Use of Portland Pozzolana Cement to Enhance the Service Life of Reinforced Concrete Exposed to Chloride Attack

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Abstract: This paper discusses the improvement expected in the service life of reinforced concrete (RC) structural elements subjected to chloride-rich environments through the use of blended cement. Comparisons are made between concretes with ordinary portland cement (OPC) and fly ash-based portland pozzolana cement (PPC) at three water-to-cement ratios ($w/c = 0.57$, 0.47, and 0.37). Through a comprehensive experimental program, the apparent chloride diffusion coefficient (D_c) and corrosion current density (i_{corr}) were evaluated for these concretes. The study reveals that (1) although service life depends on both initiation and propagation periods, the propagation period is less significant when the severity of the environment is high; and (2) the service life of an RC structure can double if PPC is used instead of OPC when chloride-induced corrosion is critical. **DOI: 10.1061/(ASCE)MT.1943-5533.0001293.** © 2015 American Society of Civil Engineers.

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Introduction

The highly alkaline environment in fresh and uncontaminated concrete protects the embedded reinforcement bars (rebars) in structural concrete from corrosion. Such protection is mainly due to the presence of a dense ferrous (Fe^{2+}) or ferric (Fe^{3+}) oxide layer (known as passive film) that prevents further oxidization of the rebar (Broomfield 2006). Chlorides from the environment can, however, diffuse through the cover concrete. The corrosion of the rebar initiates when the chloride concentration at the rebar's surface reaches a critical chloride threshold concentration (Cl_{th}), the value of which becomes an important parameter in the estimation of service life (Richardson 2002). Chloride-induced corrosion is catalytic in nature and is very difficult to stop once initiated. The volume of the corrosion products thus generated can be approximately 2–8 times that of the original volume of the steel (Yu and Bull 2006). This expansion can create significant tensile stresses, resulting in cracking and spalling of the cover concrete. Typically, a particular corrosion-related damage level such as reduction in rebar cross-sectional area (e.g., 10% or 25%) is taken to define the end of

service life unless adequate repair work is performed (Andrade et al. 1990).

Many investigators have considered RC corrosion deterioration in two phases: initiation and propagation (Tuutti 1982; Amey et al. 1998). As shown schematically in Fig. 1, the duration of each phase is important for the overall service life of a RC structure exposed to chlorides and the initiation of repair (Pillai 2003). In the initiation phase, the main processes are related to the ingress of chloride ions toward the rebar and the corrosion rate is negligible. The initiation phase endures as long as reinforcement passivity is maintained, which depends on the temperature, humidity, and chloride conditions of the environment; the quality of the cover concrete; and the Cl_{th} of the embedded steel. The propagation phase is the time between corrosion initiation and the expected damage, the level of which is normally based on a maximum reduction in the rebar cross section (or cover concrete cracking, spalling, etc.). It has been reported that the initial rate of corrosion is uniform or constant (i.e., stable) but can increase and vary (or become unstable) later on (Austroads 2000) depending on the rate of ingress of moisture, chlorides, and oxygen, and/or rate of removal of the existing corrosion products (which provide protection against further corrosion). The corrosion process/progress at later stages can differ significantly from that in stages occurring before the cover concrete is cracked. Different models have been proposed for accounting for such aspects in determining the loss in area of rebar subjected to corrosion (Rodriguez et al. 1996; Raupach 2006). Consequently, rebar corrosion can be classified into three stages (Fig. 2): initiation, stable propagation, and unstable/unpredictable propagation (Austroads 2000). The reasonable estimation of the initiation, stable propagation, and unstable propagation stages is the basis for service life prediction.

Many researchers have proposed different methodologies for predicting rebar service life when limited by chloride attack (Ahmad et al. 1997; Anoop et al. 2002; Balaji Rao et al. 2008; ACI 2010a; *Life-365*). However, in most of the literature, service life is approximated to be only the length of the corrosion initiation phase, which may be overconservative because of the

