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Discrete Sacrificial Anodes and Their Use in Service Life Extension of Chloride Contaminated Reinforced Concrete Structures

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Philemon Arito

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Declaration

This dissertation is being submitted for the degree of Master of Science in Engineering in the University of Cape Town, Cape Town, South Africa. It has not been submitted before for any degree or examination in any other University. In addition, I know the meaning of plagiarism and declare that all the work in the document, save for that which is properly acknowledged, is my own.

(Signature of candidate)

Day of 20**12**

Abstract

There is an urgent need for economic and technically sound concrete repair technologies in South Africa as the number of deteriorating RC structures needing repair is increasing. Currently, the South African concrete repair industry mainly relies on the application of patch repairs and corrosion inhibitors while other potentially successful methods such as sacrificial anodes are rarely applied.

Sacrificial anode cathodic protection (SACP) systems use metals that are higher than steel in the electrochemical series. These metals corrode preferentially to steel thereby supplying electrons to the cathodic steel surface. Common SACP systems include discrete zinc anodes installed in slots or drilled holes, non-structural jacket encasements, and overlays made from expanded zinc mesh or any other convenient form.

The effectiveness of discrete SACP systems in service life extension of chloride contaminated RC slabs has been investigated in this study. Similarly, the extent to which discrete sacrificial anodes can be used to extend the service life of corroding chloride contaminated RC structures has been assessed through an extensive review of literature as well as laboratory experiments on corroding chloride contaminated RC slabs. This study evaluates the performance of discrete SACP systems with respect to binder type, corrosion rate and the level of chloride contamination with an objective of developing guidelines for rational concrete repair when using SACP systems to repair RC structures that are deteriorating from corrosion of reinforcing steel in South Africa.

Eight slabs were investigated. Four slabs were cast using 100% PC CEM 1, 42.5N while the other four were cast using a blend of 70/30 PC CEM 1, 42.5N/FA. Each slab comprised two portions: one half of each slab was cast using concrete that was admixed with 0.6% chloride by mass of binder while the other half was cast using concrete that was admixed with 1.8% chloride by mass of binder. The corrosion in the slabs was induced and accelerated using direct anodic current. The SACP system that was used in this study comprised discrete zinc disks put in a cylinder of a proprietary high alkaline mortar. These anodes were installed in 100 x 100 x 60 mm deep cavities that had been made in concrete. The anodes were connected to the reinforcement cage using tie wires and thereafter backfilled with a repair mortar. The cavities in which the anodes were installed were arranged in a square grid of 450 x 450 mm. The corrosion rate and half-cell potential of the steel as well as the resistivity of concrete in the slabs were monitored over a duration of five months.

The discrete SACP system that has been investigated in this study can increase the service life of chloride contaminated RC structures that are deteriorating as a result of rebar corrosion. The discrete anodes reduced the corrosion rate of steel in the test slabs. The percentage reduction in the average corrosion rate was higher within the slabs that were cast using 100% PC CEM 1, 42.5N than in the slabs that were cast using a blend of 70/30 PC CEM 1, 42.5N/FA. The reduction in average corrosion rate within the slabs cast using 100% PC CEM 1, 42.5N ranged between 45-95%. Similarly, the reduction in the average corrosion rate of within the slabs that were cast using a blend of 70/30 PC CEM, 42.5N 1/FA ranged

between 54-75%. Throughout the test specimens, the portions of the slabs that was admixed with 0.6% chloride by mass of binder exhibited superior performance. These portions (admixed with 0.6% chloride by mass of binder) experienced the greatest reduction in average corrosion rate as well as the greatest shift in average half-cell potential towards values that are more negative. Finally, in order to achieve the long-term objective of increasing the service life of RC structures that are deteriorating as a result of rebar corrosion, long-term studies ought to be undertaken to ascertain, with surety, the effectiveness of discrete SACP systems in corroding structures.

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List of Symbols and Abbreviations

ACI	American Concrete Institute
ASR	Alkali Silica Reaction
C&CI	Cement and Concrete Institute
Avg.	Average
CBD	Compacted Bulk Density
CCI	Chloride Conductivity Index
CP	Cathodic Protection
CPrev	Cathodic Prevention
CSE	Copper Sulphate Electrode
DC	Direct current
DI	Durability Index
ECE	Electrochemical Chloride Extraction
ECR	Epoxy Coated Reinforcement
emf	Electromotive force
ER	Electrochemical Realkalisation
FA	Fly Ash
FHWA	Federal Highway Administration (USA)
FM	Fineness Modulus
GGBS	Ground Granulated Blastfurnace Slag
GNP	Gross National Product
HCP	Half-cell potential
ICCP	Impressed Current Cathodic Protection
OPC	Ordinary Portland Cement
RC	Reinforced Concrete
RE	Reference Electrode
RH	Relative Humidity
SABS	South Africa Bureau of Standards
SACP	Sacrificial Anode Cathodic Protection
SANS	South African National Standards

SCE	Saturated Calomel Electrode
SCM	Supplementary Cementitious Material
SF	Silica Fume
SHE	Saturated Hydrogen Electrode
USDoT	United States Department of Transportation
w/b	water-to-binder ratio

$\mu A/cm^2$	Microamperes per square centimetre
Ag/AgCl	Silver/Silver Chloride
C_3A	Tricalcium Aluminate
$Ca(OH)_2$	Calcium Hydroxide
$CuSO_4$	Copper Sulphate
E_{corr}	Corrosion potential
Fe	Iron
Fig.	Figure
I_{corr}	Corrosion current density
Na_3BO_3	Sodium Borate
NaCl	Sodium Chloride
NaOH	Sodium Hydroxide
O_2	Oxygen
OH^-	Hydroxyl ion
ppm	Parts per million
R_p	Polarisation resistance

Quote

'Concrete repair engineering is not just doing theoretical sums, nor is it based on adherence to formulas, graphs and exact code provisions. The solutions are usually open ended, often approximate solutions to exact problems. The problems necessitating the repair of the structure have to be accurately investigated and established. Then several alternative solutions depending on the project objectives can be offered. It is extremely important to find out what are the 'exact' problems. An approximate solution to an exact problem is more meaningful than the exact solution to an approximate problem'.

(Vaysburd *et al*., 2009)

CHAPTER 1: INTRODUCTION

1.1. General introduction

Concrete is a versatile, economical and successful material that has been used for construction worldwide. From its humble beginnings as a construction material with limited applications in ancient civilisations of the Greeks and the Romans, concrete has proven to be an inherently durable material when used properly (Schiessl, 1996; Costa and Appleton, 2002). Coupled with its low cost of production, the inherent durability of concrete has enabled it become the most preferred construction material. Consequently, concrete has been used as the material of choice in most modern-day buildings, bridges, dams, pavements and other components of infrastructure worldwide.

The invigorated research on materials and construction techniques, industrialisation and technological advancements in the early nineteenth century witnessed the emergence of new and efficient construction materials. With this research also came the need to re-engineer the properties of existing materials for optimal performance. Thus, this epoch witnessed the continuous development of concrete as a structural engineering material through the modification of its properties. Notable of all these developments was the invention to overcome the low tensile strength of concrete through the use of reinforcing steel. The combination of the high compressive strength of concrete and the high tensile properties of reinforcing steel produces a composite material which, compared to other construction materials, offers a wide range of applications in structural engineering (Arora *et al*., 1997; Berke, 2006; Andrade, 2009). Steel RC has become one of the most durable and widely used materials of construction.

In spite of their 'perceived' durability, RC structures have continued to deteriorate from corrosion of reinforcement. The first major corrosion-induced damages on RC structures have been documented since the 1960s (Funahashi and Young, 1998; Raupach, 2009); and since about 1975, the amount of damages that can be attributed to rebar corrosion has been increasing considerably. Currently, corrosion of steel in RC structures presents a lot of challenges towards the maintenance of infrastructure globally (Zhang, 1995; Maage *et al.*, 1996; Beamish and El-Belbol, 1998; Liang *et al.*, 1999; González *et al.*, 2004; Fang *et al.*, 2006b). More specifically, El-Reedy (2008) reports that 80% of the deterioration of RC structures is due to corrosion of reinforcing steel. Thus, it is imperative that reinforcement corrosion contributes, to a large extent, to the reduced service life of RC structures.

Ideally, reinforcing steel embedded in concrete does not corrode under normal environmental conditions due to two factors. First, a thin protective film of γFe_2O_3 (gamma iron oxide) forms on the surface of steel (Hausmann, 1964). This condition is known as passivation. Second, a sufficient volume of low permeability concrete between the steel and the atmosphere reduces the ingress of oxygen to the steel (Lees, 1992). This concrete prevents corrosion. Passivation of steel results from chemical reactions that take place between steel and the high alkalinity (pH = 12 or higher) that results from hydration of

cement in concrete (Concrete Society, 1989; Bertolini *et al*., 2004a; Ballim *et al*., 2009; Gjørv, 2009). These reactions take place at the surface of the steel; consequently leading to the formation of a layer known as the passive film – which is partly composed of metal oxide and minerals from cement. The passive film is a dense impenetrable film that, if fully established and maintained, prevents corrosion of reinforcing steel. This is due to the fact that it suppresses the rate of dissolution of steel to negligibly low values (Hunkeler, 2005; Broomfield, 2007; El-Reedy, 2008). However, the ideal scenario is hardly realised in RC structures because concrete, whatever its composition, is an inherently porous medium that transmits oxygen, moisture and aggressive electro-active species like chloride ions (Zhang, 1995; Arora *et al.*, 1997; Vaysburd and Emmons, 2000; Vaysburd and Emmons, 2004). These species destroy the passive film formed on the embedded steel thus resulting in a phenomenon known as depassivation. Corrosion of reinforcement sets in when this passive film is destroyed by stray currents, the ingress of chloride ions, or when the alkalinity of concrete is reduced as a result of carbonation (Bertolini *et al.*, 2004a; Bungey *et al*., 2006).

The rehabilitation of structures that are deteriorating as a result of corrosion of reinforcement involves high financial costs and a lot of technical effort (Hunkeler, 2005). The costs associated with corrosion constitute a considerable portion of the budget of many countries. Corrosion and its associated costs have been reported to amount to approximately 3–4% of the annual Gross National Product (GNP) in many countries (Mackechnie and Alexander, 2001; Bardal, 2004; Revie and Uhlig, 2008; Yang, 2008). While researchers such as Miranda *et al* (2006) reported that 18% of the costs of corrosion are spent on the construction industry, Bardal (2004) suggests that 20% of these costs can be avoided through the better use of existing knowledge in corrosion protection and design. Thus, it can be inferred that corrosion is an economic problem that requires an engineering solution.

The economic impact of corrosion, amongst other factors, has motivated researchers to devise techniques that can control corrosion of steel in RC structures effectively. Conventional patch repairs, chemical additives, special cements, concrete surface treatment, coatings and corrosion inhibitors are some of the methods used to control corrosion in RC structures (Al-Idi and Al-Mehthel, 1995; Rostam, 2003). However, some of these techniques have been reported to be ineffective with respect to the control of corrosion in structures that are contaminated by chlorides (Hong *et al*., 1993; Brousseau and Pye, 1997; Polder, 2005; Sergi, 2010).

Cathodic protection (CP) is a technique that has been demonstrated to be successful in providing cost effective long-term corrosion control for steel in RC structures (Weyers and Cady, 1984; BS EN 12696:2000; Chaudhary, 2002; Cramer *et al*., 2002; Young *et al*., 2009; Redaelli and Bertolini, 2011b). CP has also been reported as a technique that is effective and reliable with respect to the control of corrosion in structures that contain a high concentration of chlorides (Berkeley, 1992; Scannell and Sohanghpurwala, 1993; Ishii *et al.*, 1998; Smith and Virmani, 2000; Bertolini et al., 2004b). When used in isolation or in combination with other corrosion control techniques, CP minimises the negative structural effects that result

from chloride-induced corrosion damage and prevents possibilities of future corrosion damage (Covino *et al*., 2002).

CP systems are divided into two broad categories, namely: impressed current cathodic protection (ICCP) systems and sacrificial (or galvanic) anode cathodic protection (SACP) systems. These systems rely on the principle of applying a voltage that forces electrons to flow to the reinforcing steel. The flow of electrons makes the steel a cathode, hence the name cathodic protection. Details on how this voltage controls corrosion of reinforcing steel in RC structures will be discussed in the subsequent sections and chapters. ICCP systems rely on the application of an external source of current (or rectifier) to drive electrons through the steel. They are the most commonly used CP systems. SACP systems, however, rely on the principle of the galvanic current that is generated from coupling steel to metals that are higher than it in the electrochemical series (Whiting *et al*., 1996; Whitmore and Ball, 2005; Ball and Whitmore, 2009; Glass *et al*., 2010). Details on how SACP systems protect steel in RC structures will be elucidated in the subsequent sections and chapters as this is what this study entails.

1.2. Background of the study

The concrete repair industry in South Africa relies mainly on the application of common patch repairs and corrosion inhibitors to control corrosion of steel in RC structures. However, some of these techniques have been found to be ineffective, in the long-run, especially when used to repair RC structures that are heavily contaminated with chlorides (that is, chloride contents > 3 kg/m^3) (ACI, 2005; Daily, 2003). Techniques, such as patch repairs, have been found to work for a limited duration after which further repairs will be required (Sagüés and Powers, 1994; Orellan *et al*., 2004; Vaysburd and Emmons, 2004; Qian *et al*., 2006; Vaysburd, 2006; Sergi, 2010). Thus, several concerns have been raised with respect to the effectiveness of some of these commonly used repair techniques.

It has been observed, in South Africa (Crosswell, 2007) and internationally, that there is a progressive increase in the number of RC structures that are deteriorating as a result of corrosion of reinforcing steel. This increase, however, is inconsistent with the amount of human effort and financial resources that have been expended towards the control of corrosion (Scannell and Sohanghpurwala, 1993). The number of RC structures undergoing deterioration is on the rise. The increase in the number of RC structures that are deteriorating as a result of rebar corrosion as well as their associated costs of repair has seen both the government and building investors attract marginal returns, if any. Likewise, motorists and the public at large have continued to experience inconveniences caused by delays and disruption of traffic due to closure of roads as a result of repairs on bridges and corroding transport infrastructure. Thus, there is a need to address and minimise the negative impacts of corrosion. Similarly, there is an urgent need to reconcile the mismatch between the ever-increasing number of deteriorating RC structures on one hand and the amount of human and financial capital being expended while repairing structures that are deteriorating as a result of rebar corrosion on the other hand. These needs can be addressed through the development of

concrete repair techniques that are effective, economical and technically sound, and 'easy-to-execute.' The sacrificial anode cathodic protection (SACP) system is one such technique.

The SACP is an established corrosion control method that has been in use since 1824. It has been used successfully to protect marine vessels, buried pipelines and steel structures from corrosion. However, its use on RC structures is a relatively new concept. This is due to the fact that its use in RC structures began on a large scale in the late 1990s. Nevertheless, SACP systems have been used in RC structures to: control pitting corrosion in concrete structures exposed to seawater (Bertolini *et al*., 2002), protect bridge piles (de Rincón *et al*., 1997) and aerial structures from corroding, and in patch repairs (Wang *et al*., 2006).

