the wharf structures whose design files were lost, the cement-type and water-to-cement ratio were inferred referring to the relevant Chinese codes guiding the design and construction of wharf structures at that time [24–27], as presented in Table 1. The typical mix proportions for different concrete strength are shown in Table 2.

The structures are divided into two groups, constructed before 2000 and after 2000, because the national codes for durability design of concrete in marine environment underwent significant modifications [25–27] in 1980s to 1990s. In the 1987 code (JTJ 228-1987) [26], the splash zone was firstly set, the w/c was limited to 0.45 and the allowable maximum concrete cover depth was set to 60 mm. In the code (JTJ 268-1996) [27], the w/c was further limited to 0.40, the allowable maximum concrete cover depth was increased to 65 mm; and moreover, specifications on the use of mineral admixtures in concrete mix, e.g. blast furnace slag and fly ash, were firstly added in 1996 code. As a result, the use of supplementary cementitious materials in concrete becomes popular in marine structures after 2000, while before 2000 these materials were very cautiously and hesitantly used.

2.2. Exposure environment

The investigated structures are located in the southern subtropical marine monsoon region of China. The annual average temperature is between 21.3 and 23.5 °C. The annual average humidity is

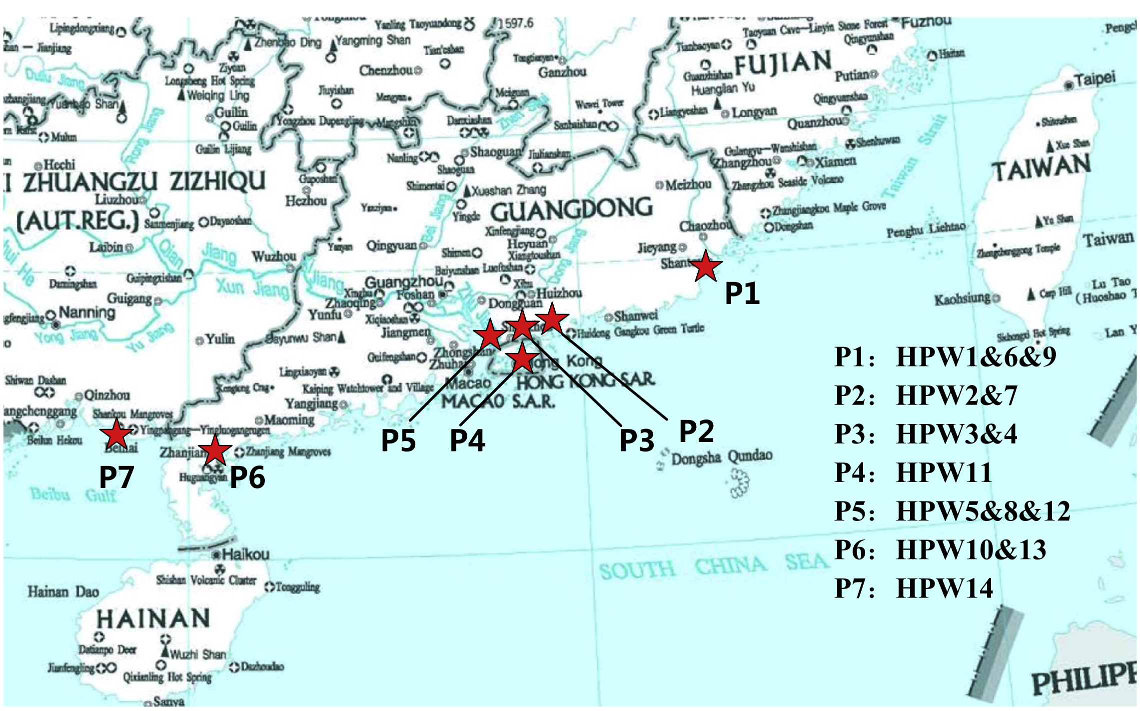


Fig. 1. Geographical locations of the investigated wharf structures.

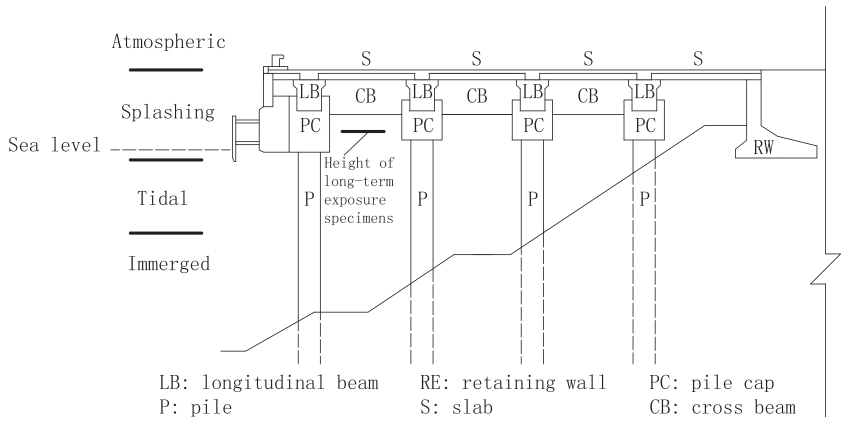


Fig. 2. Vertical view of typical wharf structures.