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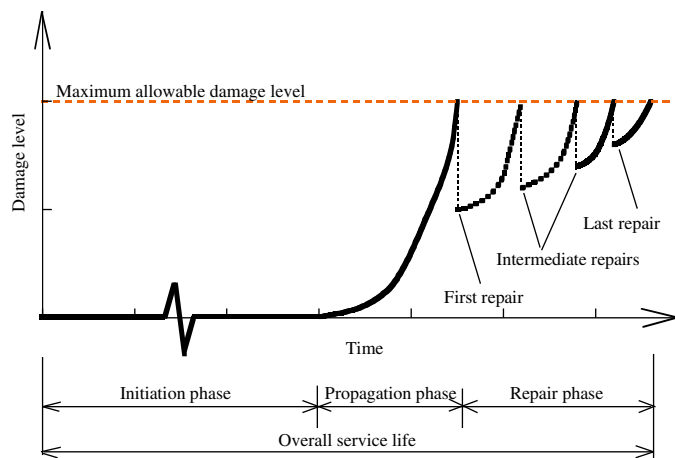
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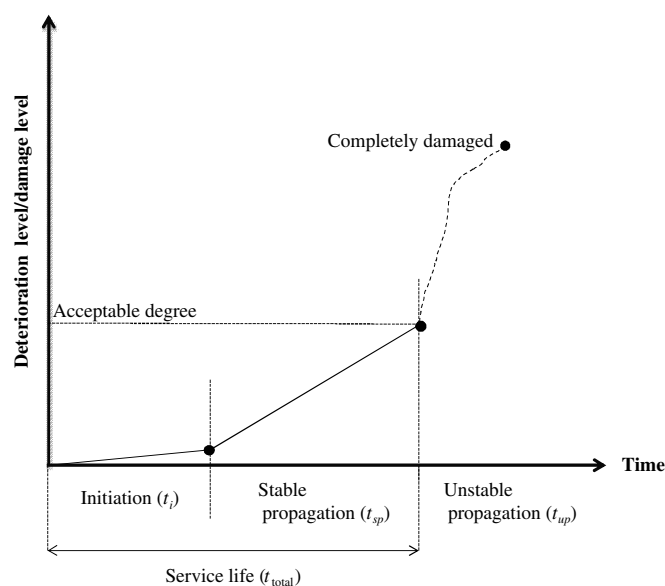**Fig. 1.** Phases in the service life of a RC structure

Fig. 2. Stages of deterioration in rebar corrosion (adapted from Austroads 2000)

insignificant cross-sectional rebar loss at the end of the corrosion initiation phase (Austroads 2000). A more viable approach may be that suggested by Rodriguez et al. (1996), where the propagation period depends on the limit of the damage level, such as rebar section loss or cracking/spalling. In the present study, service life is taken as the sum of the corrosion initiation period (t_i) and the stable corrosion propagation period (t_{sp}), as defined in Austroads (2000).

Diffusion of chlorides in concrete is complex and slow. Although many methodologies to estimate chloride profiles/chloride diffusion coefficients have been reported in the literature (Zhang and Gjørv 1994; Malhotra and Mehta 2002; Ann et al. 2009; Luping et al. 2012), not much information is available for cracked concrete. Wang et al. (1997) and Aldea et al. (1999a, b) studied the relationship of cracking, water permeability, and diffusion, and showed that both permeability and diffusion increase with crack width. Wang et al. (1997) found that crack-opening displacements (CODs) smaller than 50 μm had no effect on concrete permeability. However, permeability increased rapidly when COD increased to 200 microns; and beyond a COD of 200 μm, the rate

Table 1. Chemical Composition of OPC and PPC Cements

Compound	OPC (%)	PPC (%)
CaO	63.0–64.0	40.0–43.0
SiO_2	18.8–19.1	28.0–32.0
Al_2O_3	4.6–6.4	8.3–10.0
Fe_2O_3	3.7–5.2	3.8–6.0
MgO	0.7–3.0	1.0–2.0
SO_3	1.2–1.7	1.9–2.8

Table 2. Physical and Engineering Properties of the Materials Used

Properties	OPC	PPC	Sand	Coarse aggregates
Specific gravity	3.15	2.98	2.67	2.68
Bulk density (kg/m^3)	—	—	—	1,600
Fineness modulus	—	—	2.78	6.81
Initial setting time	140 min	160 min	—	—
Final setting time	220 min	360 min	—	—
Compressive strength of mortar cubes at 28 days (MPa)	48.9	48.0	—	—

Table 3. Quantities of Materials Used per Cubic Meter of Concrete

w/c	Cement	Sand	Coarse aggregates
0.57	300	870	1,056
0.47	362	815	1,056
0.37	460	732	1,056

of water permeability increase became steady. There is no established quantitative relationship between crack width and diffusion, but many researchers assume a certain "threshold crack width," below which cracks have little influence on transport mainly because of the self-healing effects inside them (Jacobsen et al. 1996). Wang et al. (1997) and Jang et al. (2011) suggested threshold crack widths of 50 and 80 microns, respectively, beyond which the diffusion coefficient increases. Park et al. (2012) performed accelerated migration tests on a 35-MPa-grade concrete using specimens with various crack widths. The D_c value determined for uncracked concrete was about 6×10^{-12} m^2/s, and it increased significantly when crack width increased ; the ratios between D_c values for cracked and uncracked concrete were 23.5, 38.4, 70.6, and 145.4 for crack widths of 0.1, 0.2, 0.3, and 0.4 mm, respectively. The diffusion coefficient can, however, decrease if the crack opening is sealed, as shown by Spiro (2013). Chloride diffusivity reflects the resistance of the concrete to chloride penetration as opposed to the total charge passed as measured in the rapid chloride permeability test (RCPT) [ASTM C 1202 (ASTM 2009b)].