The SACP technique is based on the sacrificial dissolution of metals that are higher than steel in the electrochemical series (See Appendix A and Table A.1 for the electrochemical series of metals). These metals include, but are not limited to, magnesium, zinc, and zinc-aluminium alloys. Common SACP systems consist of anodes that are applied onto the concrete surface by flame spraying, as a mesh or sheet adhering through a conductive gel, or any other convenient form, after which they are electrically and/or electrochemically connected with the reinforcing steel. This connection generates a potential difference which forces current to flow within the RC structure. The flow of current polarises steel to a negative potential (Whiting *et al*., 1996; Daily, 2003); therefore resulting in either a reduction or a complete halt in the rate at which steel is corroding. A schematic diagram of a typical SACP system is shown in Figure 1.1.

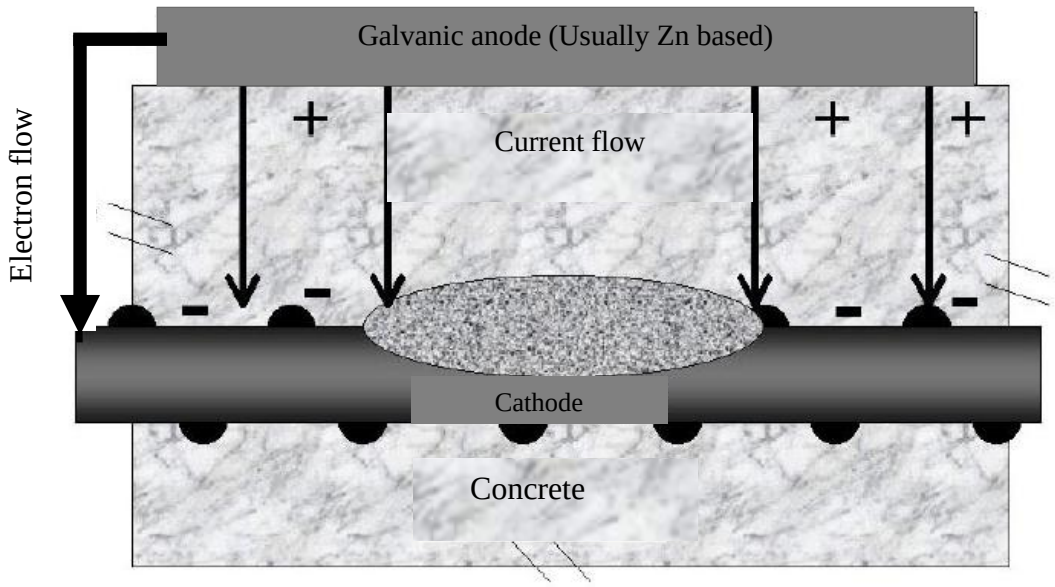


Figure 1.1: Schematic diagram of a sacrificial anode (galvanic) cathodic protection system (adapted from the Highways Agency, 2002)

The SACP system is one of the most effective techniques used to control corrosion of steel in RC structures. The Florida Department of Transportation in the USA, for example, through a study on the long-term effectiveness of cathodic protection systems on highway structures (USDOT, 2001) concluded that "*the only cost-effective method to protect marine*

substructures is galvanic cathodic protection." In spite its successful track record in controlling corrosion in RC structures, SACP systems are rarely used in South Africa. In addition, the extent and criteria on which the effectiveness of sacrificial anodes can be evaluated clearly remains an issue that has continued to generate some controversy. The criteria upon which the effectiveness of SACP systems can be evaluated as a concrete repair technique in South Africa have not been established.

1.3. Problem statement

The successful use of SACP systems in stopping the initiation of corrosion or reducing the corrosion rate of steel in deteriorating RC structures has been reported by researchers as well as concrete repair experts and agencies (Bennett and Turk, 1994; Sohanghpurwala, 2009). Similarly, extensive scientific research has been carried out to study and ascertain the principles, mechanisms and conditions upon which SACP systems work optimally. In spite of the extensive research that has been undertaken, limited research, if any, has been carried out with respect to the assessment of the effectiveness of these systems on corroding chloride contaminated RC structures.

Most of the studies that have been conducted on SACP systems are limited to new RC specimens (Sekar *et al*., 2007) as well as specimens made using 100% CEM 1. Consequently, some of the conclusions arrived at, from these studies, are far from reality as well as impractical. This is due to the fact that they have failed to incorporate the various binder types and blends that are available in the market as well as the varying corrosion rates and exposure environments that real life structures are subjected to. Thus, most of the information that has been generated from existing research is not substantial enough to warrant and develop a rational repair strategy that incorporates the use of SACP systems to protect RC structures that are corroding in South Africa.

To date, there is no documentary evidence of the use of SACP systems within South Africa despite their reported effectiveness on corrosion control. Similarly, a prescribed and 'clear-cut' methodology with respect to methods and criteria that can be used to assess the effectiveness of SACP systems in chloride contaminated RC structures that are corroding within the South African exposure environment does not exist. These technical handicaps have been aggravated further by the fact that the current code of practice used for cathodic protection, BS EN 12696:2000, has failed to offer guidance on the use of SACP systems in concrete repair. The lack of guidance has contributed, directly and indirectly, to the limited applicability of SACP systems in the repair of RC structures that are deteriorating from rebar corrosion in South Africa. Thus, SACP systems have been visualised as a 'black box' - that is, the design, installation, operation and maintenance of SACP systems has been left in the hands of a few skilled personnel. This phenomenon has consequently forced most South African engineers to specify certain repair methods which are either ineffective or less effective in lieu of SACP systems.

1.4. Objectives of the study

The principal objective of this study is to evaluate the performance of a locally available discrete SACP system with respect to binder type, corrosion rate and level of chloride contamination in reinforced concrete structures.

1.5. Significance of the study

This study intends to:

i. Provide a platform on which the effectiveness of SACP systems can be evaluated as a plausible concrete repair technique in South Africa

ii. Assist in developing a rational repair strategy that incorporates the use of SACP systems in existing RC structures that are deteriorating as a result of corrosion of reinforcing steel

1.6. Scope of the study

This study is limited to:

i. Chloride contaminated slabs reinforced using conventional/ordinary reinforcing steel

ii. RC slabs cast using Portland cement (PC CEM 1, 42.5N) as well as a blend of PC CEM 1, 42.5N and Fly Ash (FA)

iii. Laboratory cast slabs exposed to an outdoor environment in Cape Town, South Africa

iv. A commercially available discrete sacrificial anode (Sika Galvashield CC65)

1.7. Layout of the thesis

This thesis is presented in six chapters as follows:

i. The first chapter is an introduction to the study. It also gives the background to the research problem; specifies the scope of the research as well as its objectives. The significance of the study is also presented.

ii. The second and third chapters provide a critical and an in-depth review of published works, theses and journal papers. An in-depth and detailed background to the corrosion phenomenon is presented in the second chapter. Likewise, the mechanisms with which corrosion propagates and the techniques that are used to control corrosion have presented in the same chapter. The third chapter contains a critical review of literature regarding SACP systems.

iii. The fourth chapter contains the experimental methodology. All parameters that were investigated in this study, the materials and equipment used as well as the type of slabs tested are contained here.

iv. The fifth chapter contains the results that were gathered during this study. The quantitative as well as qualitative analysis of experimental data is included in this chapter. The discussions to the results that were obtained and analysed during this study have also been included within this chapter.

v. The sixth chapter contains the conclusions that have been arrived at based on all the data that was collected and analysed. This chapter also contains a list of recommendations.

CHAPTER 2: CORROSION OF STEEL IN RC STRUCTURES

Corrosion refers to a chemical or electrochemical reaction that takes place between a material, usually a metal, and its environment resulting in deterioration of the material and its properties (ASM, 2000; Bardal, 2004; Cragnolino, 2008; Revie and Uhlig, 2008). Corrosion of steel in RC structures is the principal cause of deterioration of structural concrete (Parthiban *et al*., 2008a; Zhang *et al*., 2009a; Zhang *et al*., 2010).

2.1. The Pourbaix diagram of steel

The corrosion of steel in concrete is governed by two main parameters: the potential and pH of the environment in which the reinforcement is placed (Capozucca, 1995). A pourbaix diagram is a compact summary of thermodynamic data, in the form of a potential-pH diagram, which relate the electrochemical and corrosion behaviour of any metal in water (Revie and Uhlig, 2008; Hussain and Ishida, 2009). These diagrams have the advantage of showing, at a glance, specific conditions of potential and pH under which a metal does not react (immunity), or can react to form oxides or complex ions. Depending on the pH and potential of steel, a pourbaix diagram can be used to evaluate and determine the state of the passive layer.

From a corrosion engineering perspective, pourbaix diagrams are useful in identifying the potential-pH domains where corrosion does not occur – i.e., where the metal itself is the stable phase (Hussain and Ishida, 2009). Thus, by controlling the potential (e.g., by cathodic protection) and/or by adjusting the pH in specific domains identified using pourbaix diagrams, it may be possible to prevent corrosion from taking place. The Pourbaix diagram for iron is represented in Figure 2.1. A horizontal line in this diagram represents a reaction that does not involve pH; i.e., neither H^+ (hydrogen ions) nor OH^- (hydroxyl ions) is involved. This reaction is shown in Equation 2.1.

$$Fe^{2+} + 2e^- \rightarrow Fe \quad (2.1)$$

A vertical line involves H^+ (hydrogen ions) or OH^- (hydroxyl ions) but not electrons. An example of this reaction is shown in Equation 2.2. In Figure 2.1 the vertical line separating Fe^{3+} from Fe_2O_3 corresponds to this reaction.

$$2Fe^{3+} + 3H_2O \rightarrow Fe_2O_3 + 6H^+ \quad (2.2)$$

In the Pourbaix diagram for iron, Figure 2.1, the vertical line at pH 1.76 represents the equilibrium reaction, $2Fe^{3+} + 3H_2O \rightarrow Fe_2O_3 + 6H^+$. To the right of this line (i.e., at pH > 1.76), Fe_2O_3 is the stable phase; and this oxide, as a protective film, would be expected to provide some protection against corrosion. To the left of this line (i.e., at pH < 1.76), ferric ions in solution are stable, and corrosion is expected to take place without any protection afforded by a surface oxide film (Revie and Uhlig, 2008). A sloping line involves H^+ (hydrogen ions), OH^- (hydroxyl ions) and electrons. For example, the sloping line separating Fe^{2+} from Fe_2O_3 represents the reaction shown in Equation 2.3.

$$Fe_2O_3 + 6H^+ + 2e^- \rightarrow 2Fe^{2+} + 3H_2O \quad (2.3)$$

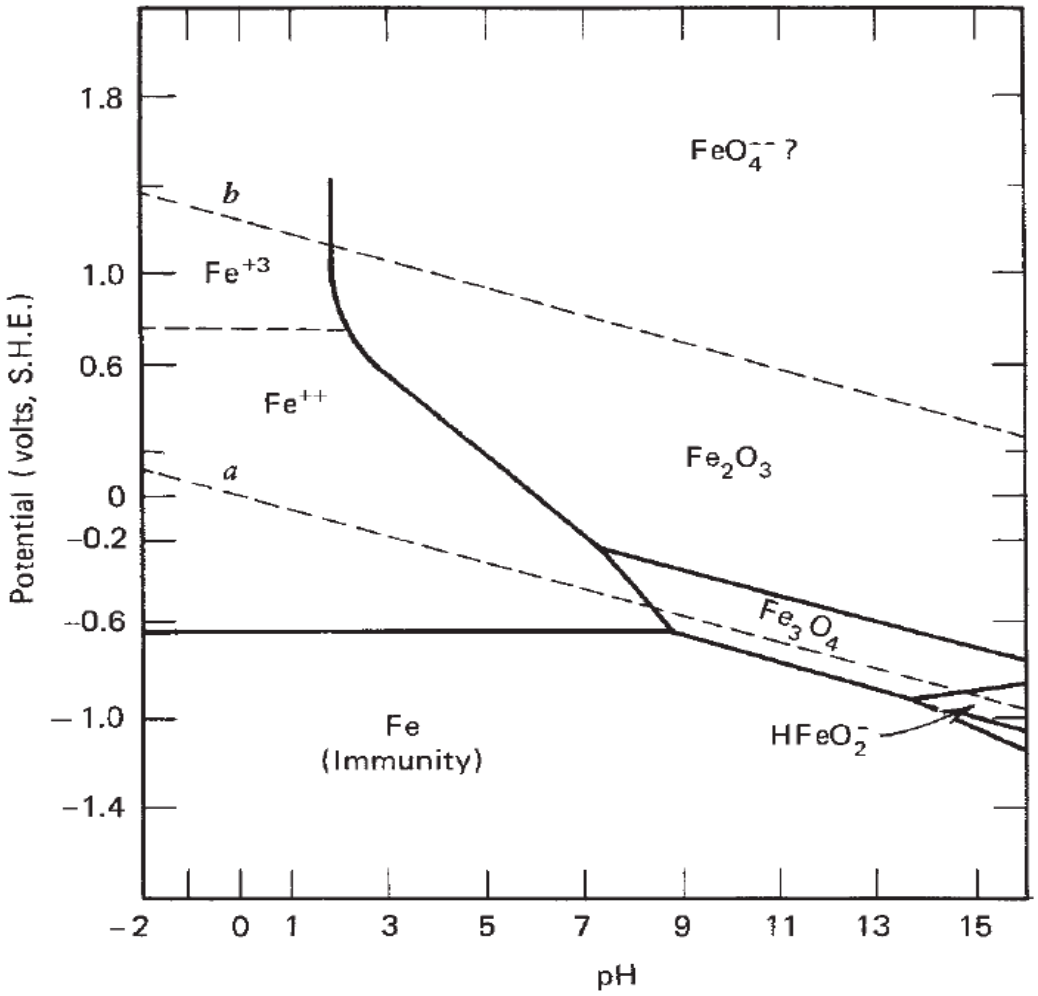


Figure 2.1: Pourbaix diagram for the iron-water system at 25°C (Revie and Uhlig, 2008).

The pH values in Pourbaix diagrams are those of a solution in immediate contact with the metal surface. This value in some instances (e.g., Fe in aerated H_2O) differs from the of the bulk solution. Soluble hypoferrites ($HFeO_2^-$) can form in very alkaline solutions within a restricted active potential range. Soluble ferrates (FeO_4^{2-}) can form in alkaline solutions at very noble potentials, but the stable field is not well defined. The fields marked Fe_2O_3 and Fe_3O_4 are sometimes labelled "passivation" on the assumption that iron reacts in these regions to form protective oxide films. This is correct only insofar as passivity is accounted for by a diffusion-barrier oxide layer (Revie and Uhlig, 2008).

2.2. Causes of rebar corrosion in RC structures

For corrosion to occur, it is necessary that both the passivating film on the steel is destroyed and that there exists a differential electrochemical potential within the steel-concrete system (Vaysburd and Emmons, 2004). The destruction of the passive film is mainly caused by acidification or de-alkalisation. Acidification or de-alkalisation can take the form of carbonation and/or the intrusion of chlorides (Concrete Society, 1989; Isecke, 1997; Broomfield, 2007; Roberge, 2008). Furthermore, stray currents have been reported to cause

corrosion in special circumstances (Berkeley, 1992; Chess, 1998a; Bertolini *et al*., 2004a; Bungey, 2006; Bertolini *et al*., 2007).

