



# Chloride-induced corrosion of steel in cracked concrete – Part I: Experimental studies under accelerated and natural marine environments



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## ABSTRACT

Parallel corrosion experiments were carried out for 2¼ years by exposing one half of 210 beam specimens (120 × 130 × 375 mm long) to accelerated laboratory corrosion (cyclic wetting and drying) while the other half underwent natural corrosion in a marine tidal zone. Experimental variables were crack width  $w_{cr}$  (0, incipient crack, 0.4, 0.7 mm), cover  $c$  (20, 40 mm), binder type (PC, PC/GGBS, PC/FA) and w/b ratio (0.40, 0.55). Results show that corrosion rate ( $i_{corr}$ ) was affected by the experimental variables in the following manner:  $i_{corr}$  increased with increase in crack width, and decreased with increase in concrete quality and cover depth. The results also show that the corrosion performance of concretes in the field under natural corrosion cannot be inferred from its performance in the laboratory under accelerated corrosion. Other factors such as corrosion process should be taken into account.

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## 1. Introduction

Corrosion of steel reinforcement is one of the main deterioration mechanisms for reinforced concrete (RC) structures in temperate, marine, and industrial environments. It has become a major durability concern for both asset owners and engineers. If left unabated, it accelerates the deterioration of RC structures and may lead to several inter-related negative consequences including, but not limited to, cracking and spalling of concrete cover, loss of steel cross-section area, degradation of steel-concrete interface bond, and ultimately reduction in service life of the RC structure. In addition, it requires high expenditures for maintenance, repair or replacement, and compromises public safety. Even though corrosion of steel in RC structures can be caused mainly by either ingress of carbon dioxide (carbonation-induced) or chlorides (chloride-induced), the latter is the chief cause of steel corrosion in RC structures. Notably, chloride-induced corrosion causes widespread damage of the RC structure and may ultimately result in failure (depending on the pre-defined limit state) within a relatively short period of time (before the structure meets its target service life).

In the presence of cracks in concrete, the aggressive nature of chloride-induced corrosion and the related rate of deterioration of the RC structure are exacerbated. Cracks impair the durability of RC structures by creating preferential paths for the penetration of corrosion-

inducing species ( $H_2O$ ,  $O_2$ ,  $CO_2$ ,  $Cl^-$ ) [1,2] leading to relatively faster initiation and propagation of steel corrosion and consequently a reduction in service life. If no cracks are present, both the initiation and propagation phases are usually a function of, among other factors, the penetrability of the concrete cover, the cover thickness, resistivity of concrete, and the corrosion resistance of the steel bars [3–5]. In the presence of cracks, studies have shown that the factors affecting corrosion rate in uncracked concrete are still relevant but their effectiveness is significantly reduced [4–9]. The effects of cracks on corrosion vary not only with their width, but also with depth, frequency, orientation (relative to the steel reinforcement), self-healing potential, and activity or dormancy. The study reported in this paper focused only on crack width.

Even though the effect of cover cracking on corrosion has been investigated in the past, most studies have been carried out in the laboratory under accelerated conditions (for which there are no standard test methods) mainly due to the slow nature of natural corrosion process coupled with the urgent need to provide working solutions to the corrosion problems in RC structures. Accelerated corrosion is usually achieved using several techniques including the use of impressed current [10,11], admixed chlorides [10,12], cyclic wetting and drying [13, 14], and the use of simulated concrete pore solutions [15,16]. In this study, a combination of impressed current and cyclic wetting and drying were used, as will be seen in the next section. The results presented in this paper provide useful information with respect to the use of accelerated corrosion results to predict the field corrosion performance of different concretes under similar exposure conditions.

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## 2. Experimental details

### 2.1. Experimental variables and mix proportions

Parallel corrosion experiments were carried out by exposing one half of 210 beam specimens (120 × 130 × 375 mm) to accelerated laboratory corrosion (cyclic 3 days wetting with 5% NaCl solution followed by 4 days air-drying) while the other half were left to undergo natural corrosion in a marine tidal zone in Cape Town (Table Bay Harbour). The beams were cast using five different concretes made using two w/b ratios (0.40 and 0.55), and three binders (100% CEM I 42.5 N plain Portland cement (PC), 50/50 PC/GGBS and 70/30 PC/FA). The 0.55 w/b ratio was not used to make PC specimens. A high yield strength 10 mm diameter steel bar was embedded in each beam. A summary of the concrete mix proportions and selected concrete properties is presented in Table 1. Other experimental variables included cover depth (20 and 40 mm) and crack width (0, incipient crack, 0.4 and 0.7 mm). An incipient crack refers to a crack just induced by 3-point loading of beam specimens and thereafter unloading. Even though previous research has shown that crack widths ≤ 0.3 mm are prone to self-healing [17–19], it was not possible to quantify crack self-healing in the incipient-cracked specimens due to lack of appropriate test equipment. Further, for deeper cover depths (say 40 mm) coupled with the susceptibility to crack healing, it can be expected that incipient cracks will have negligible effect on corrosion rate. The incipient-cracked specimens were therefore made only for the laboratory and field specimens with 20 mm cover. Both ends of the steel bars were covered with electroplating tape and epoxy-coated to provide an effective exposed surface area of approximately 86 cm<sup>2</sup> (approximately 27.5 cm long circumferential steel surface). Just before casting, the bars were cleaned and degreased with acetone. A 10 mm diameter × 150 mm long stainless steel bar was placed in each beam during casting (see Fig. 1) to act as a counter electrode for corrosion rate measurements.