Table 1
Characteristics of the investigated structures.

Structure[1]	Construct year	Investigate year	Concrete strength[2]	W/C	Cement type[3]
HPW1	2008	2012	C40	0.4	MAC
HPW2-3#4#	2008	2012	C50	0.4	MAC
HPW2-5#6#	2008	2012	C50	0.4	MAC
HPW2-7#	2008	2012	C50	0.4	MAC
HPW3	2004	2012	C45	0.4	MAC
HPW4	2000	2012	C45	0.4	BFSC
HPW5-6#	1994	2008	C30	0.45	OPC
HPW6	1994	2012	C30	0.45	OPC
HPW7	1988	2005	250#	0.4	OPC
HPW5-5#	1988	2008	250#	0.45	OPC
HPW8	1987	2007	300#	0.4	OPC
HPW9	1986	2012	300#	0.45	OPC
HPW10	1984	2005	250#	0.45	OPC
HPW11	1984	2007	250#	0.65	OPC
HPW12	1984	2008	250#	0.5	OPC
HPW13	1979	2008	250#	0.65	OPC
HPW14	1971	2007	250#	0.5	OPC

[1] Abbreviation "HPW2-#3#4" means the berth 3 and berth 4 of High Pile Wharf 2.
[2] '300#' corresponds to a design strength of 28 MPa; '250#' corresponds to a design strength of 25 MPa.
[3] Abbreviation "MAC" means mineral admixtures, e.g. fly-ash, slag or silica fume, were added in cement, "BFSC" means blast furnace slag was added in cement; "OPC" means Ordinary Portland cement.

between 78 and 85% with large seasonal variation, and the seasonal humidity can reach 100% (spring and summer) and drop to 15% (winter). The hydrology data show that the chloride ions (Cl^-) content in sea water is in the range of 14.22–18.35 g/L (see Table 3 for details).

2.3. Exposure trials

Results from long-term exposure trials were also included in the study. In 1987, some reinforced concrete blocks were cast using Ordinary Portland cement (OPC), with a w/c of 0.40. In 2002, some reinforced concrete specimens of w/c = 0.35 were cast incorporating blast-furnace slag (BFS) as supplementary cementitious materials (with content of 60%). These specimens were all located in the splash zone at Zhenjiang Exposure station (location P6 in Fig. 1). Surveys of OPC concrete specimens were carried out at 3, 5, 10, 17 and 20 years after exposure, while surveys of BFS concrete specimens were carried out at 90 days, 180 days, 1, 2, 4 and 7 years after exposure. In situ measurements were taken and chloride ingresses were determined by analysis of 60 mm cores retrieved from the blocks and cut into slices.

3. Analysis of long term chloride ingress data

3.1. Chloride ingress

The ingress of chloride into concrete involves a number of mechanisms, but it is greatly accepted that diffusion is the principle mechanism. Therefore, the empirical Fick's model based on diffusion theory can be used to model chloride ingress, and the coefficients derived from Fick's model fitting are apparent values, which makes theoretical predictions match field observations. According to the Fick's second law of diffusion, the profile of chloride concentration $C(x, t)$ along the depth x and with time t, writes,

$$C(x,t) = C_0 + (C_{sa} - C_0)\left[1 - \mathrm{erf}\left(\frac{x}{2\sqrt{D_a t}}\right)\right] \tag{1}$$

in which C_0 is the initial chloride content; C_{sa} is the apparent chloride surface content; D_a is the apparent chloride diffusion coefficient in concrete and erf is the mathematical error function. The apparent diffusion coefficient is found to be time-dependent, and a power law is recommended for its ageing behavior [28],

$$D_a(t) = D_{a,0}\left(\frac{t_0}{t}\right)^a \tag{2}$$

where $D_{a,0}$ stands for the average diffusivity for chloride at concrete age t_0; a for the age exponent describing the decrease of the average

Table 2
Typical proportions for concrete strength.

Concrete strength	Binding material				Sand	Coarse aggregate	Water	Admixture
	Cement	Fly-ash	Silica fume	Blast furnace slag				
250#	1	0	0	0	2.56	4.36	0.65	0
300#	1	0	0	0	1.86	3.45	0.45	0
C25	1	0	0	0	1.68	2.75	0.45	0
C40	1	0	0	0	1.55	2.33	0.4	0.003
C45	0.72	0.25	0.03	0	1.74	2.50	0.36	0.017
C30	0.4	0.4	0	0.2	1.75	2.41	0.36	0.010
C50	0.45	0.1	0	0.45	1.54	2.21	0.32	0.002

Table 3
Exposure environments of wharf structures.