There are three main parameters that are necessary for the initiation of corrosion of steel in RC structures. They include: the moisture content and resistivity of concrete, the availability of oxygen and the presence of aggressive species (chloride ions or carbon dioxide from the atmosphere) (Ahmad, 2003; Hunkeler, 2005; Bungey *et al*., 2006). The interaction between these parameters during the process of corrosion is shown in Figure 2.2. The rate at which corrosion takes place is also affected by other factors such as temperature, cement and aggregate type and additives (Chess, 1998a; Pour-Ghaz *et al*., 2009).

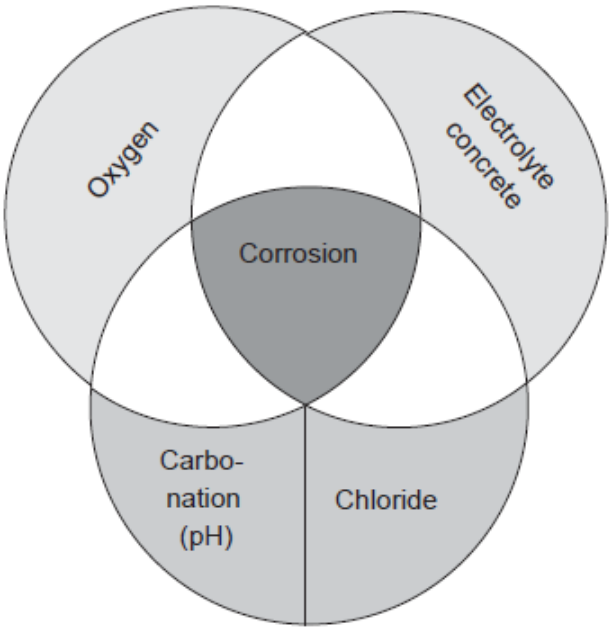


Figure 2.2: Conditions necessary for corrosion of steel in concrete (Hunkeler, 2005)

Carbonation refers to the chemical reaction that takes place between atmospheric carbon dioxide and the alkaline hydroxides (e.g. Calcium hydroxide) that exist in concrete (Bertolini *et al*., 2004a; Redaelli and Bertolini, 2011a). Carbonation-induced corrosion of reinforcing steel is mainly caused by insufficient cover to concrete or insufficient compaction. The process of carbonation begins when carbon dioxide (CO_2) diffuses into concrete and dissolves thereafter to form a weak acid known as carbonic acid. This acid subsequently dissociates into carbonate and hydrogen ions and reacts with the hydroxide compounds to form less soluble carbonates such as calcium carbonate ($CaCO_3$). Carbonation reactions decrease the pH of the pore solution from pH = 13.6 to values close to, or lower than 9 (Bertolini *et al*., 2003; Osterminski and Schießl, 2009; Trejo *et al*., 2009; Nasser *et al*., 2010). The reduction in pH makes the passive film unstable (Concrete Society, 1989; Ballim *et al*., 2009); therefore increasing the susceptibility of the reinforcement to corrosion. The relationship between the amount of rust produced (the rate at which the passive film breaks down) and pH is shown in Figure 2.3.

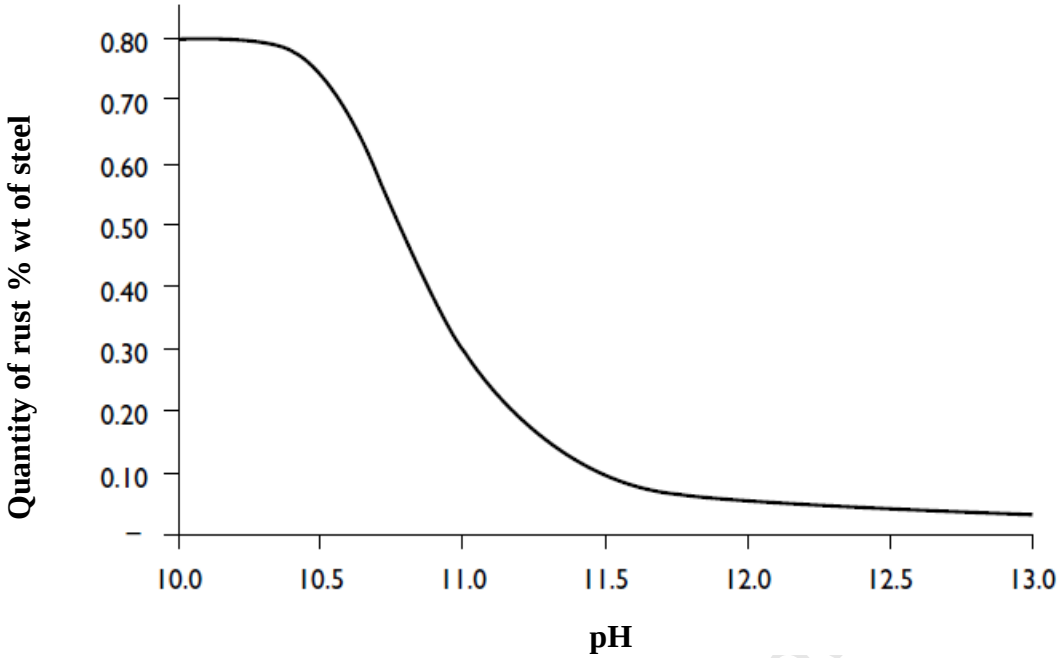


Figure 2.3: Corrosion of steel in aqueous solution as a function of pH (adapted from Broomfield, 2007)

The process of carbonation begins from the outermost layers of concrete. Thereafter, it progresses into the deeper layers of concrete. The rate at which this process progresses depends on concrete properties (mainly its 'permeability') and exposure conditions (mainly the humidity content in the concrete) (Redaelli and Bertolini, 2011a). Hussain and Ishida (2009) reported that carbonation-induced corrosion starts when the carbonation front reaches a critical depth of 80% of the total depth of the concrete cover. The carbonation process is very aggressive when chloride ions are present (Bertolini *et al.*, 2003; Assouli *et al*., 2005); however, in the absence of initial chloride in concrete, the pore solution following carbonation is composed of almost pure water. Thus, steel that has been embedded in humid carbonated concrete corrodes as if it was in contact with water. Another consequence of carbonation is that chlorides bound in the form of calcium chloroaluminate hydrates and otherwise bound to hydrated phases may be liberated. The liberation of bound chlorides may make the pore solution more aggressive (Lees, 1992; Ihekwaba *et al*., 1996; Bertolini *et al*., 2004a; Ballim *et al*., 2009). Carbonation takes place most rapidly in conditions of intermediate humidity (50-70% relative humidity) (Banfill, 1997). Carbonation damage is particularly apparent in low-grade concrete structures where builders were economical with cement and liberal with water (Chess, 1998a). However, Beamish and El-Belbol (1998) suggest that carbonation damage can be limited to insignificant levels through the specification of an appropriate depth of good quality concrete cover as well as good workmanship.

Chloride-induced corrosion is one of the main causes of corrosion of reinforcement in today's infrastructure (Elsener, 2001; Arya *et al*., 1996; Liu and Weyers, 1998; Bertolini *et al.*, 2004a; Poursaee and Hansson, 2009). The source of chlorides may be internal and/or

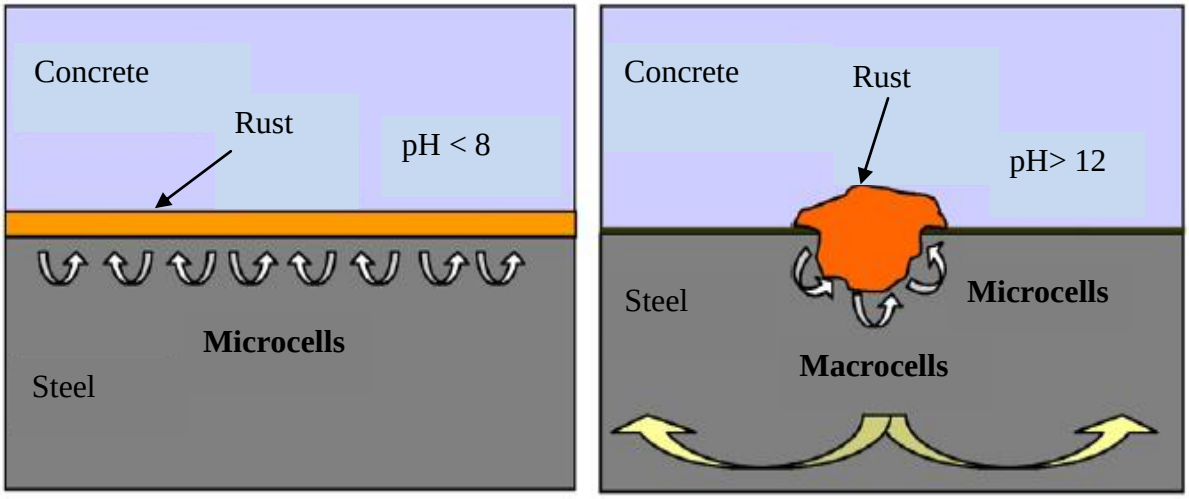
external. External sources of chlorides include chloride ions that diffuse into concrete from de-icing salts on bridges and seawater or sea salt spray on maritime structures. Internal sources mainly comprise salt-contaminated aggregates. In exceptional cases, critical amounts of chlorides may penetrate into fresh concrete mix as a 'set accelerator' (Calcium chloride), in the mixing water, through PVC fires (Raupach, 1996; Bertolini *et al*., 2004a; Melchers and Li, 2009), or through the exposure to industrial brine (Yeih *et al*., 2006). Chloride has a fourfold effect in RC structures, namely: it destroys the passive film thus making corrosion attack possible, it reduces the pH of the pore water due to the fact that it reduces the solubility of calcium hydroxide ($Ca(OH)_2$), it increases the moisture content of concrete because of the hygroscopic properties of salts present in concrete (e.g. $CaCl_2$, NaCl) and, it increases the electrical conductivity of concrete (Yalçyn and Ergun, 1996; Covino *et al*., 2002; Hunkeler, 2005).

An increase in the concentration of chloride near the steel reinforcement corresponds to an increase in corrosion risk (Hassanein *et al*., 1998; Smith and Virmani, 2000). Thus, chloride-induced corrosion is initiated once a critical chloride threshold is reached at the level of the reinforcing steel. The chloride threshold level is defined as the content of chloride at the steel depth that is necessary to sustain local passive film breakdown and hence initiate the corrosion process (Ann and Song, 2007; Trejo *et al*., 2009). This threshold, however, is not unique (Poupard *et al*., 2004; Angst and Vennesland, 2009; Pradhan and Bhattacharjee, 2011) and is known to depend on factors related to concrete and the environment (Taylor *et al*., 1999; Alonso *et al*., 2000; Sharp *et al*., 2002; Assouli *et al*., 2005; Sekar *et al*., 2007). More specifically, the chloride threshold of concrete is affected by factors such as: the composition, quality and pH of concrete, the potential of steel, cover depth, tricalcium aluminate (C_3A) content of the cement, saturation level of the concrete, the concentration of oxygen, the presence of microvoids in the cement paste and atmospheric conditions (Alonso *et al*., 2000; Smith and Virmani, 2000; Andrade, 2009; Angst and Vennesland, 2009; Bertolini and Redaeli, 2009; Beushausen and Alexander, 2009; Trejo *et al*., 2009). Previous research has established that the chloride threshold increases when the potential of steel is lowered (de Rincón *et al*., 1997; Sekar *et al*., 2007). For this reason, corrosion hardly initiates in parts of RC structures that are permanently immersed in sea water, since lack of oxygen leads to very negative values of potential.

Finally, it is important to note that once the passive layer on the reinforcing steel has been disrupted and corrosion is activated, the chemical reactions are similar whether the corrosion was initiated by chloride attack or carbonation (Mackechnie and Alexander, 2001).

2.3. Corrosion cells and forms of corrosion

Corrosion cells can occur as microcells - which lead to uniform iron removal; or macrocells - which cause local iron removal or pitting corrosion (Raupach, 1996; Elsener, 2002; Berke, 2006; Subramaniam and Bi, 2010). A schematic representation of these cells is shown in Figure 2.4.



a) Microcells b) Macrocells

Figure 2.4: Schematic representation of corrosion cells in concrete (adapted from Zhang *et al*., 2009b)

Corrosion macrocells result from the potential difference that exists between sections of steel that are in electrical (steel) and electrolytical (concrete) contact (Gulikers, 1996). Microcell corrosion, however, refers to the situation where both the active dissolution of the metal and its corresponding half-cell reaction (the reduction of dissolved oxygen) take place at adjacent parts of the same metal part (Hansson *et al*., 2006). Macrocell corrosion is the dominant corrosion process in structural members that have multiple layers of rebars. It can occur when an actively corroding rebar is coupled to a rebar that is passive or has a lower corrosion rate. They can also form on a single bar exposed to different environments within the concrete or when part of the bar extends outside the concrete.

Reinforcement corrosion of steel in concrete manifests itself in two main ways, namely: general (uniform or homogeneous) corrosion and localised (or pitting) corrosion (Schwenk, 1997; Hunkeler, 2005; Cragnolino, 2008). On one hand, general corrosion occurs continuously over substantial areas of steel. It is characterised by the presence of corrosion microcells which are microscopic in size, so that externally they appear to result in the uniform removal of steel (Raupach, 1996; Hassan *et al*., 2010). General corrosion is primarily caused by carbonation (Capozucca, 1995; Nasser *et al*., 2010). However, González *et al* (1995) and Bertolini *et al* (2004a) reported that general corrosion can be caused when steel is in the vicinity of a high concentration of chloride ions. On the other hand, localised corrosion results from chloride attack. This form of corrosion is characterised by the presence of macrocell corrosion. Furthermore, localised corrosion consists of anodically acting areas, where the critical chloride content has been reached, and large cathodes being next to the anodes or sometimes far away from the anode (Raupach, 1996; Raupach, 2006; Cragnolino, 2008; Beushausen and Alexander, 2009; Subramaniam and Bi, 2010). Therefore, localised corrosion is characterised by the continuous occurrence of uncorroded and pitted reinforcement (Arora *et al*., 1997; Val *et al*., 1998).

2.4. Electrochemical reactions and processes during corrosion of steel in concrete

An electrochemical reaction is defined as a chemical reaction that involves the transfer of electrons within an electrolyte (Roberge, 2008). These reactions involve oxidation and reduction. Oxidation refers to the loss of electron(s) by an atom while reduction refers to the gain of electron(s) by an atom(s). Essentially, oxidation and reduction reactions take place at electrodes. Electrodes refer to points where current (or electrons) enter or leave an electrolyte. The electrode where chemical reduction takes places is called the cathode while the one where chemical oxidation occurs is called the anode (Gu *et al*., 2001; Papavinasam, 2008).

Corrosion of steel in concrete is an electrochemical process (Taylor *et al*., 1999; Pradhan and Bhattacharjee, 2009; Zhang *et al*., 2009b; Wong *et al*., 2010). It comprises an anode, a cathode, an electrolyte, and an electrical connection between the anode and the cathode. This process has been shown in Figure 2.5. The electrical connection helps in ensuring that electrons are transferred within the concrete. Coupled anodic and cathodic reactions take place on the surface of the reinforcing steel. Concrete pore water acts as the electrolyte, and the body of reinforcement can either function as an anode or as a cathode depending on the direction of current flow (Hausmann, 1964; Gu *et al*., 2001 Trejo *et al*., 2009). Reinforcing steel provides the electrical connection between the anode and the cathode. Cathodes and anodes may be located on the same rebar (microcell) or on different rebars (macrocell) that are electrically connected through metallic ties or chairs (Trejo *et al*., 2009).