Experimental Program

Ordinary portland cement (OPC) and fly ash-based portland pozzolana cement (PPC) both available in the Indian market, were used in the experimental study, along with potable water, river sand, and graded crushed granite (with maximum grain sizes of 20 and 12 mm in the ratio of 1.5 to 1). The relevant chemical compositions and the physical and engineering properties of the materials used are presented in Tables 1 and 2, respectively. ACI 211 guidelines (ACI 2010b) were followed in the concrete mix design, and water-to-cement ratios (w/c) of 0.57, 0.47, and 0.37 were used. The corresponding material quantities per cubic meter of concrete are given in Table 3. Specimens such as 150-mm cubes for compressive

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Table 4. 28-Day Compressive Strength and RCPT Values

Parameter	Concrete type	w/c		
		0.57	0.47	0.37
Cube strength (MPa)	OPC	33.2	44.7	53.7
	PPC	30.7	41.4	51.2
RCPT (C)	OPC	2,600	2,100	1,900
	PPC	861	720	520

strength tests and cylinders of 100-mm diameter and 200-mm height for rapid chloride penetration and chloride diffusion coefficient tests were cast. Table 4 lists the 28-day average compressive strength and charge passed in the RCPT per ASTM C 1202 (ASTM 2009b) for the different concretes. In each case, three specimens were tested and the average values were reported. It was observed that the both mechanical and durability parameters improved as the w/c ratio decreased, which was expected. It was also observed that the respective compressive strength of the OPC and PPC concretes used here were nearly the same. However, the RCPT value for PPC concrete is only about one-third that of OPC concrete. Similar trends were reported by Sivasundaram et al. (1989), Prabakar et al. (2011), and Malhotra and Mehta (2002) in high-volume fly ash concretes. The lower RCPT value is attributed in part to depletion of hydroxyl ions due to pozzolanic reactions (Claisse et al. 2010). Nevertheless, the incorporation of fly ash is known to increase resistance to chloride ion penetration because the microstructure is more compact and less conductive (Mangat and Molley 1991; Mangat et al. 1994; Mackechnie and Alexander 1996).

Determination of the Chloride Diffusion Coefficient

The accelerated chloride diffusion test procedure, or NT Build 355 (Nordtest 1997) proposed by Zhang and Gjørv (1994) was employed in this work to determine the steady-state rate of chloride penetration (Gjørv 1994). The setup, shown in Fig. 3, entailed testing 50-mm-thick slices cut from concrete cylinders of 100-mm diameter and 200-mm height at the age of six months. The two reservoirs were filled with 0.25-N sodium chloride (NaCl) and
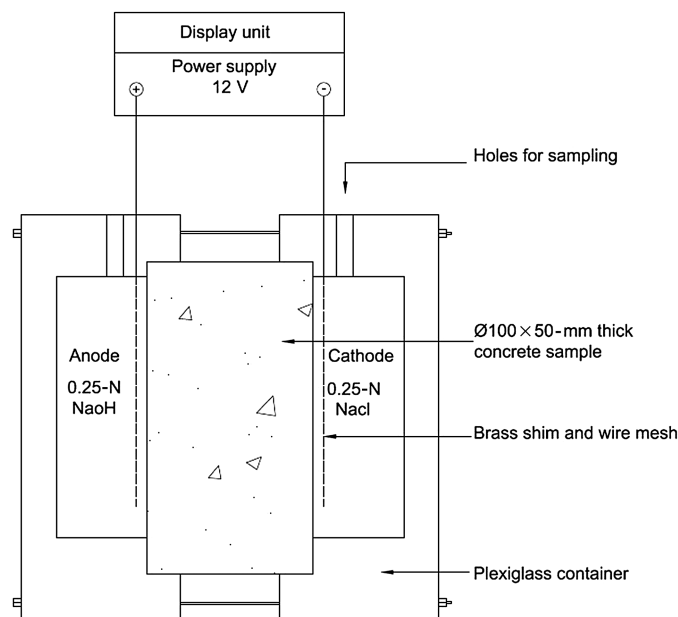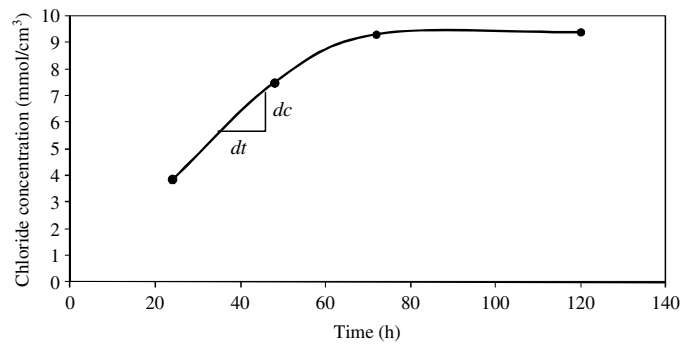
Fig. 4. Typical data obtained in the accelerated chloride diffusion test