### 2.2. Induction of active corrosion state

After 28-days of water-curing (at 23 ± 2 °C) and 10 days air-drying (temperature: 25 ± 2 °C, relative humidity: 50 ± 5%) in the laboratory, and prior to cracking the beam specimens, anodic impressed current (IC) was used with the intention to initiate an active corrosion rate (i.e. eliminate the corrosion initiation phase) in all the 210 beam specimens. The set-up used to initiate active corrosion in the specimens using IC is shown in Fig. 1. The theoretical time ( $t_{steel\ level}$ ) and amount of IC required to drive the chlorides through the cover depth to the steel level was estimated using Nernst-Planck Equation in the following form:  $t_{steel\ level} = c \cdot [-D(zF/RT) \cdot (\partial E/\partial x)]^{-1}$  where  $D$  is the diffusion

coefficient of the ionic species (m<sup>2</sup>/s) in concrete,  $z$  is the valence of the ionic species (-1 for Cl<sup>-</sup>),  $F$  is Faraday's constant (96,500 C/mol),  $R$  is the universal gas constant (8.314 J/mol.K),  $T$  is the absolute temperature (298 K),  $c$  is the cover depth (20 or 40 mm) and  $E$  is the applied electrical potential (V).

The diffusion coefficient ( $D$ ) was obtained from the measured 28-day chloride conductivity index (CCI) based on empirical correlations between  $D$  and CCI which take into account the effect of marine exposure environment, concrete ageing and binder type [20–22] i.e. the correlation model can estimate the time- and exposure-dependent apparent chloride diffusion coefficient for commonly used binder types (mainly plain PC, 70/30 PC/FA and 50/50 PC/GGBS) and marine exposure environments (tidal, splash and spray) in South Africa [20–22]. Part II of this paper gives more details on the determination of diffusion coefficient from the CCI. The CCI was obtained from the rapid chloride conductivity test [21,23] for each of the concrete mixes in Table 1. The 28-day and 90-day diffusion coefficients of the concretes used are presented in Table 2.

In order to determine the anodic IC required to induce active corrosion rate in the specimens of a given cover depth for each of the five concrete mixes, and after a series of iterations (of time and IC), the time to apply the IC was fixed at 1.5 hours to limit the effective applied current to less than 2.0 Amperes. The term 'effective applied current' is used here to refer to the actual applied current taking into account the corresponding 28-day measured resistance of the (water-saturated) concrete. The IC was limited to less than 2.0 Amperes to ensure chlorides reach the steel and to minimize the steel mass loss, if any. It is important to note that the applied currents were theoretically not expected to cause any steel mass loss; even though not ascertained by actual measurement of chloride content at the steel level, the objective was to drive the chlorides to the steel level within approximately 1.5 hours (taking into account cover depth and concrete quality). Beam specimens of the same binder type, w/b ratio and cover depth were connected in series, and the appropriate anodic IC applied continuously for 1.5 hours. The applied IC currents were as follows, in each case respectively for 20 mm and 40 mm cover depths: (a) PC-40: 0.03 A and 0.13 A, (b) FA-40: 0.44 A and 1.67 A, (c) FA-55: 0.25 A and 1.01 A, (d) SL-40: 0.57 A and 1.70 A, and (e) SL-55: 0.44 A and 1.67 A. After 1.5 hours, all the 72 beams were connected in series and an effective current of 8.6 μA applied for a further 2 hours – this was expected to result in a corrosion rate of approximately 0.1 μA/cm<sup>2</sup> in all the specimens, the assumption being that the whole exposed steel surface area of 86 cm<sup>2</sup> was polarized. A corrosion rate of 0.1 μA/cm<sup>2</sup> is conventionally taken to denote the transition from passive (initiation) to active (propagation) corrosion [24,25].

**Table 1**  
Summary of concrete mix proportions and selected properties.

Material (kg/m <sup>3</sup> )	Binder composition w/b ratio Mix label	100 % PC		50/50 PC/GGBS		70/30 PC/FA	
		0.40	0.55	0.40	0.55	0.40	0.55
		PC-40	SL-40	SL-55	FA-40	FA-55	
Portland cement, PC (CEM I 42.5 N)		500	231	168	324	236	
Ground granulated blastfurnace slag (GGBS)		-	231	168	-	-	
Fly ash (FA)		-	-	-	139	101	
Fine aggregate: Klipheuwel sand (2 mm max.)		529	749	855	749	855	
Coarse aggregate: Granite (13 mm max.)		960	1040	1040	1040	1040	
Water		200	185	185	185	185	
Superplasticizer <sup>a</sup> (SP)		2.1 <sup>b</sup> (0.4) <sup>c</sup>	1.8 (0.4)	0.3 (0.1)	0.4 (0.1)	-	
Slump (mm)		120	105	150	85	200	
28-day compressive strength (MPa)		58.2 (3.0) <sup>d</sup>	48.1 (2.0)	35.3 (0.9)	50.7 (0.9)	28.6 (1.9)	

<sup>a</sup> Chemical base: Naphthalene Formaldehyde Sulphonate.

<sup>b</sup> Percentage of SP by mass of total binder.

<sup>c</sup> Amount of SP in litres/m<sup>3</sup>.

<sup>d</sup> Standard deviation.

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