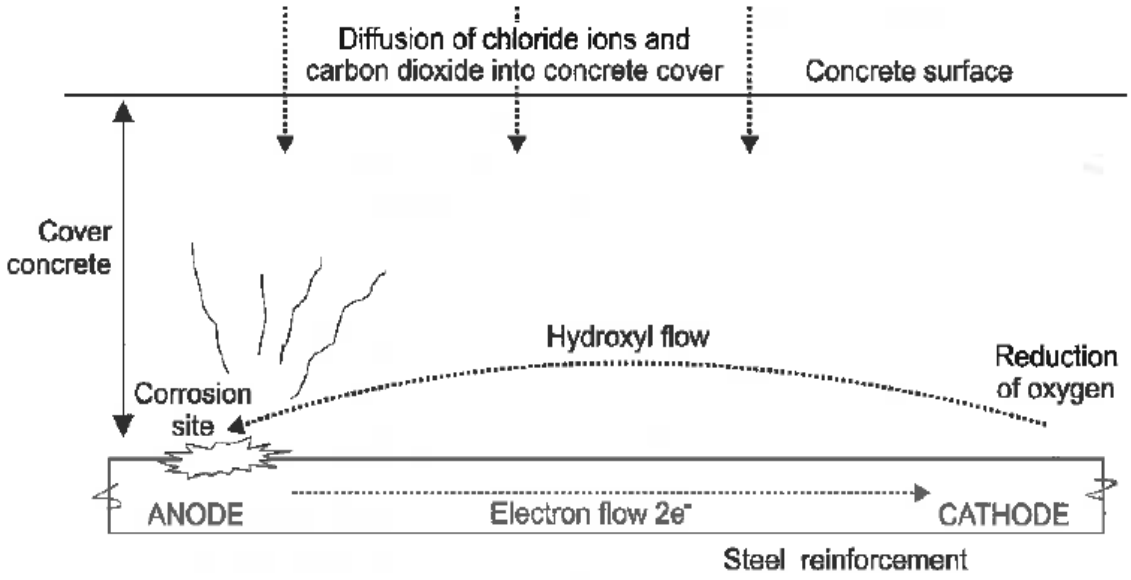


Figure 2.5: Schematic of corrosion of steel in concrete (Mackechnie and Alexander, 2001)

The electrochemical processes that take place during corrosion develop through an electrochemical potential difference that develops from the dissimilarities in the chemical environment of steel. The dissimilarities in the chemical environment include non-uniform carbonation, the variation of penetration of chlorides, moisture, oxygen, etc (Vaysburd and Emmons, 2000). Corrosion is mainly influenced by the electrical resistivity of concrete and

the steel, the alkalinity of the cement composite in contact with steel, the presence of electrolyte in the concrete and the concentration of oxygen dissolved in the pore liquid near the reinforcing steel (Balabanić *et al*., 1996). The oxidation of iron (Fe) to iron ions (Fe^{2+}) takes place at the anode while the reduction of oxygen (O_2) to hydroxyl ions (OH^-) takes place at the cathode. Anodes and cathodes form a short-circuited corrosion cell whereas the flow of electrons in the steel and of ions in the pore solution takes place within the concrete (Koleva *et al*., 2006).

There are two main reactions that are normally associated with corrosion. They are: the anodic reaction and the cathodic reaction. These reactions are also known as 'half-cell reactions'. Half-cell reactions are necessary for corrosion to occur. They take place concurrently (Smith and Virmani, 2000). For steel in concrete, in the presence of oxygen but without chlorides, the following half-cell reactions occur:

Anodic reaction: $2Fe \rightarrow 2Fe^{2+} + 4e^-$ (2.4)

Cathodic reaction: $O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$ (2.5)

Sum of the reactions: $2Fe + 2H_2O + O_2 \rightarrow 2Fe(OH)_2$ (rust) (2.6)

The anodic reaction is the oxidation process (Ahmad, 2003). It represents the dissolution of iron and the release of electrons (Gulikers, 1996; Huang *et al*., 1996; Raupach, 1996). It is favoured by the breakdown of the passive layer. This reaction is used to measure the rate of corrosion of reinforcing steel. The type of anodic reaction depends on the pH of the interstitial electrolyte, the presence of aggressive ions, the availability of oxygen and the electrochemical potential at the steel surface (Ahmad, 2003; Qian *et al*., 2006).

Similarly, the cathodic reaction represents the reduction process. It is characterised by the conversion of electrons, water and oxygen into hydroxyl ions (Raupach, 1996; Smith and Virmani, 2000; Ahmad, 2003; Bertolini *et al.*, 2004a). Hydroxyl ions generated by the cathodic reaction increase the local alkalinity of concrete. The increase in the alkalinity of concrete contributes to the strengthening of the passive layer (Broomfield, 2007). The possible cathodic reactions that can take place in concrete depend on the availability of oxygen at the interface of the reinforcement, the pH of the solution in the vicinity of the steel surface and the electrochemical potential (Ahmad, 2003; Qian *et al*., 2006).

2.5. Corrosion rate and its influencing factors

The corrosion rate is an important factor for consideration during the corrosion process. The rate at which corrosion occurs is controlled either by the rate of the anodic (or cathodic) reactions or by the ease with which ions can be transferred between these reactions (Bungey *et al*., 2006; El-Reedy, 2008).

The corrosion of steel in concrete is influenced by many factors. Generally, corrosion rate depends on the availability of oxygen and water, the type of cations, the number and size of voids and cracks, and the concrete resistivity and temperature (Taylor *et al*., 1999). However, for a given concrete quality, the corrosion rate after initiation depends mainly on moisture

content and resistivity of concrete (Andrade *et al*., 2002), temperature, oxygen availability and pH of pore water. Other factors such as type of cement and admixtures, cover depth, quality of concrete, curing conditions and the presence of cracks also affect the corrosion rate (Raupach, 1996; Yalçyn and Ergun, 1996; Morris *et al*., 2002; Ahmad, 2003; Christodoulou, 2009; Subramaniam and Bi, 2010). Independent studies by Dehghanian (2003) and Andrade *et al* (2002) have also reported that corrosion rate depends on the condition of the metal surface (metal finish, presence of mill scale, or prior corrosion products) and the influence of climatic changes respectively.

The rates of anodic and cathodic reactions (Equations 2.4 and 2.5) can be changed by withdrawing electrons or supplying additional electrons to the steel. From Le Chatelier's principle on chemical equilibrium, it can be argued that the constant withdrawal of electrons from reinforcing steel will result in an increase in the anodic reaction rate (Equation 2.4). Conversely, a continuous supply of additional electrons from an external source to the reinforcing steel will result in a decrease in the rate of anodic reaction. Therefore, to prevent, or minimise the corrosion rate, electrons ought to be supplied continuously to the steel from an external source – as in the case of CP. Alternatively, the conditions at the concrete/steel interface can be modified such that the anodic reaction is minimised. In summary, the corrosion rate can be stopped, or reduced to negligible values, when one or a combination of the following conditions exists:

i. the anodic process is slow because the reinforcement is passive, as when the concrete is not carbonated and does not contain chlorides

ii. the cathodic process is slow because the rate at which oxygen reaches the surface of the reinforcement is low, as in the case of water-saturated concrete

iii. the electrical resistance of the concrete is high - as in the case of structures exposed to environments which are dry or low in relative humidity

Bertolini *et al* (2004a) refers to the first case as passive control, the second as control of oxygen transport, and the third as ohmic control.

2.6. Manifestation of rebar corrosion

Corrosion of steel in concrete is usually manifested by the formation of rust. Rust is an expansive product whose volume per unit mass lies between 2 and 14 times that of steel. Thus, the formation of rust is usually accompanied by the generation of tensile forces. These forces may exceed that of concrete thus resulting in the cracking, spalling and delamination of cover concrete (Cleland *et al*., 1997; Martínez and Andrade, 2009; Stephenson and Kumar, 2009; Aveldaño and Ortega, 2011). This phenomenon is shown in Figure 2.6.

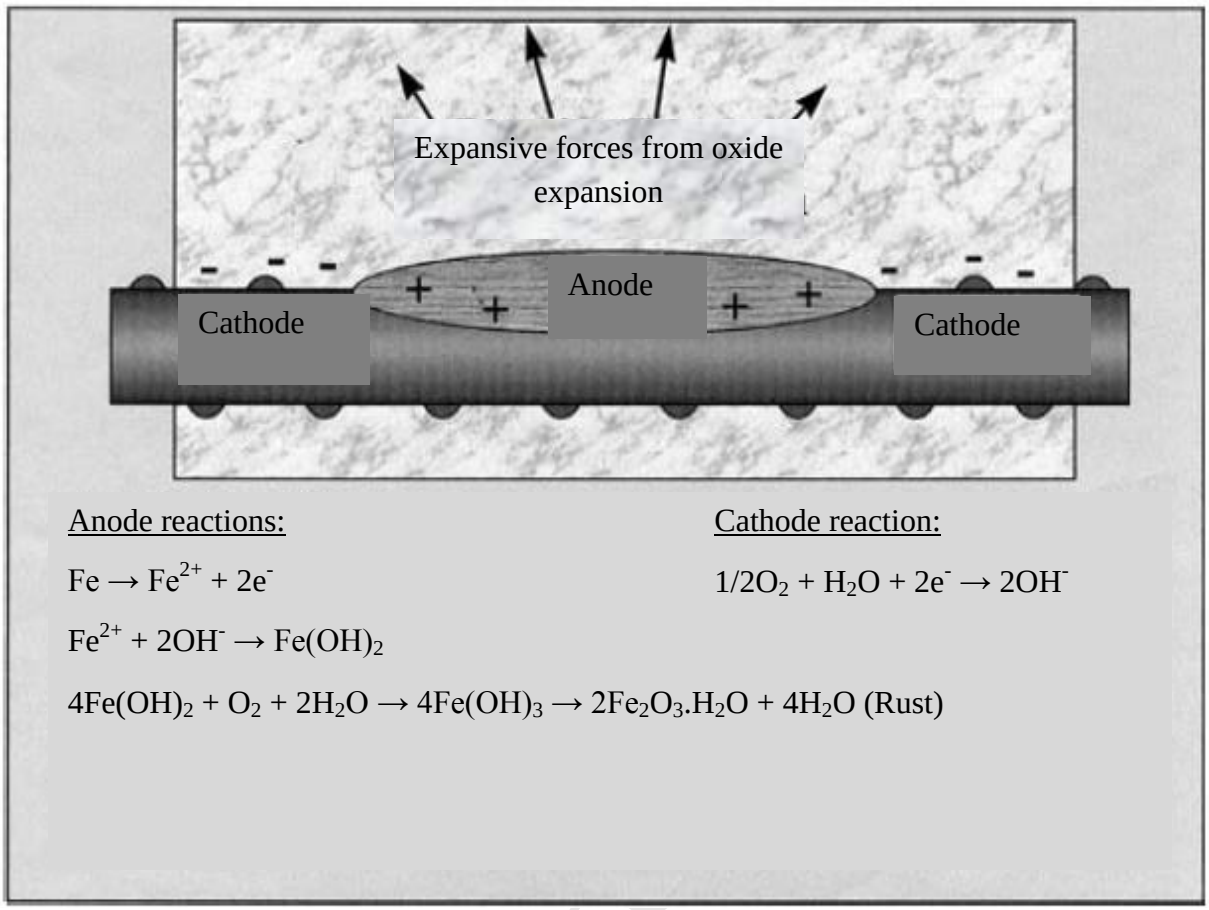


Figure 2.6: The corrosion process of steel in concrete (adapted from Revie and Uhlig, 2008)

Corrosion-induced cracks reduce the time to corrosion initiation as well as increase the corrosion rate (Gu *et al*., 2001; Subramaniam and Bi, 2010). This is due to the fact that cracks expose the reinforcing steel to the external environment. In addition, cracks facilitate the faster movement of oxygen, moisture and chlorides. They also modify the chemical, electrochemical (Aveldaño and Ortega, 2011) and transport properties of concrete. Corrosion-induced cracks results in loss of serviceability and failure of RC structures (Allan, 1995; Andrade and Alonso, 2001; Wu and Wu, 2010). Case studies conducted by Costa and Appleton (2002) and Poupard *et al* (2006) provide comprehensive information on manifestation and effects of corrosion in RC structures and structural elements exposed to a marine environment respectively. Figures 2.7-2.9 show some RC structures that are deteriorating as a result of corrosion of reinforcing steel.

Figure 2.7: Corrosion ravaged columns of a highway bridge in Illinois (Roberge, 2008)

Figure 2.8: Concrete spalling from the underside of a bridge deck (Covino *et al*., 2002)

Figure 2.9: Deterioration of a deck wall due to corrosion (Costa and Appleton, 2002)

In summary, Mackechnie and Alexander (2001) reported that the manifestation of corrosion depends on influences such as:

i. Geometry of the element (large diameter bars at low covers allow easy spalling)
ii. Cover depths (deep cover may prevent full oxidation of corrosion product)
iii. Moisture condition (conductive electrolytes encourage well defined macrocells)
iv. Age of structure (rust stains progress to cracking and spalling)
v. Rebar spacing (closely spaced bars in walls and slabs encourage delaminations)
vi. Crack distribution (cracks may provide low resistance paths to the reinforcement)
vii. Service stresses (corrosion may be accelerated in highly stressed zones)

2.7. Corrosion and its influence on service life of RC structures

Concrete provides an ideal environment for protecting steel embedded in it from corrosion (Mietz and Isecke, 1996; Balafas and Burgoyne, 2010). The protective effect is attributed to the passive film that forms on the steel surface (as explained in chapter one) and the fact that concrete acts as a barrier to outside elements which are aggressive to the steel reinforcement. However, when reinforcement corrosion is initiated, it can be assumed that it will progress almost at a steady rate; thus shortening the service life of the RC structure. Therefore, the condition of the passive film and the rate at which it breaks down plays an important role in

defining the service life of RC structures (Rodriguez *et al*., 1997; Hassanein *et al*., 1998; Val *et al*., 1998).

The service life of RC structures can be defined in different ways (Maage *et al*., 1996). However, in the context of this study, service life refers to the time between construction and the point where a structure reaches a limit state; that is, a point where the consequences of corrosion cannot be tolerated on grounds of structural safety and serviceability. According to Tuutti's classical model (Tuutti, 1982); the service life of corroding RC structures can be divided into two main phases: the initiation phase and the propagation phase (Gulikers, 1997; Liang *et al*., 1999; Martin-Perez and Lounis, 2003; Castellote and Andrade, 2009). Nevertheless, Torres-Acosta *et al* (2007) reports a third phase known as the residual life phase. A phase of service life is defined as a time period, within the whole service life, at the end of which (but not during which) intervention, such as repairs, strengthening or rehabilitation, is required (Li, 2004). A schematic representation of the service life of an RC structure is shown in Figure 2.10.