0.25-N sodium hydroxide (NaOH) solutions, and a constant voltage of 12 V DC was applied. The chloride ions consequently migrated from the NaCl reservoir, at the negative side of the specimen, to the NaOH compartment, at the positive side. The quantity of chloride ions that migrated was determined by periodically measuring the chloride concentration in the anode solution using volumetric titration. The test continued until a constant concentration was observed over a period of time. A plot was made of chloride concentration (mmol/cm^3) and duration (hours) (Fig. 4) for OPC concrete at w/c = 0.57. The slope, dc/dt, over the linear portion of the plot, giving the steady-state migration rate of chloride ion, was determined and used to calculate the apparent diffusion coefficient (Dc) as (Zhang and Gjørv 1994)

$$D_c = \beta_\circ \frac{300 \; kT}{ze_\circ \Delta_\Psi} \frac{LV}{c_\circ A_\circ} \frac{dc}{dt} \ldots (\text{cm}^2/\text{s}) \qquad (1)$$

where $\beta_\circ$ = correction factor for ionic interaction (1.22–1.7 for 0.1–0.5 M of NaCl in this study—i.e., $0.25 \; \text{M} - \beta_o = 1.4$); k = Boltzmann's constant ($= 1.38 \times 10^{-16}$ ergs/K/ion); T = temperature, K (here, 32°C = 305 K); L = specimen thickness (= 5 cm); V = volume of container (= 250 cm^3); z = ion valency in the testing chloride salt (= 1); e_o = charge of proton ($= 4.8 \times 10^{-10}$ esu); Δ_Ψ = applied electrical potential (= 12 V); c_o = initial chloride concentration (= 151 mmol/cm^3 for 0.25-N of NaCl); and A_o = area of specimen cross section (= 78.50 cm^2).

D_c values for the different concretes are presented in Table 5, which shows those values decreasing as w/c decreases, as expected. More important, the values for PPC concretes are significantly lower than those for corresponding OPC concretes; for example, for w/c = 0.57, D_c values for OPC and PPC concrete are 7.67×10^{-13} and 3.92×10^{-13} m^2/s, respectively. The reduction in D_c for PPC concrete can be attributed to the pozzolanic reaction between fly ash and $Ca(OH)_2$ to form secondary calcium silicate hydrates, which cause pore size refinement, lower conductivity, fly ash chloride-binding ability, depletion of hydroxyl ions, and the like (Saraswathy and Song 2007; Yuan et al. 2009; Claisse et al. 2010; Bhaskar 2013).

Accelerated Corrosion Tests

A specially designed U-shaped specimen (with and without precracks) with a thermo-mechanically treated (TMT) rebar of 12-mm diameter, embedded with 20-mm clear cover at the bottom (Fig. 5), was used for studying corrosion. The specimens were cured in water for 28 days and in air for 30 days, after which cracks of the required width were created by stressing the tie rods. Initially, the specimens were immersed in a 3.5% NaCl solution such that the solution level was well above the rebar position, and a 10-V anodic

Fig. 3. Accelerated chloride diffusion test setup

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Table 5. Apparent Diffusion Coefficients and Estimated Corrosion Initiation Periods (t_i in Years) for Different Cover Thicknesses (x)