Structure	Annual average temperature (°C)	Annual average humidity (%)	Average [Cl^-] content (g/L)
HPW1	21.3	82	15.95
HPW2-3#4#	22.1	78	16.00
HPW2-5#6#	22.1	78	16.00
HPW2-7#	22.1	78	16.00
HPW3	22.6	80	15.95
HPW4	22.6	80	15.95
HPW5-6#	22.4	79	15.9
HPW6	21.3	82	15.95
HPW7	22.1	78	16.00
HPW5-5#	22.4	79	15.90
HPW8	22.4	79	14.22
HPW9	21.3	82	15.95
HPW10	23.5	85	15.05
HPW11	22.8	82	16.58
HPW12	22.4	79	15.90
HPW13	23.5	85	15.05
HPW14	22.6	81	18.35

Table 4
Parameters of chloride ingress by field investigation results.

Structure	Period of exposure (year)	Surface chloride content (% binder)	Apparent diffusion coefficient (10^{-12} m^2/s)
HPW1	4	0.82	5.23
HPW2-3#4#	4	1.22	3.42
HPW2-5#6#	4	1.04	2.98
HPW2-7#	4	1.30	2.92
HPW3	8	0.92	1.36
HPW4	12	0.92	0.90
HPW5-6#	14	1.74	2.77
HPW6	18	1.78	2.35
HPW7	17	2.74	0.78
HPW8	20	2.66	0.86
HPW5-5#	20	1.38	1.15
HPW10	21	2.58	0.90
HPW11	23	1.68	3.45
HPW12	24	2.12	0.49
HPW9	26	4.16	1.19
HPW13	29	1.04	2.35
HPW14	36	4.12	0.35

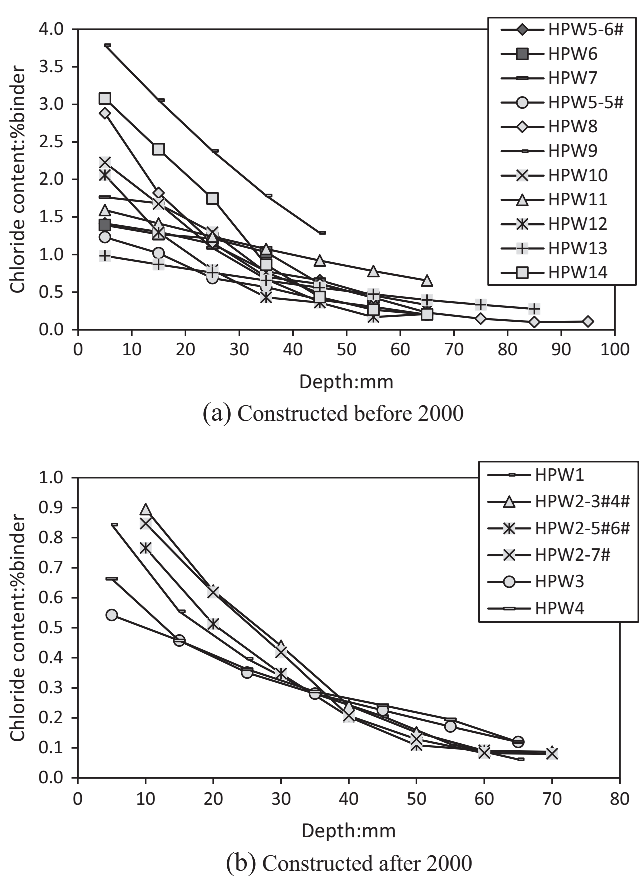


Fig. 3. Chloride profiles from the beams of real wharf structures.

diffusivity with exposure time. Physically, the decrease of D_a with time is related to the long-term evolution of microstructure of concrete materials.

In the following, the resulting chloride profiles are fitted to the Fick's second law to obtain the apparent C_{sa} and the apparent D_a. The initial chloride content of the concrete, C_0, was assumed to be 0 for exposure trials and less than 0.1% mass of cement for existing wharf structures, because the initial chloride content is limited to 0.1% in codes for reinforce concrete members in marine environment [26,27].