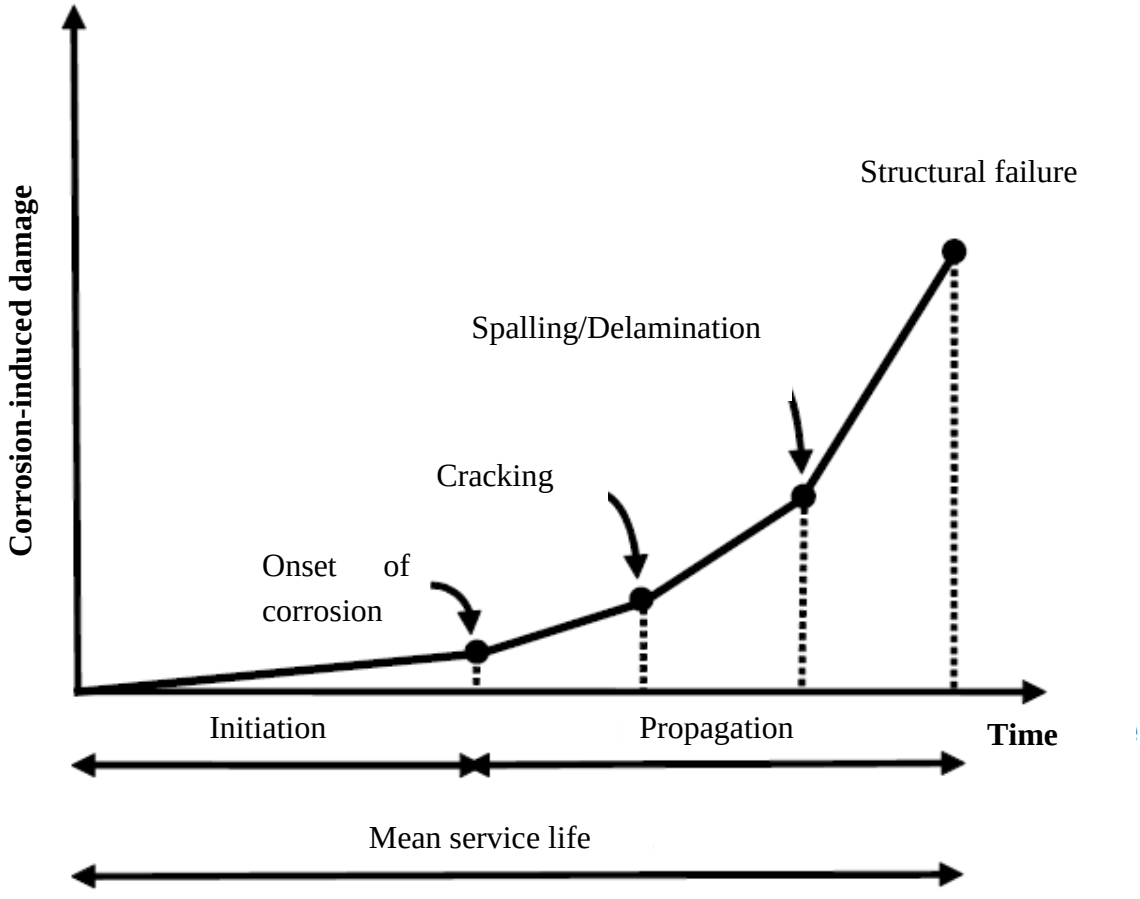


Figure 2.10: Service life of an RC structure (adapted from Tuutti, 1982)

The initiation (or incubation) phase is the period between the completion of a newly built structure and the initiation of corrosion (Bertolini *et al*., 2004a; Li, 2004; Osterminski and Schießl, 2009; Reou and Ann, 2009). During this phase, steel remains passive, while the chemical composition of concrete pore solution is gradually altered due to ingress of

aggressive species (notably chlorides and carbon dioxide) from the outer environment into the bulk of the concrete. The initiation period is characterised by a very low corrosion rate (L'Hostis *et al*., 2008). The duration of the initiation phase depends on concrete quality and depth of cover (e.g. permeability and the nature and intensity of cracks), the presence of protective systems (membrane, epoxy coating of reinforcement), the rate of penetration of the aggressive species and the concentration necessary to depassivate reinforcing steel (Martin-Perez and Lounis, 2003; Poupard *et al.*, 2004; Hunkeler, 2005; Castellote and Andrade, 2009).

The propagation phase is the period between the point where corrosion initiates (as a result of depassivation) and the point where a structure reaches a limit state; that is, a condition beyond which the consequences of corrosion cannot be tolerated further on grounds of structural safety, serviceability and aesthetics (Maage *et al*., 1996; Martin-Perez and Lounis, 2003; Castellote and Andrade, 2009). This period is characterised by active corrosion at a significant rate. The propagation phase ends when the first crack that appears on the concrete surface is less than 0.1 mm wide (Torres-Acosta *et al*., 2007). The third phase (or residual life stage) starts from the time of external visual degradation (cracks < 0.1 mm wide) until a degradation level is such that the design load factors (for dead and live load) equals unity, before structural collapse.

2.8. Effects of rebar corrosion on RC structures

Although corrosion of steel may not, per se, immediately affect the integrity and the ultimate load capacity of a RC structural member, it is the most complex, insidious and destructive form of damage (Swamy and McHugh, 2006). Once it starts, it is almost impossible to stop the process until eventually the safety, stability and design service life are all drastically reduced with time.

The effects of corrosion of steel in RC structures are vast. However, they can be categorised into four broad groups, namely: structural effects, durability effects, aesthetic effects and socio-economic effects. These effects are described below.

2.8.1. Structural effects

Corrosion of steel affects the serviceability, safety and the overall structural performance of RC structures (González *et al*., 1995; Bertolini *et al*., 2004a; Yehia and Host, 2010). Most important, corrosion leads to the reduction in strength of RC structures. The main parameters (which pertain to structural strength) that are affected during corrosion are: the steel-concrete bonding system, the cross sectional areas of rebars, the effective cross-sectional area of the concrete section, load-carrying capacity, ductility, shear and torsional strength (Andrade and Alonso, 2001; Lee *et al*., 2002; Imperatore and Rinaldi, 2009). Vidal *et al* (2007) have also reported on the reduction in the bending stiffness of corroding structural members.

Corrosion results in reduction in the cross-sectional area of steel. It also results in loss (or decrease) of bond. The reduction in the cross sectional area of steel as well as the loss of bond usually leads to unserviceability and loss of strength or load-carrying capacity (Fu and

Chung, 1997; Rodriguez *et al*., 1997; Zhang *et al*., 2009b; Shodja, 2010; Pradhan and Bhattacharjee, 2011). Pitting (or localised) corrosion, for example, results in the progressive reduction in the cross-sectional area of rebars to a point where rebars are incapable of withstanding the applied load (Ahmad, 2003). Thus, pitting corrosion may result in a catastrophic failure. Martin-Perez and Lounis, (2003) have also reported that corrosion reduces a structure's tensile capacity. Figure 2.11 shows some bars whose cross sectional area has been reduced as a result of corrosion.

Figure 2.11: Corroding prestressing steels after opening of the duct (Source: http://www.efcweb.org/Working+Parties/WP+11/WP+11+History.html accessed on 18th November 2011)

Corrosion-induced cracks create a compressive softening effect. This effect may lead to reduction in strength. In addition to the loss of strength, the compressive softening effect affects a structure's ductility and load-deformation response. Capozucca, (1995) and Revie and Uhlig (2008) reported on the consequences of compressive softening effect. The reduction of bond between concrete and reinforcing steel is a parameter of utmost importance. This is due to the fact that the bond between concrete and steel is used while assessing the structural performance of RC structures. The reduction in bond is primarily caused by: reduction of rebar confinement that results from cracking of the surrounding concrete (Lundgren and Plos, 2006) and by the reduction in rebar ribs due to pitting (Capozucca, 1995; Fischer *et al*., 2009). The reduction in bond has been investigated and reported by several researchers. A study by Imperatore and Rinaldi (2009), for example, found out experimentally that a 2% diameter loss due to corrosion results in 80% bond reduction. Similar observations have been reported by Fang *et al* (2006b) and Chung *et al* (2008a). Cabrera (1996), Rodriguez *et al* (1997) and Ballim and Reid (2003) also found out that corrosion of reinforcement results in an increase in the magnitude of mid-span deflection and crack widths at service loads respectively. Cabrera's findings were also arrived at

through an independent study by Li (2004). Lee *et al* (2002) and Fang *et al* (2004) conducted tests on concrete specimens and found out that bond rigidity and the maximum bond strength of RC decreases with an increase in the percentage of corrosion. Fang *et al* (2006a) also found out that corrosion reduces the bond of reinforcement under cyclic loading.

Cracking and spalling of concrete also results in reduction/damage of the section (Capozucca, 1995; Beamish and El-Belbol, 1998; Rostam, 2003). Damage to the concrete section causes a variation of the bending rigidity of the structural element. Cross sectional damage also causes an inclination of the neutral axis which is necessary for the internal equilibrium of the section. This is due to the fact that geometric reductions in concrete sections may displace the shear centre from its original position; consequently resulting in an unexpected torsion which can be difficult to evaluate (Capozucca, 1995). Such effects are undesirable in thin-walled sections which are only designed for bending stresses

Finally, corrosion of shear reinforcement affects the shear resistance mechanism in RC sections by: reducing the length of the concrete struts as a result of damage due to corrosion of the core reinforcement (stirrups and bars); and by reducing the cross-section of the core reinforcement which may cause yielding of steel (Capozucca, 1995).

2.8.2. Durability effects

Corrosion has been considered as the main cause of the limited durability of RC structures (Cairns, 1995; González *et al*., 2004; Vidal *et al*., 2007). It results in cracking and spalling of concrete within the cover zone. This phenomenon has been discussed in previous sections of this chapter. The penetrability of concrete, which is mainly governed by the quality and condition of the cover concrete, can substantially increase in the presence of cracks. An increase in penetrability, for example, aggravates corrosion of steel in concrete by creating preferential paths for the ingress of potentially deleterious species. Acidic gases and aggressive ions, for example, penetrate cracked materials much easier in cracked than in crack-free concrete. Cracks also enhance the transfer of aggressive ions/substances through capillary suction. Thus, it is evident that the increase in penetrability, which results from the presence of cracks, causes a decrease in the service life of RC structures. The reduction in service life is attributed to the reduction in the corrosion initiation period as well as the accelerated corrosion damage during the propagation period.

2.8.3. Aesthetic effects

Artistic structures and monuments play a significant role in cultural heritage because of their historical relevance. Consequently, their aesthetic appeal as well as their service life is an important factor to be considered. The end of service life of these structures is usually defined by, among many other parameters, the presence of stains of corrosion products on the concrete surface (Redaelli and Bertolini, 2011a). Thus, rust stains cannot be accepted while such structures are serving their purpose. In addition, the presence of rust stains on corroding structures necessitates the need for re-painting. Similarly, the presence of wide and/or visible cracks on columns, beams, etc compromises the aesthetics of RC structures.

2.8.4. Social and economic effects

Corrosion of reinforced concrete structures interferes with cultural heritage (Bertolini *et al*., 2008); especially when it leads to the demolition of structures which are treated as antiquities or historic landmarks. Covino *et al* (2002), for example, reports that the replacement of the Alsean Bay Bridge in Waldport (Oregon) in 1987 resulted in a public protest. Also, though not to a large extent, the loss of human life as a result of structural collapse, or big pieces of spalling concrete, imparts a heavy burden on the society (Rostam, 2003; Broomfield, 2007). Corrosion related accidents; such as the one reported by Costa and Appleton (2002), may result in heavy financial losses and reduced trade between countries. Corrosion-induced failure of RC cell structures and other containers used in nuclear plants may be catastrophic (L'Hostis *et al*., 2008).

Corrosion control costs also drain national economies as they consume a large proportion (about 3% - 4% of the GNP) of a country's total annual spending (Huang and Yang, 1997; Smith and Virmani, 2000; Rapp, 2006; Yang, 2008). Chloride-induced corrosion, for example, has been reported to result in high maintenance costs on old coastal bridges (Covino *et al*., 2002; Chagnon and Lounis, 2006; El-Reedy, 2008). In the US alone, the costs of corrosion - according to reports by Huang and Yang (1997) and Smith and Virmani (2000) - are increasing at an annual rate of $500 million. Similarly, a recent report by Trejo *et al* (2009) stated that the FHWA, in 2001, reported that the direct cost of corrosion incurred by industry and government agencies in the US amounted to $276 billion per year. Moreover, Sohanghpurwala (2009) estimates that $5.8 billion will be incurred as direct costs during the maintenance of bridges that are deteriorating from rebar corrosion in the US during a 20-year period from 1999-2019. In the UK, the Department of Transport estimated a total repair cost of £616.5 million due to corrosion damage to 200 motorway bridges (Broomfield *et al*., 2002). From these figures, it is evident that a lot of resources have been diverted into corrosion control at the expense of improving the standards of living and quality of life through healthcare and other relevant sectors of the economy. In addition, the economical impact of corrosion can be significant in underdeveloped countries.

2.9. Corrosion control techniques

The lack of durability of RC structures, as a result of corrosion of reinforcing steel, is associated with negative economic implications to a nation's well-being as well as a great threat to the sustainable growth of concrete and construction industries (Swamy and McHugh, 2006). As a result, a lot of focus has been directed towards the development of a number of corrective and preventive strategies which have the potential to extend the durable service life of RC structures. Essentially, the control of corrosion in RC structures is accomplished by controlling all the factors that initiate and enhance the corrosion process. The main principles used to control/prevent corrosion include (Bardal, 2004):

i. Appropriate material selection

ii. The change of concrete properties/environment

iii. Suitable design

iv. Electrochemical techniques, that is, cathodic or anodic protection

v. Application of coatings

Techniques such as the use of pozzolanic cement replacement materials, concrete surface coating, galvanised, stainless and epoxy-coated reinforcement, corrosion inhibitors and electrochemical techniques have been widely used to confer longer service in RC structures. A brief description of these techniques is given below.

2.9.1. Conventional repair, supplementary cementitious materials and polymers

Conventional repair was one of the first approaches used to repair structures that are deteriorating from corrosion of reinforcing steel. It involves the repair of specific portions of RC structures (and structural elements) that are deteriorating from rebar corrosion. Conventional repair incorporates patch repairs and the use of non-structural concrete jackets.

Patch repairs use sand/cement mortars, fibre reinforced composites and other cementitious materials. It involves the removal and the consequent replacement of deteriorated concrete with sound and uncontaminated concrete. Despite its wide use, many patch repairs have failed from corrosion along the periphery of the repaired patch - a phenomenon known as incipient anode or halo effect (Sohanghpurwala and Scannell, 1994). Non-structural jackets have been used to repair corrosion-induced deterioration in bridge pilings. They come in many forms and are mainly made from wood and fibreglass. They also use fillers made from sand/cement mortars and synthetic polymers. They are installed in a manner that ensures that they encapsulate the structural member that will be protected. Non-structural jackets, however, have been found to be incapable of providing protection against future corrosion damage. Sohanghpurwala and Scannell (1994) reported that the use of jackets may accelerate the corrosion process around the jacketed area.