Concrete	Apparent diffusion coefficient, D_c (m^2/sec)	$x = 20$ mm	$x = 30$ mm	$x = 40$ mm	$x = 60$ mm
OPC-0.57-0	7.67×10^{-13}	2.1	4.8	8.6	19.4
PPC-0.57-0	3.92×10^{-13}	4.2	9.5	16.8	37.9
OPC-0.47-0	4.40×10^{-13}	3.8	8.4	15.0	33.8
PPC-0.47-0	1.25×10^{-13}	13.2	29.7	52.8	118.9
OPC-0.37-0	2.40×10^{-13}	6.9	15.5	27.5	61.9
PPC-0.37-0	5.20×10^{-14}	31.7	71.4	127.0	285.7
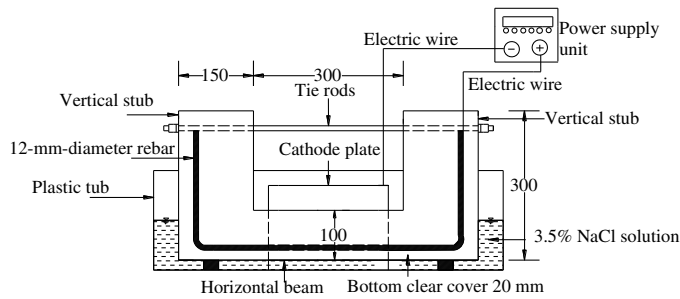
Fig. 5. Specimen immersed in NaCl solution in the accelerated corrosion test (adapted from Bhaskar 2013; Bhaskar et al. 2011)

potential was applied for 22 days. Later, the specimens were broken open and the rebars removed and cleaned per ASTM G1 (ASTM 2009a) to determine the extent of corrosion. The gravimetric weight loss (i.e., the amount of corrosion) was determined as the loss in weight over the middle 250-mm rebar length [see Bhaskar et al. (2011), and Bhaskar (2013), for test details]. The corrosion current density, i_{corr}, was obtained from the weight loss measurements as (Ijsseling 1986; Ahmad 2009)

$$i_{corr} = \frac{W_l F}{\pi d l W t_{ic}} \quad (2)$$

where W = equivalent weight of steel, taken as the ratio of atomic weight of iron to the valency of iron (27.925 g); W_l = weight loss in the rebar after 22 days (g); F = Faraday's constant (96,487 amp s); l = length of rebar considered (25 cm); d = diameter of the rebar (1.2 cm); and t_{ic} = duration of induced corrosion (22 days = 1,900,800 s).

Methodology for Service Life Estimation

As mentioned earlier, the service life (t_{total}) is taken as the sum of the corrosion initiation period (t_i) and the stable propagation period (t_{sp}), which is expressed as (Austroads 2000)

$$t_{total} = t_i + t_{sp} \quad (3)$$

Corrosion Initiation Period

Many researchers have used the closed-form solution of Fick's second law of diffusion for predicting t_i when the concrete is exposed to chlorides as per ACI 365.1 R (ACI 2010a; Life-365). After applying specific boundary conditions, $C_x = 0$ at $t = 0$, $0 < x < \infty$; $C_x = C_s$ at $x = 0$, $0 < t < \infty$, the model can be simplified as (Andrade 1993; Gjørv 1994)

$$C(x, t) = C_s \left[1 - \text{erf} \left(\frac{x}{2\sqrt{(D_c t_i)}} \right) \right] \quad (4)$$

where $C(x, t)$ = chloride concentration at depth x (or Cl_{th} in this case) after time t_i; C_s = chloride concentration at the concrete surface; erf = Gaussian error function; and x = clear cover thickness.

Once the values of Cl_{th}, C_s, and D_c are available, t_i can be obtained as the time taken to reach Cl_{th} at the rebar surface. Though it is well known that the parameters Cl_{th}, C_s, and D_c vary significantly, reasonable values are assumed based on knowledge of exposure conditions and the materials and mixture proportions used. However, this has to be done with care. The value of Cl_{th} depends on various factors that include the microstructure and metallurgical parameters of the rebar and the surrounding concrete, the pH of the concrete pore solution, and the test procedures used to evaluate this parameter (Alonso et al. 2000; Pillai and Trejo 2005; Angst et al. 2009). Taking a higher than appropriate threshold value leads to overestimation of the service life provided other conditions are the same. In the present study, experimentally observed D_c values and literature-based Cl_{th} and C_s values were used (Austroads 2000; Life-365). Cl_{th} was assumed as 0.05% by mass of concrete (i.e., 0.4% by mass of cement), a value commonly used for the estimation of t_i (Austroads 2000; ACI 2010a; Life-365). In general, the value of C_s increases with exposure time, although data from the field indicate that it tends to reach a "maximum" value in the range of 5–6% by mass of cement, which depends on concrete porosity, chloride-binding capacity and aggressiveness of the chloride environment (Ann et al. 2009). However, to be on the conservative side, a higher C_s value of about 8% by mass of cement was assumed here (i.e., 1% by mass of concrete) as Life-365 for extreme marine tidal zone exposure.