3.2. Chloride files

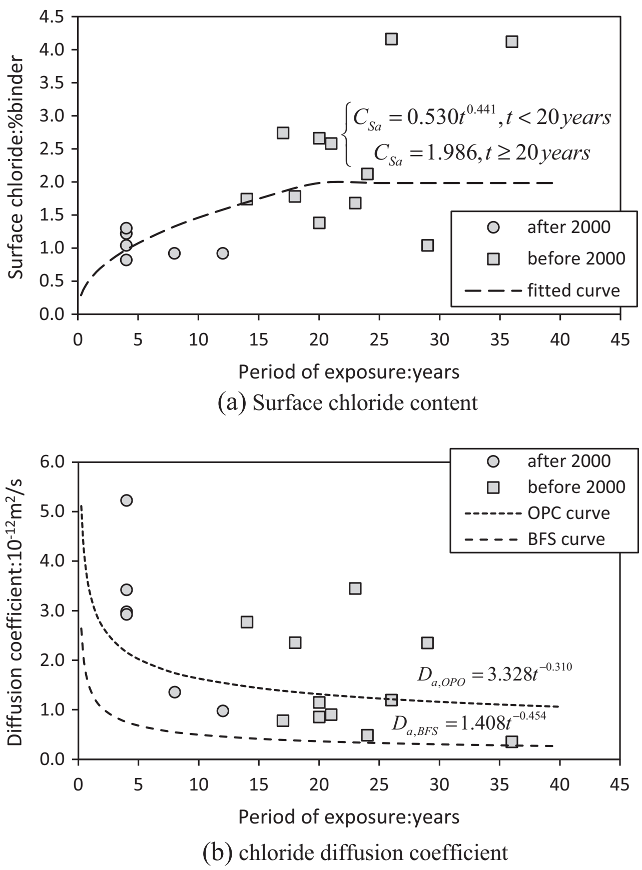
To assess chloride penetration, the concrete powder samples were exacted from the cross beams of existing wharf structures by drilling. Drilling was made at every10 mm depth upon reaching reinforcement, i.e., 5 drillings were made and 5 concrete dust samples were collected for a concrete cover of 50 mm. The number of drilling holes was 8 for each tested beam. In the laboratory, the concrete dust samples at different depth were pulverized firstly, and then put through a nitric acid digestion process, and finally the chloride content at different depth was determined using automatic potentiometric silver nitrate titration of the remaining liquid solutions, thus the chloride ingress profile was obtained for each drilling hole.

The measured chloride ingress profiles from the cross beams of the 17 wharf structures are presented in Fig. 3(a) and (b) for the wharf structures constructed before and after 2000, respectively. For clarity only the mean profiles are presented in the figures. With the chloride content of the powder sample at the depth of 0–10 mm discarded, the apparent diffusion coefficient and the surface chloride content can be determined by fitting the measured data to Eq. (1), and they are listed in Table 4.

The chloride profiles of the wharf structures constructed after 2000 (mineral admixture-added concrete) are generally lower than those of the wharf structures constructed before 2000 (OPC concrete), because their period of exposure is shorter. To demonstrate the effects of the exposure time on chloride ingress, the surface chloride content and the chloride diffusion coefficient derived from the chloride profiles are shown in Fig. 4(a) and (b) respectively against the period of exposure. Both the surface chloride content and the chloride diffusion coefficient are subjected to considerable scatter, which is understandable noting the results were derived from a variety of wharf structures under different working conditions.

The surface chloride content presents an increasing trend with exposure time for OPC concrete, and it appears to stabilize after 20 years. The results also suggest that OPC concrete has higher surface chloride content than the mineral admixture-added concrete. This may be due to the difference in w/c, or due to the incorporation of mineral-admixture, however no clear explanations are available at present.

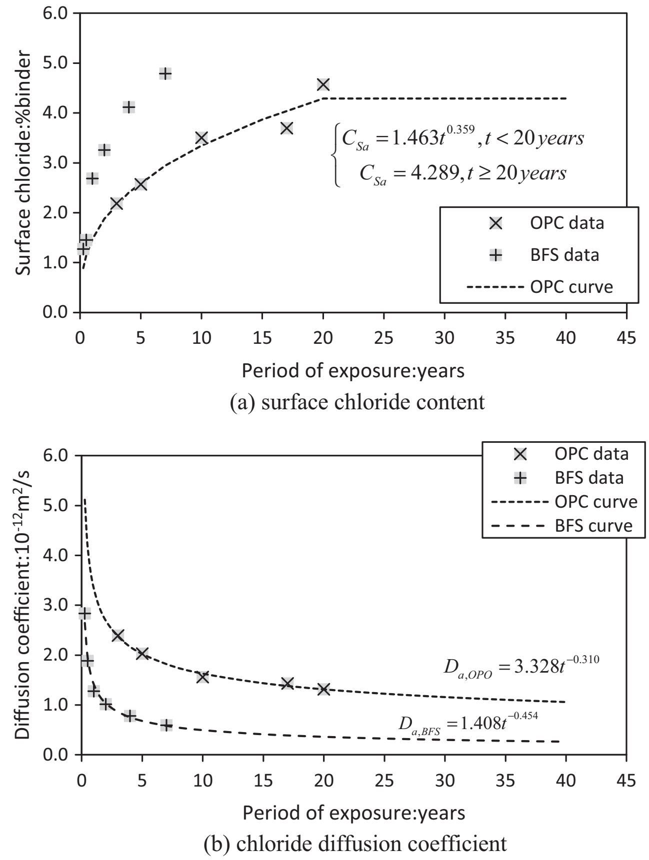
For the mineral admixture-added concrete (wharf structures constructed after 2000), the chloride diffusion coefficient has significant reduction as the exposure time becomes longer, e.g. after



(a) Surface chloride content

(b) chloride diffusion coefficient

Fig. 4. Variation of the chloride ingress parameters with the period of exposure (derived from wharf structures).