The advancement in knowledge with respect to materials technology has also helped in reducing corrosion through the use of supplementary cementitious materials (SCMs). Latex modified concrete, ground granulated blast furnace slag (GGBS), fly ash (FA) and silica fume (SF) concrete have been used in new construction as well as in repair materials (Sohanghpurwala and Scannell, 1994). The beneficial effects of SCMs are attributed to the refined, less continuous pore structure that leads to a decrease in permeability and chloride solidification (Asrar *et al*., 1997; Rha *et al*., 2001; Trejo *et al*., 2009). Thus, concrete made from SCMs have a lower permeability and electrical conductivity than those made from OPC. Their use, therefore, results in a reduction in the rate of corrosion. Polder and Peelen (2002) reported that slag and fly ash in concrete help in delaying the onset of corrosion by slowing down the ingress of chloride. Materials such as high alumina cement also help in reducing corrosion due to the fact that they not only suppress the electrochemical reaction on the steel surface, but they also have a high chloride threshold and binding capacity (Ann *et al.*, 2010). Asrar *et al* (1997 and 1999) have also reported on the improved corrosion protection and durability that is associated with the use of microsilica. Wheat (2002) also reports that the

addition of polymer fibres and the encapsulation or encasement of concrete in a polymer resin or polymer composite helps to reduce the corrosion of concrete. This is due to the fact that these substances reduce the permeability of concrete.

2.9.2. Coating the reinforcement

Coatings enhance the performance of reinforcing steel under aggressive conditions where passivity cannot be guaranteed. The following mechanisms govern the way with which coatings work (Bardal, 2004):

i. Barrier effect: where any contact between the corrosive medium and the metallic material is prevented

ii. CP: where the coating material acts as a sacrificial anode

iii. Inhibitor passivation, including cases of anodic protection

Organic coatings such as epoxies form an impervious barrier that prevents corrosion of steel in RC structures. Epoxy coated reinforcement (ECR) are produced by applying epoxy powder on freshly blasted steel surfaces at high temperatures. They have been used to protect rebar corrosion in many structures. However, a lot of structures that incorporate ECR fail to realise their desired service life due to cathodic disbondment, the loss of the epoxy's insulating properties under macrocell action, and the evolution of hydrogen at secondary cathodes after the pH has decreased due to macrocell action (Trejo *et al*., 2009). Sohanghpurwala and Scannell (1994) have also reported on the inability of ECR to prevent corrosion in structures exposed to marine environments.

Coatings, such as hot-dip galvanizing, have been reported to increase the service life of RC structures that are exposed to chloride environments (Bautista and González, 1996). However, the success with which corrosion can be prevented depends on the cleanliness of the rebar surface amongst other factors. Also, the friction between the reinforcing steel ought to be minimised while manufacturing and painting steel (El-Reedy, 2008; Berke, 2006).

2.9.3. Corrosion inhibitors

A corrosion inhibitor is a chemical substance that reduces the corrosion of metals without a reduction in the concentration of corrosive agents (Mackechnie and Alexander, 2001). They are manufactured and sold as organic or inorganic compounds. Inhibitors control corrosion in RC structures through a combination of the following mechanisms (Trépanier *et al*., 2001; Trejo *et al*., 2009):

i. They delay the onset of corrosion by reducing the rate of ingress of chlorides

ii. They increase the chloride threshold value for corrosion initiation

iii. They reduce the rate of corrosion once it is initiated

iv. They affect the degree to which chlorides are chemically bound in the concrete cover

v. They affect the electrical resistance and chemical composition of the concrete system

Inhibitors are classified in many ways; however, they can be grouped into three broad classes depending on their effect on the corrosion process. These classes are: (a) anodic, (b) cathodic and (c) mixed inhibitors (Trépanier *et al*., 2001; Vaysburd and Emmons, 2004; Saraswathy and Song, 2007; Trejo *et al*., 2009). Other inhibitors, named pore blockers, retard the ingress of chlorides and/or oxygen by filling pores in the concrete matrix (Trépanier *et al*., 2001). Corrosion inhibitors can be added to the fresh concrete, dissolved into the water used for mixing, or can be applied on the surface of the hardened concrete (Sánchez and Alonso, 2011). In addition, corrosion inhibitors are either used as a preventive or as a curative measure to increase the service life of RC structures. In preventive applications, inhibitors are either applied as an admixture to the fresh concrete or applied on the surface of hardened concrete. Inhibitors used in these applications have to penetrate the concrete cover to reach the steel surface. In curative applications, inhibitors are applied on the surface of hardened concrete with the goal of reducing the corrosion rate of the rebars.

Corrosion inhibitors admixed to fresh concrete extend the initiation time and/or reduce the corrosion rate of reinforcement steel after depassivation (Bertolini *et al*., 2004a). The most common inhibitor used for RC applications is calcium nitrite. This is due to the fact that it offers a number of advantages such as being non-detrimental to the mechanical properties of concrete (Saricimen *et al*., 2002). MFP (sodium monofluorophosphate) has also been used for concrete applications. Compared to other preventive methods, corrosion inhibitors offer the advantages of low cost and easy handling (Ormellese *et al*., 2006).

2.9.4. Concrete surface treatment

Concrete surface treatment is a corrosion control technique that involves the application of materials on the surface of concrete. These materials reduce the permeability of concrete. The reduction in permeability reduces the corrosion rate of steel by retarding or preventing the ingress of chlorides, humidity and oxygen into concrete (Orlikowski *et al*., 2004; Berke, 2006; El-Reedy, 2008; Gjørv, 2009). Surface treatment also results in an increase in concrete resistivity. The increase in concrete resistivity as well as the reduction in permeability suppresses the corrosion reactions; thus resulting in a reduction in corrosion rate.

The composition of materials used for surface treatments vary from polymeric to cementitious, while their dimensions and location vary from atomic layers to centimetres and from ‘on top’ of the concrete surface to completely inside the concrete pores respectively (Bertolini *et al*., 2004a). Surface protection products may be grouped into four classes (Gjørv, 2009):

i. Organic coatings that form a continuous film on the concrete surface
ii. Hydrophobic treatments that line the interior surface of the concrete pores
iii. Treatments that fill the capillary pores
iv. Thick and dense cementitious layer

Surface coating is usually carried out using two popular methods, namely: spraying of liquid materials or painting by brush and using sheets and membranes from rubber and plastic or textiles immersed in bitumen (El-Reedy, 2008). The permeability of the concrete cover can also be decreased through the application of a surface sealer after the concrete has hardened (Berke, 2006) or through the use of a latex admixture.

2.9.5. Electrochemical techniques

Electrochemical repair techniques have emerged as very promising means to prevent the rapid deterioration of RC structures in highly corrosive environments (Miranda *et al*., 2006). They are characterised by their ability to remove aggressive species (e.g. chlorides) while maintaining the concrete cover. They not only raise the chloride threshold, but also promote the creation and sustainability of passivation of reinforcement (Yeih *et al*., 2006; Sánchez and Alonso, 2011). Electrochemical repair techniques can be divided into three main categories, namely: cathodic protection, electrochemical realkalisation, and electrochemical chloride extraction (or desalinisation). The preference with which any of these techniques can be used to control rebar corrosion depends on the type of structure and the aggressiveness of the exposure environment.

Cathodic protection (CP) is the oldest of the three electrochemical techniques (Andrade *et al*., 2009). It controls corrosion of steel in RC structures through the use of an electric current that is generated from a rectifier or a sacrificial anode. CP, when applied/installed properly, can prevent or stop corrosion of steel in concrete structures that are subjected to carbonation (Redaelli and Bertolini, 2011b) and/or chloride-induced corrosion. However, Berkeley (1992) reports that CP is not appropriate as the primary means of combating corrosion of structures that are affected by gaseous diffusion. Details on how CP systems work will be presented in the next chapter.

Electrochemical realkalisation (ER) is a technique that is used to stop rebar corrosion in carbonated concrete. During ER treatment, hydroxyl ions are produced at the rebar surface, at the same time, carbonate ions penetrate from the surface of the concrete through electro-osmosis, diffusion and/or capillary absorption (Redaelli and Bertolini, 2011a). As the treatment proceeds, the alkaline regions may extend to the whole thickness of the concrete cover. Thus, this technique aims at restoring the alkalinity of concrete through a twofold mechanism, namely: the production of alkalinity at the steel reinforcement and the penetration of an alkaline electrolyte in which the anode is embedded (Bertolini *et al*., 2008). Realkalisation turns carbonated concrete into alkaline concrete, and therefore, re-creates an environment favourable for the passivation of steel. The current for ER application can be generated from sacrificial anodes (Tong *et al*., 2011) or through an external current. External current is supplied using a temporary anode system placed at the concrete surface. Currents between 1 - 2 A/m^2 are usually applied for a period that ranges between three and seven days (Banfill, 1997). Nevertheless, for a given applied current density and period of treatment, the actual realkalisation depends on the characteristics of the material as well as on geometrical factors, such as the thickness of the concrete cover (Redaelli and Bertolini, 2011a). Sodium

carbonate is usually used as the preferred electrolyte for ER applications. Anode systems used for ER are usually applied on the structure during the period of treatment, at the end of which they are removed, leaving the surface unchanged. This feature, therefore, makes ER an attractive repair technique for use in the conservation of historical buildings and cultural heritage where the conservation of the original materials and surfaces is often a stringent requirement in the design of the repair. An example of an application where ER was effectively used to repair concrete in a historical building built in the 1920s is given in a report by Bertolini *et al* (2008).

Electrochemical chloride extraction (ECE) is an in-situ restoration technique that is designed to remove chlorides and increase the alkalinity adjacent to the reinforcing steel. A temporary treatment is attached to the concrete and the applied voltage causes a direct current, which can be up to 1 A/m^2, to flow through the concrete for typically 4 to 8 weeks (Sharp *et al*., 2002). During this operation, the steel reinforcement acts as the cathode, and an extended anode is placed in a suitable electrolyte, usually NaOH, $Ca(OH)_2$ or Na_3BO_3, on the concrete surface. The negative terminal is connected to the steel. The negative polarity that is induced on the steel repels anions and attracts cations present in the concrete pore solution. ECE can be improved when it is combined with corrosion inhibitors (Sánchez and Alonso, 2011). Although similar to CP, ECE is economically more attractive as it does not need ongoing, long term maintenance (Miranda *et al*., 2006; Streicher *et al*., 2009). A study by Orellan *et al* (2004) on laboratory cast samples showed that ECE can reduce the free chloride content in contaminated concrete by up to 40 - 45%. Compared to traditional repair methods, ECE and ER offer the merits of being much less costly and friendlier to the environment and worker health (Miranda *et al*., 2006). Despite the fact that ECE is capable of removing the cause of corrosion, it cannot halt the damage if the process of deterioration is advanced. Moreover, ECR is not capable of repassivating highly corroded steel surfaces (Miranda *et al*., 2007).

Finally, it is important to note that electrochemical repair methods can be very effective when they are applied at the right time, that is, if applied before the passive-active transition takes place (Miranda *et al*., 2006). This is due to the fact that they realkalise the concrete and substantially reduce the chloride level. When applied as a preventive measure, electrochemical repair methods can prolong the corrosion initiation time indefinitely if used repeatedly, provided that the structure's chloride profile makes it advisable (Miranda *et al*., 2006). However, if they are applied 'at the wrong time', as a corrective measure, they are ineffective.

2.10. Summary

In this chapter, the electrochemical nature of corrosion, its causes and mechanisms as well as its negative effects have been critically discussed. Corrosion of steel in concrete is an electrochemical process that involves the movement of electrons. It takes place when the passive film that is formed on concrete during the hydration of cement is destroyed by acidification or de-alkalisation. The acidification and de-alkalisation of the passive film can

result from carbonation, chloride ingress or stray currents. Corrosion is initiated and enhanced when moisture, oxygen and aggressive species such as chlorides are available.

Corrosion can manifest itself in the form of macrocells and microcells. Macrocells result in the uniform removal of iron while microcells result in local removal of iron (or pitting corrosion). Corrosion of steel in concrete is normally characterised by the formation of rust. The formation of rust generates expansive forces that cause the cover concrete of structural members to crack. Corrosion of steel affects the structural integrity and aesthetics of RC structures. In addition, corrosion also results in huge economic losses. Corrosion of steel in RC structures can be controlled through: conventional repair techniques, the use of coatings and corrosion inhibitors, concrete surface treatment or electrochemical techniques.

CHAPTER 3: SACRIFICIAL ANODE CATHODIC PROTECTION SYSTEMS

Cathodic protection (CP) is a technique that has been used to control corrosion of steel in RC structures. It relies on the application of voltage to the reinforcing steel. The voltage that drives CP can be generated through: an external power supply (or rectifier), or through the use of a metal that corrodes preferentially to the reinforcing steel. CP systems that operate on the former method are called impressed current cathodic protection (ICCP), or active, systems; while those that operate on the latter method are called sacrificial (or galvanic or passive) anode cathodic protection (SACP) systems. Galvanic systems use metals that are higher than steel in the electrochemical series.

3.1. History of CP in RC structures

The principles of SACP were discovered in 1824, when Sir Humphrey Davy used iron anodes to protect copper sheathing in ships against corrosion from seawater (Concrete Society, 1989; Gummow, 2000; Whitmore and Ball, 2005; Broomfield, 2007). Thereafter, there was a prolific increase in the scope of CP usage – a phenomenon that was facilitated by continuous developments in CP technology. Notable of these developments came during the 1920s when CP was used to protect gas pipelines and underground steel storage tanks. The successful implementation of CP to protect pipelines and underground steel tanks aroused the need to extend its use in the protection of concrete structures.

Despite the fact that a review of available literature has failed to show the exact time when CP was first used in RC structures, Redaelli and Bertolini (2011b) report that pioneer work on CP of steel in concrete structures began in the 1950s. More specifically, CP of steel in concrete began prior to 1955 on prestressed concrete water pipelines. Other researchers have also reported that CP was first applied to above-ground structures in 1957 as part of an experimental programme that was aimed at investigating ways to repair highway bridges that were suffering from severe chloride-induced corrosion. The UK Highways Agency (2002) reports that trial applications of CP to atmospherically exposed RC highway structural elements commenced in 1958. Pocock (1996), however, reports that CP was first applied during field studies in an estuary in western France in the mid-sixties.

In spite of the lack of clarity in the exact times on when CP of concrete structures began, it can be inferred that early CP systems were not sophisticated. They relied heavily on the principles that had been developed following the successful use of CP to protect buried pipelines. In fact, it has been reported that early CP systems relied on either soil or water to apply and distribute current (Concrete Society, 1989; Berkeley, 1992; ELTECH Research Corporation, 1993; Highways Agency, 2002). In addition, Covino *et al* (2002) reported that early CP systems used anodes that consisted of embeddable wires, strips or mesh. Though successful, these anodes had a limited ability to distribute current uniformly to the rebar. In spite of these limitations, the need to apply CP to protect reinforcement against corrosion in aerial as well as submerged concrete structures contributed to an invigorated research in CP systems in the years that followed. The outcome of these researches resulted in the

development of several modifications which made CP systems more realistic and appropriate for application to concrete structures.