Stable Propagation Period

A value for the stable propagation period (t_{sp}) can be reasonably estimated using the model proposed by Rodriguez et al. (1996) in terms of the evolution of the rebar diameter $\theta(t)$ (in mm) with time t (in years)

$$\theta(t) = \theta(0) - p(t) \quad (5)$$

$$p(t) = 0.0116 \alpha i_{corr} t_{sp} \quad (6)$$

where $\theta(0)$ = initial diameter of the rebar (in mm); $p(t)$ = penetration of corrosion attack (in mm); i_{corr} = average corrosion current density (in $\mu A/cm^2$); 0.0116 = factor that converts $\mu A/cm^2$ to mm/year; and α is a factor that accounts for highly localized pitting, normally associated with chloride-induced corrosion; $\alpha = 5$–10 (González et al. 1995). This has been formulated based on Faraday's Law, with the assumption that the number of electrons freed by Fe ions during the corrosion reaction is 2.

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Table 6. i_{corr} Values Obtained in the Laboratory for the Different Concretes

Concrete	i_{corr} (μA/cm^2)	Concrete	i_{corr} (μA/cm^2)
OPC-0.57-0	768	PPC-0.57-0	289
OPC-0.57-0.2	1,258	PPC-0.57-0.2	631
OPC-0.57-0.4	1,385	PPC-0.57-0.4	731
OPC-0.47-0	581	PPC-0.47-0	212
OPC-0.47-0.2	968	PPC-0.47-0.2	548
OPC-0.47-0.4	1,045	PPC-0.47-0.4	677
OPC-0.37-0	500	PPC-0.37-0	191
OPC-0.37-0.2	758	PPC-0.37-0.2	482
OPC-0.37-0.4	939	PPC-0.37-0.4	513

Service Life Estimates for the Different Concretes

Based on Eq. (4), the corrosion initiation period, t_i is given as

$$t_i = \frac{x^2}{4D_c[\text{erf}^{-1}(1 - \frac{Cl_{th}}{C_s})]^2} \tag{7}$$

Accordingly, t_i has been estimated, for various cover thicknesses, with Cl_{th} = 0.05% by weight of concrete and $C_s = 1\%$ by weight of concrete; its values are given in Table 5 for the different OPC and PPC concretes. Each series of data is denoted in the form "cement type-w/c-flexural crack width" (i.e., OPC-0.57-0 denotes a OPC binder, w/c of 0.57, and zero crack width or uncracked concrete). Corresponding to a cover thickness of 20 mm, it can be seen that the corrosion initiation period of PPC-0.57-0 concrete is 4.2 years, which is double that of the corresponding OPC-0.57-0 concrete. It can also be observed that the corrosion initiation period increases as w/c decreases, as expected (*Life-365*). The corrosion initiation period is obviously strongly dependent on cover thickness, the minimum value of which is specified in design codes as a function of exposure condition.

For the computation of t_{sp}, the corrosion current density (i_{corr}) and the damage level are needed. Based on gravimetric weight loss measurements, i_{corr} values for different concretes were estimated using Eq. (2); they are presented in Table 6. For PPC concrete rebar, i_{corr} is lower than it is for OPC concrete rebar. This is mainly due to the increased resistivity and lower chloride ion permeability of PPC concrete (Scott and Alexander 2007; Bhaskar et al. 2011). It is generally observed that i_{corr} for cracked concrete is much higher than that for uncracked concrete, and it increases with crack width. Nevertheless, the difference in i_{corr} between the uncracked specimens and those with 0.2-mm crack width was higher than that between the specimens with 0.2- and 0.4-mm crack widths. This implies that the presence of crack influences corrosion more than actual crack width does, perhaps because of the effect of localized pitting corrosion, and can indicate a nonlinear relationship between flexural crack width and degree of corrosion. The trend observed contradicts the argument of some researchers that crack width has no influence on corrosion rate (Beeby 1983; François and Arliguie 1999; François et al. 2012), which may be true only for small crack widths or short-term corrosion. However, it is clear that the corrosion rate reflected by i_{corr} will be higher in cracked concrete, thereby reducing t_{sp}. More important, the beneficial effect of fly ash was confirmed by the results showing much higher i_{corr} values for cracked OPC specimens than for PPC specimens; for example, i_{corr} for OPC-0.57-0.2 was almost twice the value obtained for PPC-0.57-0.2.