(a) surface chloride content

(b) chloride diffusion coefficient

Fig. 5. Variation of the chloride ingress parameters with the period of exposure (derived from exposure trials).

10 years exposure, the chloride diffusion coefficient drops to lower than $1.0 * 10^{-12}$ m^2/s. For the OPC concrete (wharf structures constructed before 2000), it appears that the chloride diffusion coefficient has no decreasing trend with time when the exposure time is more than 15 years. And the chloride diffusivity of OPC concrete is greatly larger than that of admixture-added concrete.

Although the results come from different wharf structures and large uncertainties are associated with them, the trends shown herein are representative of typical wharf structures in marine environment, and similar results have been found in previous studies [18,29,30], which demonstrate the necessity to consider the time-dependency of these two parameters in durability design and assessment of RC structures in marine environments [31].

The time-dependent characteristics of surface chloride content and chloride diffusion coefficient of the OPC concrete and the BFS concrete were also investigated by exposure trials of RC specimens. Using the measured chloride ingress data from the exposure trials at different times, the surface chloride content and chloride diffusion coefficient were determined by the best fitting diffusion profiles using Eq. (1) with C_0 being 0, and plotted in Fig. 5(a) and (b). Fig. 5(a) shows the increase of C_{sa} with exposure time for both OPC concrete and BFS concrete. It can be seen that the BFS concrete has higher C_{sa} values than OPC concrete. However, noting that the exposure specimens of the two type concretes were cast at different times and had different w/c, i.e., 0.4 and 0.35 respectively, the difference in C_{sa} may be attributed to multifarious reasons, although some researches did show that the C_{sa} of BFS concrete is larger than that of OPC concrete [3]. Fig. 5(b) shows the decrease of chloride diffusion coefficient with exposure time for both OPC concrete and BFS concrete. Compared with the OPC concrete, the BFS concrete has significantly high rate of decreasing in D_a. The D_a of BFS concrete is approximately 1/4 of that of OPC concrete after 10 years, and the difference increases with the exposure time, indicating that the BFS concrete has substantially better resistance to chloride penetration than the OPC concrete.The mean relationship between the surface chloride content and the exposure time can be expressed [18], by a power function

$$C_{sa}(t) = C_{sa,1} \cdot t^n \tag{3}$$

with $C_{sa,1}$ stands for the surface chloride content after 1 year exposure; n is an empirical coefficient;. Parameters $C_{sa,1}$ and n can be obtained by applying Eq. (3) to the experimental results. Since the difference in C_{sa} between the OPC concrete and the BFS concrete is not evident from the long-term filed results, the paper neglected this difference, and the relation between C_{sa} and t was obtained from the data of OPC concrete, as shown in Figs. 5a and 4a for exposure specimens and real wharf structures respectively. It appears that the surface chloride contents of the cross beams of the wharf structures are much lower than those of the exposure specimens. One possible reason is that the beams of wharf structures, although located at the splashing zone, are near the atmospheric zone in height (see Fig. 2 for reference), where the exposure condition becomes less severe compared with the locations of exposure specimens (the locations of exposure specimens are also shown in Fig. 2, which are close to the sea level). To reflect the change of surface chloride content with height in splash zone, the zone where the cross beams locate is named "semi-splash" zone in the paper. The existing of "semi-splash" zone is evident in the field investigation of wharf structures. Fig. 6 presents the comparison of chloride ingress profiles between the cross beam and the pile cap of a wharf

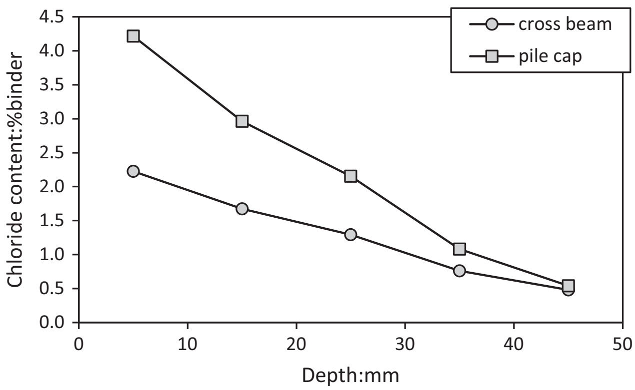


Fig. 6. Comparison of chloride ingress profiles between the cross beam and the pile cap of HPW10.

structure (HPW10). Both members locate at the splash zone, but the chloride contents of the pile cap is greatly larger than those of the cross beam. More detailed analysis on this issue will be reported later.