Major developments in CP of steel in RC structures can be categorised into three distinct phases. The first phase of development began in North America, in June 1973, when R.F. Stratfull applied the theory of CP to a bridge deck (Sly Park Bridge) in Placerville, California, USA (ELTECH Research Corporation, 1993; Bertolini *et al*., 2004a; Xu and Yao, 2009; Yehia and Host, 2010). This system used silicon iron anodes embedded in a layer of coke breeze. Coke breeze was used as a conductor to distribute the current uniformly over the bridge deck. An overlay of asphalt was thereafter placed on top of the coke breeze mix because coke breeze did not have the requisite material properties to serve as the riding surface (Sohanghpurwala, 2009). The success associated with this project initiated numerous trial as well as commercial small scale projects throughout North America. Similarly, the installation of the first field galvanic CP system took place on a bridge deck in Illinois in 1977 within the cooperative highway research program. This system comprised zinc ribbons and perforated zinc sheets which were placed in saw-cut holes and then covered using open graded asphalt. Nevertheless, in comparison to its ICCP counterpart in California, its success could not be realised because environmental factors, such as temperature, moisture, and salt content, appeared to play a detrimental role in its operation. Also, these systems could not achieve adequate protection because of poor distribution of CP current to the rebars as well as their relatively high cost (Funahashi and Young, 1998; Schwarz *et al.*, 2011). Consequently, further developments in galvanic CP systems declined in favour of ICCP systems.

The first phase lasted approximately a decade and was mainly concerned with the protection of bridge decks contaminated by chlorides (Pedeferri, 1996; Pocock, 1996; Whiting *et al*., 1996). One of the most important outputs from this phase was the development of new feeding and monitoring systems (anodes, overlays, reference electrodes, etc.). Similarly, new protection criteria, which were completely different from those used in CP in soil and sea water, were proposed. Following its successful application in the first phase, CP was proposed as an effective solution to steel reinforcement corrosion, especially in presence of high chloride levels where other traditional repair systems are inefficient or very expensive (Scannell and Sohanghpurwala, 1993; Bertolini *et al*., 2004a). At the end of this phase, in 1982, the US Federal Highway Administration (FHWA) issued a memorandum that stated that *'the only rehabilitation technique that has proven to stop corrosion in salt contaminated bridge decks regardless of chloride content of the concrete is CP'* (ELTECH Research Corporation, 1993; Hong *et al*., 1993; Pedeferri, 1996; Pruckner *et al*., 1996; Gummow, 2000).

The second phase of development saw the introduction of CP outside North America. This phase also saw the development of new anode materials and configurations. In addition, the scope of application of CP was expanded. CP found a range of new uses; some of which included: the protection of bridge slabs and piles, marine constructions, industrial plants, car parks, garages, water towers and buildings affected by chloride corrosion (Pedeferri, 1996; Pocock, 1996; Sohanghpurwala, 2009). Notable projects where CP systems were applied

during this period include berths in Abu Dhabi and a container terminal in Hong Kong (Pocock, 1996). This phase witnessed CP develop a track record of success and reliability, if properly designed and applied, as well as showing significant capital cost savings when compared to conventional methods of concrete repair (Pedeferri, 1996). By 1989, a total of 275 bridge structures in the US and Canada had been cathodically protected with slotted CP systems being installed in more than 100 bridge decks in the same year (Sohanghpurwala, 2009).

The third phase of development began in the 1990s. It was characterised by the introduction of cathodic prevention as well as intense commercial application of CP in corrosion control. The use of CP to control carbonation-induced corrosion in RC structures was also introduced during this phase (Redaelli and Bertolini, 2011b). Consequently, CP continued to be one of the most preferred methods of controlling the rate of corrosion in chloride contaminated structures. CP was also used to improve the corrosion resistance of steel in new RC structures which were expected to become contaminated (Pedeferri, 1996). This phase saw the extensive and rapid growth of CP due to the extensive research and trials that had been conducted in the earlier phases. Thus, CP was regarded as a technique of proven efficacy and reliability (Highways Agency, 2002). Owing to its proven reliability and efficacy, the use of CP spread from the US, Canada and Europe further across the globe. CP of atmospherically exposed RC structures, for example, had been applied to at least 100,000m^2 of concrete in bridges, car parks, harbour facilities, tunnels and buildings within the UK and in excess of 2,000,000m^2 outside UK by the end of the year 2000 (Highways Agency, 2002; Broomfield, 2007).

In spite of their failure during field trials in the earlier phases, the third phase has also been characterised by the large scale use of SACP systems since the late 1990s (Bertolini *et al*., 2002; Bertolini *et al*., 2004a; Roberge, 2008). Broomfield (2007) and Redaelli and Bertolini, (2011b) reported that the use of SACPs is progressing rapidly. Specifically, it has been reported that SACP systems have been used to protect at least 200,000m^2 of RC structures in less than five years of serious commercial exploitation. Most modern SACP systems use anodes made from zinc. Common SACP systems include: sprayed zinc anodes, zinc foil glued to the concrete surface (Zinc hydrogel systems), and zinc mesh pile jackets around bridge columns filled with sea water. In addition, zinc anodes embedded into the concrete overlay have been used to protect reinforcing steel during concrete repair.

Recent developments in the third phase involve the introduction of data loggers and telephone modem connections which have enabled remote sensing and control from a computer in the office. More specifically, Sohanghpurwala, (2009) reports that modern remote monitoring and control units can be accessed by means of telephone connection, cellular (mobile) phones or the internet and are capable of sending alarms when any of the system parameters stray out of the normal operating values. These advancements have contributed towards the development of smart structures.

Finally, it is clear that the development of cathodic protection has experienced growth as well as development with respect to technology, materials and scope. Owing to its remarkable track record, it is anticipated that future CP systems will be highly efficient. In this regard, it is imperative to know the basic principles upon which this promising technique operates.

3.2. Principles of operation of SACP systems in RC structures

The critical parameters that promote and maintain the passivity of steel in RC structures are an alkaline environment, the absence of significant ion contamination and relatively unrestricted cathodic reaction kinetics (Glass, 2001; Glass *et al*., 2001). Since corrosion is an electrochemical process that depends on the electric potential of reinforcement, a shift in potential of steel to more negative values reduces the rate of corrosion through the suppression of the anodic reaction.

SACP relies on cathodic polarisation of steel. This polarisation hinders anodic dissolution of steel by moving the steel potential into the range of passivity or by reducing the macrocouple activity on its surface (Naish and McKenzie, 1998; Glass *et al*., 2000; Raupach and Bruns, 2003; Bertolini *et al*., 2004a; Bruns and Raupach, 2009). Cathodic polarisation also increases the pH at the cathode. The increase in pH enhances the reduction of oxygen (also called cathodic reaction) (Weyers and Cady, 1984; Concrete Society, 1989; Montoya *et al*., 2009). A schematic diagram of an SACP system is shown in Figure 3.1.

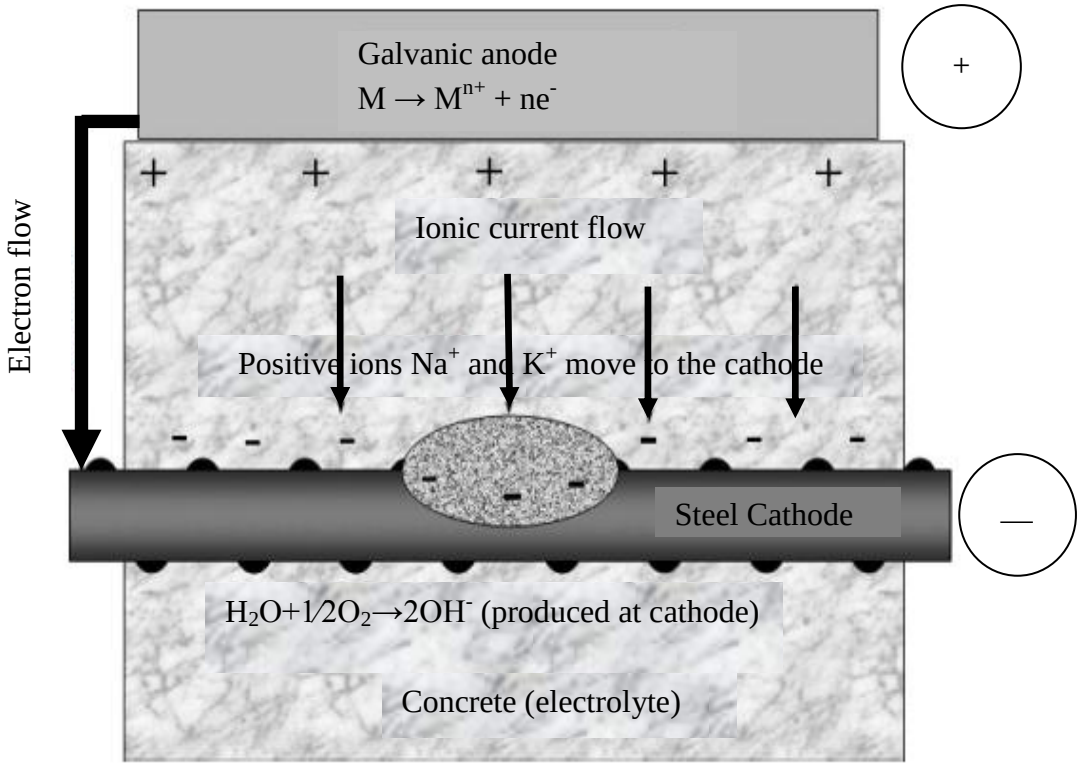


Figure 3.1: Schematics of an SACP system (adapted from Roberge, 2008)

SACP makes use of two important electrochemical phenomena: electrolysis and electromigration. Electrolysis refers to electrochemical reactions that take place at the electrodes (anode and cathode) as a result of flow of current. These reactions produce hydroxyl ions at the cathode as a result of one of the following overall reactions:

Oxygen reduction: $O_2 + 2H_2O + 4e^- \rightarrow 4OH^-$ (3.1)

Water reduction: $2H_2O + 2e^- \rightarrow 2OH^- + H_2$ (3.2)

The extent and preference for either of the two reactions above – water reduction or oxygen reduction - occurs depends on the availability of oxygen at the steel. In addition, the availability of oxygen depends on current density and moisture content of concrete (Polder, 2005). Thus, at low current densities, and in well-aerated (semi dry) concrete, the dominant reaction is oxygen reduction (Equation 3.1). However, at high current densities and/or if the concrete is (near) water saturation, insufficient oxygen can reach the steel to sustain the oxygen reduction reaction (Equation 3.2). Such conditions cause the steel potential to become very negative (Pedeferri, 1996; Polder, 2005). Furthermore, cathodic processes reduce oxygen content and produce alkalinity on the surface of reinforcing steel. These effects are beneficial in stopping and preventing corrosion. This is due to the fact that they widen the range of passivity (passive region) as well as depolarise the cathodic process. In the case of non-corroding steel they hinder local acidification and also interfere with the initiation of pits ('buffer effect') (Pedeferri, 1996).

Electromigration refers to the movement of ions within concrete. Normally, negatively charged ions move from steel towards the positive electrode (the anode) while positively charged ions move towards the negative electrode (the cathode) as shown in Figure 3.2. Together, this flow of ions carries current through concrete.

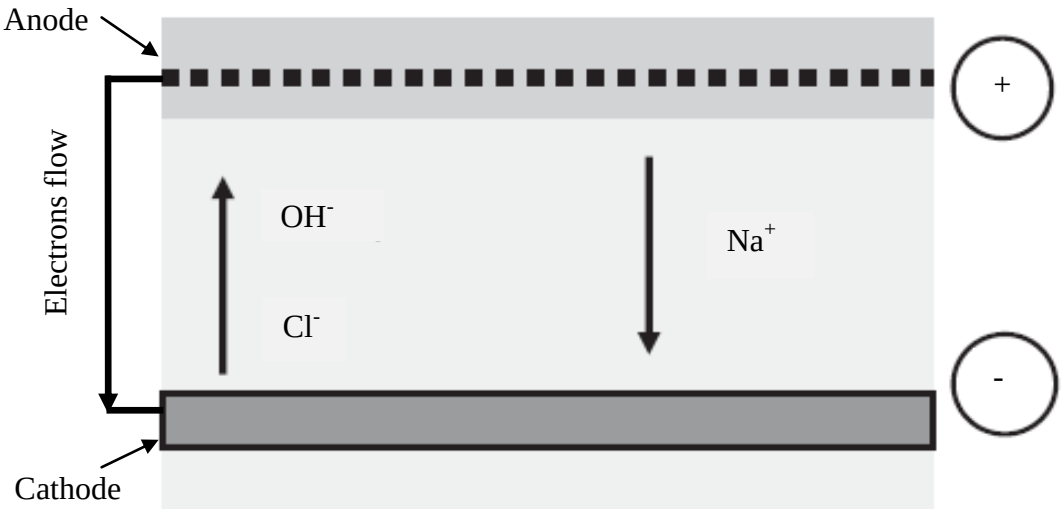


Figure 3.2: Flow of ions (electromigration) in an electrical field in the concrete cover (adapted from Polder, 2005)

Furthermore, cathodic polarisation of steel in concrete facilitates a tendency for the material in the vicinity of the steel cathode to become enriched with respect to hydroxyl ions

(OH^-), lithium ions (Li^+), sodium ions (Na^+) and potassium ions (K^+) whilst being depleted of chloride ions (Cl^-) and oxygen (O_2). Correspondingly, at the anode the surrounding material becomes progressively acidified and enriched with chloride ions. Electromigration, therefore, results in a reduction in corrosion risk (Ali and Rasheeduzzafar, 1992; Glass and Buenfeld, 1995; Glass *et al*., 2000; Parthiban *et al*., 2008a and b). The enhanced hydroxyl ion concentration, which results from the reduced alkalinity, and the reduction of chloride ion concentration near the steel surface are both beneficial in reducing the risk of corrosion. Furthermore, the generation of hydroxyl ions as well as the negative polarisation in the potential of reinforcing steel as a result of CP leads to an increase in the critical chloride threshold (de Rincón *et al*., 1997; Bertolini *et al*., 2002; Holmes *et al.*, 2011). Holmes *et al* (2011), for example, reported that the chloride threshold can be raised up to values that are greater than 2% chloride by weight of cement.

For alkaline concrete (of approximate pH 13) at normal ambient temperature, corrosion of steel may be expected to vary with potential and chloride content of the concrete as summarised in Figure 3.3.