However, i_{corr}-values obtained under accelerated conditions were, as expected, much higher than those occurring in the field; Andrade et al. (1990) and González et al. (1995) reported values of
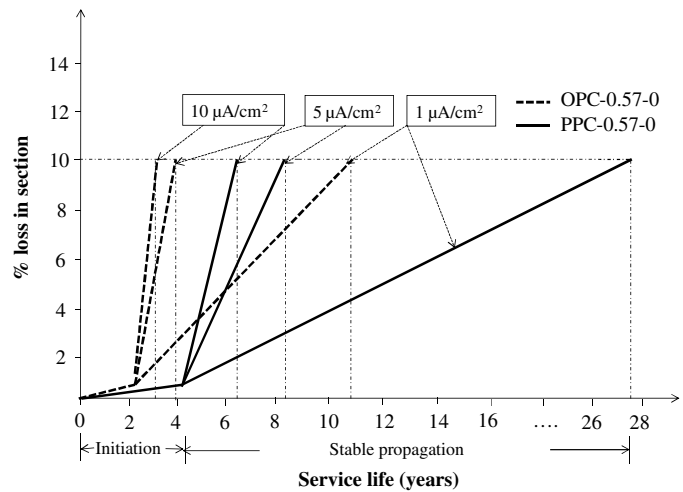

Fig. 6. Estimated loss in rebar cross-sectional area with time, at different i_{corr} values

1–3 μA/cm^2 in the case of active corrosion and stated that values on the order of 10 μA/cm^2 and higher are seldom observed in the field. Therefore, service life estimates in this work considered field i_{corr} values of 1, 5, and 10 μA/cm^2 for OPC-0.57-0 concrete. In the absence of published field data for concretes with fly ash and with different crack widths, the corresponding field i_{corr} values for all concretes were assumed to vary proportionally to those (Table 6) obtained in the accelerated corrosion tests (Morinaga et al. 1994). Consequently, for any concrete, say XC, values were obtained from the ratio of laboratory values of XC and OPC-0.57-0 concretes as

$$(i_{corr,XC})_{\text{Field}} = \left(\frac{i_{corr,XC}}{i_{corr,OPC-0.57-0}}\right)_{\text{Lab}} \times (i_{corr,OPC-0.57-0})_{\text{Field}} \tag{8}$$

Another important parameter that is needed for estimating t_{sp} is the definition of deterioration corresponding to the limit of serviceability or load-carrying capacity of the structure. Here, the end of the stable propagation period (t_{sp}) was taken to be when the cross-sectional area of the steel rebar had decreased by 10%, in accordance with CEB (1983) and Andrade et al. (1990). Further, the pitting factor α was taken as 6 for uncracked concretes and as 8 and 10 for concretes with crack widths of 0.2 and 0.4 mm, respectively; more rebar pitting was observed as the crack width increased.

Service Life Comparisons for Uncracked Concrete

The plots in Fig. 6 show the evolution of rebar cross-sectional area loss with time for the field i_{corr} values 1, 5, and 10 μA/cm^2, considering the parameters obtained for OPC-0.57-0 and PPC-0.57-0 concretes. The plots reflect the slower rate of corrosion initiation (i.e., longer t_i) in the PPC concrete than in the OPC concrete, higher rates of corrosion propagation with higher i_{corr} values, and longer periods of stable corrosion propagation (t_{sp}) in the PPC concrete. The values for the two stages of corrosion and the total estimated service life (t_{total}) for the different uncracked concretes are given in Table 7. Similar trends are also seen for the concretes with lower w/c. It is clear that taking the same t_{sp} value for different types of concrete and corrosion rates, such as the six years in *Life-365*, may be unconservative.

It can be seen that t_{total} for PPC-0.57-0 is approximately 2.6 times that of OPC-0.57-0 when i_{corr} for the latter is 1 μA/cm^2.

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Table 7. Service Life Estimates for Uncracked Concretes for Different Field i_{corr} Values

Concrete	Corrosion initiation period, t_i (years)	Stable propagation period, t_{sp} (years)	Service life, t_{total} (years)
i_{corr} value of OPC-0.57-0 concrete = 1 $\mu A/cm^2$			
OPC-0.57-0	2.1	8.6	10.7
PPC-0.57-0	4.2	23.3	27.5
OPC-0.47-0	3.8	12.3	16.1
PPC-0.47-0	13.2	28.7	41.9
OPC-0.37-0	6.9	14.4	21.3
PPC-0.37-0	31.7	43.1	74.8
i_{corr} value of OPC-0.57-0 concrete = 5 $\mu A/cm^2$			
OPC-0.57-0	2.1	1.7	3.8
PPC-0.57-0	4.2	4.6	8.8
OPC-0.47-0	3.8	2.4	6.2
PPC-0.47-0	13.2	5.8	19.0
OPC-0.37-0	6.9	2.8	9.7
PPC-0.37-0	31.7	8.6	40.3
i_{corr} value of OPC-0.57-0 concrete = 10 $\mu A/cm^2$			
OPC-0.57-0	2.1	0.9	3.0
PPC-0.57-0	4.2	2.3	6.5
OPC-0.47-0	3.8	1.2	5.0
PPC-0.47-0	13.2	2.9	16.1
OPC-0.37-0	6.9	1.4	8.3
PPC-0.37-0	31.7	4.3	36.0