The relation between the chloride diffusion coefficient and the exposure time is expressed in Eq. (2). The parameters $D_{a,0}$ and a with t_0 being 28 days were obtained by fitting Eq. (2) to the measured data, and shown in Fig. 5b for OPC concrete and BFS concrete respectively.

Figs. 5b and 4b are compared to examine the difference of chloride diffusion coefficients between the exposure specimens and the real wharf structures. It appears that the chloride diffusivities derived from both the exposure specimens and the real wharf structures show similar changing trends with time, except that the chloride diffusion coefficient of the real wharf structure is slightly smaller and associated with significantly large scatter. Therefore, the mean relations derived from exposure trails data between the chloride diffusion coefficient and the exposure time for OPC concrete and BFS concrete are also representative of real wharf structures since their results are basically consistent.

3.3. Probability model of chloride ingress parameters

The uncertainty associated with the measurements of chloride ingress was analyzed in this section to determine the probability model of chloride ingress parameters, C_{sa} and D_a, whose mean values have been determined in last section. The typical scatter of measured chloride ingress can be found in Fig. 7a and b. Fig. 7a shows the 9 chloride ingress samples of exposure specimens at the age of 3 years; Fig. 7b shows the 8 chloride ingress samples of the wharf structure HPW7 whose age is 17 years. The coefficient of variation (COV) associated with the chloride content is around 0.25 in Fig. 7a, and around 0.40 in Fig. 7b. Similar observations were found in other chloride ingress profiles of exposure specimens and wharf structures, so the COV for the chloride content at different depths is 0.25 for exposure specimens and 0.40 for wharf structures.

The surface chloride content and the chloride diffusion coefficient were derived from the measured chloride ingress profiles by regression analysis, thus their uncertainties come from two sources: (1) the intrinsic randomness (aleatory), reflected by the scatter of chloride ingress data about their means, as seen in Fig. 7, and (2) the errors introduced by the assumed model, associated with the regression analysis (epistemic). To analyze the uncertainty associated with C_{sa}, Eq. (3) is transformed into its logarithmic form:

$$\ln C_{sa}(t) = \ln C_{sa,1} + n \ln t \tag{4}$$

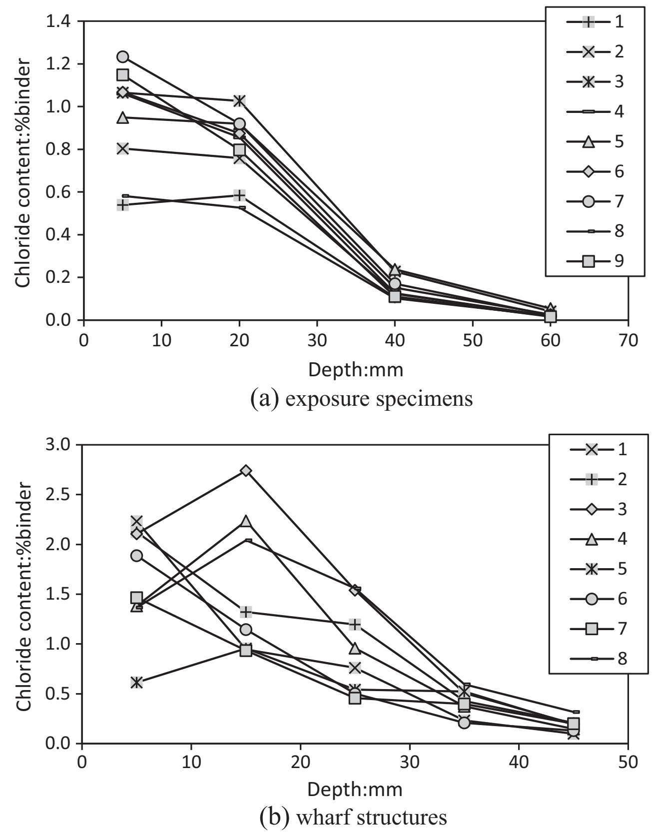


Fig. 7. Scatter of measured chloride ingress profiles.

A simple linear regression analysis can be performed to determine the constants $C_{sa,1}$ and n. The conditional standard deviation of $\ln C_{sa}$ given t can be estimated by:

$$\mathrm{SD}(\ln C_{sa}) = \sqrt{\frac{\sum_{i=1}^{N}\left(\ln C_{sa}(t_i) - \ln \hat{C}_{sa}(t_i)\right)^2}{N-2}} \tag{5}$$

in which $C_{sa}(t_i)$ is the measured surface chloride content at time t_i; $\hat{C}_{sa}(t_i)$ is the calculated surface chloride content at time t_i according to the derived mean relationship; N is the total number of data.

For exposure specimens (the data are plotted in Fig. 5a), the standard deviation of $\ln C_{sa}$ was calculated, which is 0.07; while for wharf structures (the data are plotted in Fig. 4a), $\mathrm{SD}(\ln C_{sa})$ is much larger, it is 0.42.

As suggested by existing researches [13,31,32], the chloride content follows lognormal distribution, thus $\ln C_{sa}$ follows normal distribution. The standard deviation of $\ln C_{sa}$, which is called the logarithmic standard deviation of C_s, is denoted by ξ_{Csa}. Including epistemic and aleatory uncertainties, ξ_{Csa} is written as:

$$\xi_{Csa} = \sqrt{\left(\xi_{Csa}^{(1)}\right)^2 + \left(\xi_{Csa}^{(2)}\right)^2} \tag{6}$$

with $\xi^{(1)}{}_{Csa}$ accounting for the errors associated with the regression analysis, which is 0.07 and 0.42 for exposure trials and real structure respectively, $\xi^{(2)}{}_{Csa}$ accounting for the scatter of data samples around the mean, which is determined from the COV of the chloride content samples at the same exposure time. For exposure trials and real structures $\xi_{Csa}^{2} = \sqrt{\ln(1+\mathrm{COV}^2)} = 0.25$ and 0.39 respectively.

The uncertainty associated with the chloride diffusion coefficient can be analyzed similarly to that of the surface chloride content since they both follow power-law function with time.

Table 5
Statistical properties of chloride ingress parameters.

Random variable	Distribution	Statistical property	Calculations and values
C_{sa} from exposure specimens	Lognormal	Mean (% binder)	$\begin{cases} C_{Sa} = 1.463t^{0.359}, & t < 20 \text{ years} \\ C_{Sa} = 4.289, & t \geqslant 20 \text{ years} \end{cases}$
		Logarithmic SD	0.260
C_{sa} from wharf structures	Lognormal	Mean (% binder)	$\begin{cases} C_{Sa} = 0.530t^{0.441}, & t < 20 \text{ years} \\ C_{Sa} = 1.986, & t \geqslant 20 \text{ years} \end{cases}$
		Logarithmic SD	0.571
$D_a\,(t)$ for BFS concrete	Lognormal	Mean (10^{-6} mm^2/s)	$D_{a,BFS} = 1.408t^{-0.454}$
		Logarithmic SD	0.210
$D_a\,(t)$ for OPC concrete	Lognormal	Mean (10^{-6} mm^2/s)	$D_{a,OPC} = 3.328t^{-0.310}$
		Logarithmic SD	0.203
Cover thickness, d	Normal	Mean (mm)	60
		SD (mm)	5.3
Critical chloride content, C_{cr}	Beta	Upper bound (% binder)	1.25
		Lower bound (% binder)	0.45
		Alpha	0.22
		Beta	0.36

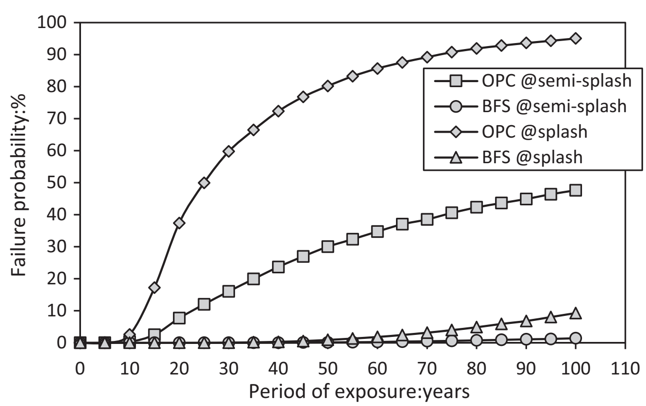


Fig. 8. The probability of corrosion initiation of reinforcement with exposure time.

In a summary, the means and standard deviations of C_{sa} and D_a can be found in Table 5.

4. Service life prediction

Since the chloride ingress processes involve great uncertainties and randomness, a probabilistic approach is applied in this section to analyze the penetration of chloride ions into concrete as well as to predict the service life of RC structures in marine environment.

The corrosion of concrete structures is generally described as two stages: corrosion initiation and corrosion propagation [33,34]. The corrosion initiation stage corresponds to the process of chloride ions penetrating into concrete until the reinforcing steels are depassivated, and it occupies the most part of the service life of RC structures in marine environment because the concrete cracking develops quickly once the corrosion of steel bars begins. Therefore, the duration of corrosion initiation stage is often used to represent the service life or durability of RC structures. The probability of corrosion, $P(t)$, at a given time t is given as:

$$P(t) = \Pr[C(d,t) > C_{cr}] \tag{7}$$

in which, d is the depth of concrete cover; C_{cr} is the chloride threshold level at which corrosion of steel reinforcement initiates. d and C_{cr} are also random variables, their statistical properties have been investigated thoroughly to support the durability design and construction of Hongkong-Zhuhai-Macau sea-link project [12,13], and are also given in Table 5.