Table 8. Estimated Diffusion Coefficients and Corrosion Initiation Periods for Cracked OPC-0.57 Concretes

Concrete	Diffusion coefficient, D_c (m^2/sec)	Corrosion initiation period, t_i (years)
OPC-0.57-0	7.67×10^{-13}	2.1
OPC-0.57-0.2	2.95×10^{-11}	0.06
OPC-0.57-0.4	1.1×10^{-10}	0.01

The relative difference between the t_{total} estimates of the PPC and OPC concretes decreases slightly as the i_{corr} values increase by an order of magnitude; however, the ratio is still more than double. On the other hand, the ratio of the t_{total} estimates for the PPC and OPC concretes increases as the w/c decreases, as expected. The estimates confirm that the PPC concretes have better corrosion resistance than do OPC concretes (Mehta 2002) because of a longer corrosion initiation period and a decreased corrosion rate during the propagation period.

It can be observed from the results that for lower i_{corr}, the duration of both the initiation and the propagation period are significant for overall service life whereas for higher i_{corr} only the initiation period is significant because the propagation period is relatively short (e.g., 0.9 and 2.3 years, respectively, for the OPC-0.57-0 and PPC-0.57-0 concretes when $i_{corr} = 10$ $\mu A/cm^2$).

Effect of Cracking on Service Life Estimates

The proposed methodology can be extended to cracked concrete provided the values of the influencing parameters considered (i.e., D_c, and i_{corr}) are available. In general, the diffusion of chloride ions should be much faster in cracked concrete, making its D_c much higher (again, depending on crack geometry, exposure conditions, etc.) than the D_c of uncracked concrete. Consequently, t_i is much shorter in cracked than in uncracked concrete. Because t_{sp} is also shorter, as indicated by the higher gravimetric weight loss observed in the rebars in cracked concrete, the estimated total

Table 9. Service Life Estimates at a Reference Field i_{corr} Value of 1 $\mu A/cm^2$, with and without Cracks

Concrete	Corrosion initiation period, t_i (years)	Stable propagation period, t_{sp} (years)	Total service life, t_{total} (years)
OPC-0.57-0	2.1	8.6	10.7
PPC-0.57-0	4.2	23.3	27.5
OPC-0.57-0.2	0.06	4.0	4.1
PPC-0.57-0.2	0.11	8.1	8.2
OPC-0.57-0.4	0.01	2.9	2.9
PPC-0.57-0.4	0.03	5.7	5.7

service life in cracked concrete is much shorter than in uncracked concrete. Based on the D_c values obtained by Park et al. (2012) for both cracked and uncracked concrete similar to OPC-0.57, it is considered here that D_c for OPC-0.57-0.2 and OPC-0.57-0.4 can be approximately 38 and 145 times that for OPC-0.57-0. With the consequent values, estimated t_i for these concretes is given in Table 8. As discussed earlier, the corrosion initiation period is very short for both 0.2- and 0.4-mm crack widths. Taking the same ratios for the PPC concretes, service life can be estimated and compared. Table 9 presents the service life for OPC-0.57 and PPC-0.57 concretes with surface crack widths of 0, 0.2, and 0.4 mm for a reference field i_{corr} value of 1 $\mu A/cm^2$. As expected, the presence of the crack considerably decreases service life in all cases. However, it is evident that PPC concretes perform better.

Conclusions

In the present study, overall service life (t_{total}) was taken as the sum of the corrosion initiation period (t_i) and the stable propagation period (t_{sp}), which were estimated from parameters obtained from tests of chloride migration and accelerated corrosion. Within the parameter ranges studied here, the following conclusions can be drawn:

- The corrosion initiation period, t_i, and the stable propagation period, t_{sp}, can be estimated based on experimentally obtained values for diffusion coefficient D_c and the estimated field values of corrosion current density, i_{corr};
- When i_{corr} values are low, both t_i and t_{sp} contribute significantly to overall service life. However, at higher i_{corr}, values only t_i is significant because t_{sp} becomes very short; and
- The estimated service life of PPC-based concrete is at least twice that of corresponding OPC-based concretes, even under cracked conditions, confirming that cement blended with fly ash can be expected to perform significantly better in terms of protecting steel reinforcement when the concrete is subjected to chloride attack.

Acknowledgments

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