A Monte Carlo simulation was used to generate the relationship between the probability of corrosion initiation and the exposure time. The increase of corrosion probability with exposure time is plot in Fig. 8 for 4 cases: (1) the OPC concrete at splash zone; (2) the BFS concrete at splash zone; (3) the OPC concrete at semi-splash zone and (4) the BFS concrete at semi-splash zone. The comparisons between case 1 & case 3 and between case 2 & case 4 demonstrate the effect of semi-splash zone. It can be found that the corrosion probability is significantly reduced if the member locates at the semi-splash zone than at the splash zone, e.g., at 50 years, the failure probability is reduced from 80% to 30% for OPC concrete, from 0.92% to 0.14% for BFS concrete. The impact of adding BFS in mix on the durability of concrete is evident from the comparison between the failure probabilities of OPC concrete and BFS concrete. For example, after 100 years of exposure, the corrosion probability of OPC concrete is above 40% even at the semi-splash zone, while the corrosion probability of BFS concrete is below 10% at the splash zone.

By Monte Carlo simulation, the corrosion initiation times, T_i, were also generated, the histograms for the 4 cases are shown in Fig. 9. Great uncertainties are associated with the corrosion initiation time. As expected, the case 1 has the shortest service life, which is around 17 years, while the service life of the case 4 is the longest, which is around 120 years. The effects of surface chloride content and the concrete mix on the expected service life are significant.

5. Conclusions

This study was conducted to investigate the durability assessment of concrete structures in marine environment based on long-term filed investigation results, and a comparison was made between field surveys of existing wharf structures and exposure tails. The following conclusions can be drawn.

(1) Both the real wharf structure survey and the exposure trials show that the surface chloride content and the diffusion coefficient have a strong time dependency at the splash zone. The incorporation of such time dependency is fundamental in the service life prediction of concrete structures.

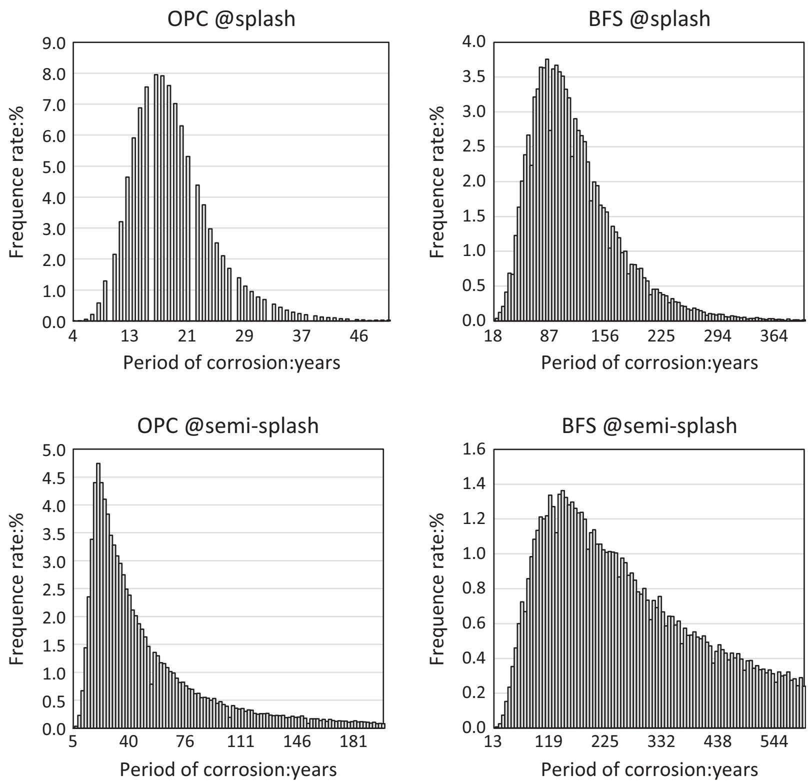


Fig. 9. Histograms of corrosion initiation time.

(2) In splash zone, the surface chloride content varies with the height above the sea level. At the location near the atmospheric zone, named semi-splash zone in the paper, the surface chloride content is significantly smaller than that near the sea level.
(3) Field investigations of real wharf structures and exposure trials both show that the adding of BFS in concrete mix causes a significant improvement on the resistance of concrete to chloride ingress.
(4) The data from real wharf structures and the exposure specimens show consistent decreasing trend of chloride diffusion coefficient with time for both OPC concrete and BFS concrete, which means the chloride diffusion coefficient derived from long-term exposure trails can be reasonably applied to the durability assessment of real marine structures.
(5) The existing of semi-splash zone and the adding of BFS in concrete have significant effects on the service life of concrete structures.

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