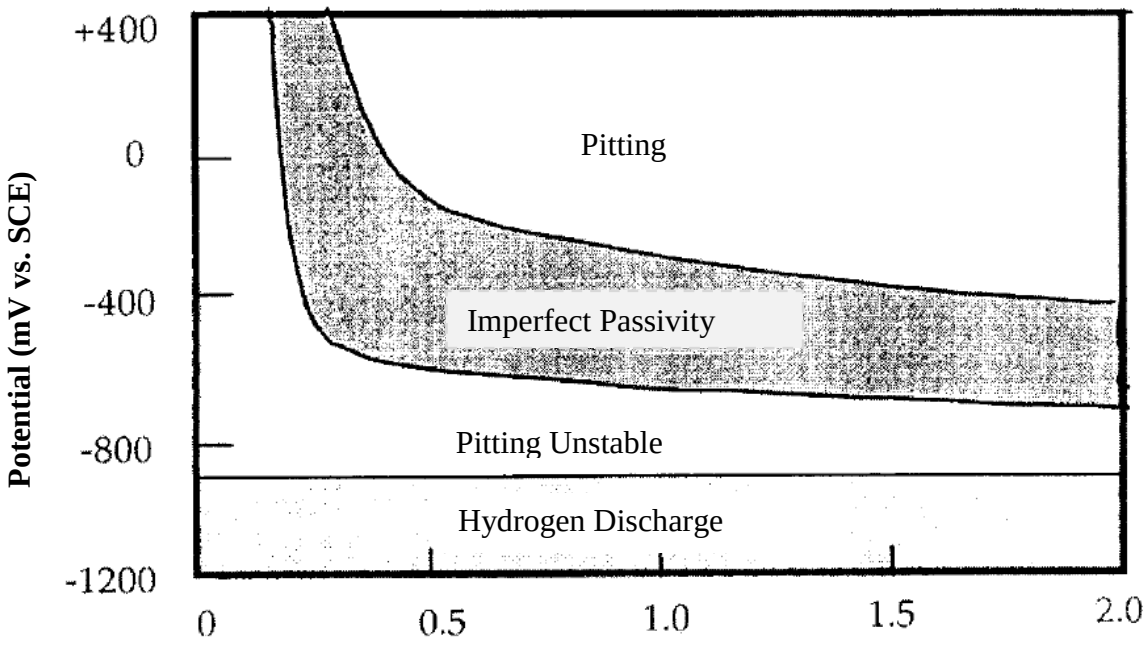


Figure 3.3: Approximate domains of electrochemical behaviour of steel in concretes with different levels of chloride contamination (adapted from Sergi and Page, 1999)

From Figure 3.3, 'Pitting' refers to conditions that can lead to initiation and propagation of pits on initially passive steel. 'Imperfect passivity' refers to conditions that allow pre-existing pits to propagate but do not favour the initiation of new pits on initially passive steel. 'Pitting unstable' refers to conditions that do not allow the initiation or propagation of pits, so that pre-existing pits tend to repassivate. Finally, 'Hydrogen discharge' refers to a condition that results from highly negative potentials that are sufficient to render the passive film

thermodynamically unstable. In such cases, hydrogen is formed cathodically (Sergi and Page, 1999).

The main objective of SACP in concrete is to reduce the susceptibility of the metal to pitting in the presence of chloride ions. The means by which this is achieved may be understood by reference to Figure 3.3. If, as is normally the case, CP is to be applied as a remedial treatment to a structure in which chloride-induced corrosion has already been occurring for some time, then the desired intention is to polarise the steel from its condition of pitting to the domain where pitting is unstable so that complete repassivation will be effected (Sergi and Page, 1999; Page and Sergi, 2000). In the event that the required extent of polarisation cannot be achieved, then depressing the potential of steel from a position within the pitting domain to one in the region of imperfect passivity will be beneficial. This is due to the fact that depressing the potential of steel not only reduces the rate of propagation of existing pits but also prevents the initiation of new ones. If CP is to be used as a means of preventing the initiation of corrosion in a structure that has not been contaminated by chloride ions, then only modest polarisation is needed to maintain the steel at a potential where pits cannot initiate (cathodic prevention). This, therefore, implies that the potential must remain below the upper boundary of the imperfect passivity domain as chloride penetration progresses (Page and Sergi, 2000).

3.3. The relationship between sacrificial anodes and service life

From Section 3.2., it can be inferred that SACP systems are capable of extending the service life of RC structures. The extension in service life can be achieved through the following effects:

i. They lengthen the corrosion initiation phase: through cathodic prevention, SACP systems are capable of prolonging the time taken to initiate corrosion

ii. They also reduce the corrosion rate during the propagation phase: Thus, the gradient of the corrosion propagation phase (i.e., the rate of deterioration) is reduced

iii. They can stop the corrosion process

From Tuutti's service life model of corroding structures, the effect of SACP systems on the service life of structures can be represented by a modified service life model of corroding structures as shown in Figure 3.4. From the diagram, it is clear that structures that incorporate SACP systems take a considerably longer period of time to reach a limit state than those structures that do not incorporate SACP systems.

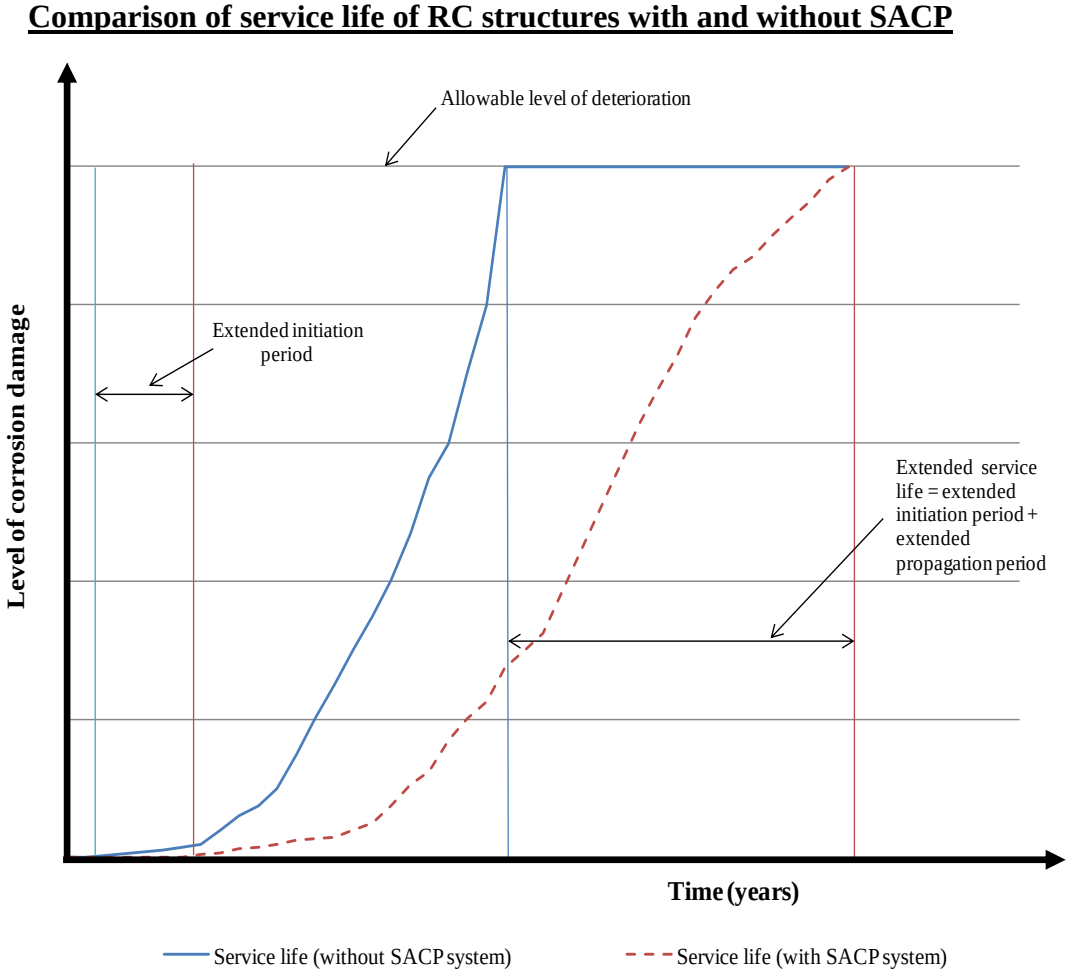


Figure 3.4: Modified service life model of corroding RC structures under SACP

3.4. Common materials for use as anodes in SACP systems

The focus of any SACP system is the anode material (Chagnon and Lounis, 2006). Therefore, the selection of an appropriate anode material is one of the most important factors that ought to be considered before an SACP system is installed. Commonly used anode materials include: metals such as Zinc, Aluminium, Magnesium and their corresponding alloys. Cement overlays with bare or nickel-coated carbon fibres and conductive polymers have also been used as sacrificial anodes (Sekar *et al*., 2007). Conductive asphalt, though not widely used nowadays due to their weight and limitations of dimensions, was widely used as an anode material on bridges in North America (Al-Idi and Al-Mehthel, 1995; Bertolini *et al*., 2004a).

3.4.1. Zinc anodes

Zinc is one of the most evaluated and widely used sacrificial anode material in RC structures, cathodic protection research, and commercialisation market (Whitmore and Ball, 2005; de Rincón *et al*., 2008; Ball and Whitmore, 2009; Christodoulou, 2009). It has been used in different configurations such as: arc sprayed, expanded mesh, perforated sheets, foil with

adhesive, ribbon pucks, expanded zinc sheets in jackets with compression panels and as a solid rectangular mass (bulk anode) (USDoT, 2001; Raupach and Bruns, 2003; Sohanghpurwala, 2009).

Zinc offers the following principal advantages: It is more electronegative than carbon steel in the standard EMF table (-0.763V versus SHE), it is readily available, environmentally compatible and easy to handle. The fact that zinc can be sprayed on the surface of concrete indicates that it can be ideally shaped to maximise contact with concrete and be as close as possible to the metal to be protected (Sagüés and Powers, 1994). Moreover, zinc offers the advantage of having a low volume resistivity. Compared to metals like aluminium and magnesium, zinc exhibits a relatively small corrosion expansion that can be managed easily through mortar shell formulation or by providing sufficient porosity and reinforcement within the anode unit (Ball and Whitmore, 2009). This property, therefore, makes zinc anodes particularly suitable for applications where anodes are embedded into concrete structures (Figure 3.5). Zinc has a high corrosion efficiency - which means that a high percentage of the electrons that are discharged as the zinc corrodes are available to protect the steel. Finally, zinc anodes are suitable for use with prestressed and/or post-tensioned concrete because their negative potential is insufficient to generate hydrogen atoms (hydrogen evolution) or cause hydrogen embrittlement (Whitmore and Ball, 2005).

In spite of the advantages stated above, zinc suffers from: poor wear and corrosion resistance; a high tendency to oxidise; high thermal expansion coefficient and high material and processing costs (Fu and Chung, 1995; Hou and Chung, 1997; USDoT, 2001). The high rate of consumption limits the service life of zinc anodes (Sohanghpurwala, 2009). de Rincón *et al* (1992) have also reported on the failure of zinc anodes to produce adequate polarisation to the protected steel. Thus, most zinc anodes have to be alloyed with other metals to enhance their performance, lifetime and efficiency. The alloying of zinc with other metals contributes to additional costs. In addition, an experimental study by Wang *et al* (2006) has also reported that the effectiveness of zinc in patch repairs is affected by the presence of leaking joints. Leaking joints refer to joints that form at the interface of the old concrete substrate and the new repair material. They usually provide a path through which water and aggressive ions, such as chlorides, can attack the reinforcement.

Zinc and zinc-aluminium alloys are normally applied onto the surface of concrete by flame spraying, as a mesh or sheet adhering via a conductive gel (Hou and Chung, 1997; Polder, 2005). Electrical contact between steel and the zinc anode is achieved either by a connecting wire, or more conveniently by direct spraying of zinc on the exposed reinforcement. The latter procedure is suited to the common morphology of damage where some reinforcement is often exposed (Sagüés and Powers, 1994). Embedded zinc anodes are suitable for use with conventionally reinforced, prestressed, and post-tensioned concrete (Ball and Whitmore, 2009). Zinc anodes for use in SACP systems are shown in Figure 3.5 and Figure 3.6.

Figure 3.5: Embedded zinc anodes used in concrete rehabilitation to prevent the formation of new corrosion sites adjacent to completed patch repairs (Roberge, 2008)

Figure 3.6: Perforated zinc cage used as a sacrificial anode in bridge pilings (Scannell and Sohanghpurwala, 1993)

Zinc anodes tend to passivate while operating. Thus, two methods have been used to overcome this problem. The first method involves increasing the pH around the zinc to be in the range of pH 14 to pH 14.5+. This environment favours the formation of soluble corrosion by-products, hence inhibiting the formation of a solid oxide film on the surface of the zinc. These types of systems are referred to as 'alkali-activated' systems (Sergi and Page, 1999; Whitmore and Ball, 2005). Alkali-activated anodes include zinc anodes with a precast mortar matrix saturated with lithium hydroxide (LiOH). The presence of lithium ions (Li^+) is usually intended to inhibit problems of alkali silica reaction in the surrounding concrete is susceptible aggregates happen to be present (Page and Sergi, 2000; Jordan and Page, 2003). The second method of activation involves putting zinc in a corrosive salt environment where the salt is in direct contact with the surface of the zinc anode. The most common type of salt used to activate zinc anodes embedded in or applied to concrete structures are halide salts (Whitmore

and Ball, 2005 Sohanghpurwala, 2009).These salts prevent the formation of a stable oxide film on anode. As zinc corrodes, it reacts with the available halides in the environment to form soluble corrosion by-products (such as zinc chlorides) that are able to diffuse away from the interface before oxidizing further. These systems are referred to as halide-activated systems. Examples of halide-activated systems include bulk zinc anodes placed on concrete piles in a saltwater environment (Whitmore and Ball, 2005).

3.4.2. Aluminium anodes

Embedded aluminium alloys have not been widely used as sacrificial anodes. Nevertheless, they have been used in experimental projects in tropical environments to protect bridge pilings (Sagüés and Powers, 1994). Principal advantages offered by aluminium include its low density, large electrochemical equivalent (high current capacity), availability and reasonable cost (Talavera *et al*., 2002; Shibli and Gireesh, 2005; Broomfield, 2007). Compared to magnesium, aluminium provides efficient protection to the reinforcing steel but with a much smaller volume of oxidation products which have better diffusion properties. Therefore, by using aluminium and a good design, concrete cracking can be avoided and adequate protection of the steel can be provided (de Rincón *et al*., 1992).

The success of aluminium anodes depends on the alloying of certain metals whose surface role is to prevent the formation of a continuous, adherent and protective oxide film on the alloy. These alloys permit the continued galvanic activity of the aluminium (Talavera *et al*., 2002). Alloy metals like selenium, tin and bismuth have been found to produce desirable galvanic performance of aluminium anodes. In fact, Shibli and Gireesh (2005) have observed that a 20% increase in the galvanic efficiency of aluminium anode can be achieved through the incorporation of 0.1% selenium.

Aluminium anodes also have inherent disadvantages. Compared to zinc, aluminium anodes are expensive. They also generate voluminous by-products of corrosion which attack the concrete cover (Sagüés and Powers, 1994; Broomfield, 2007; Christodoulou, 2009). The cracking of concrete cover may lead to further corrosion. The cracking of the concrete cover may also compromise the durability of the RC structure. Studies by de Rincón *et al* (1992) also found that the corrosion rate of aluminium anodes increases with increasing chloride concentration and porosity of concrete mortar.

3.4.3. Magnesium anodes

Magnesium, when compared to other anode materials like aluminium, has a high current output per unit weight. However, it has not been widely used as a sacrificial anode because its oxidation products crack concrete in a short period of time after installation (de Rincón *et al*., 1992; Christodoulou, 2009). The efficiency of magnesium anodes can be improved by alloying it with metals such as calcium, aluminium, zinc, and manganese (Kim *et al*., 2000). Darowicki *et al* (2003) have suggested that the best working parameters of magnesium sacrificial anodes can be achieved if magnesium (II) hydroxide is formed on the anode's surface.

3.4.4. Conductive polymers

Conductive polymers have been used as anodes for SACP. They are usually applied as a coating on the surface of concrete without heating. In addition, conductive polymers offer two main advantages, namely: they are inexpensive as well as easy to place on concrete structures under dry and semi-humid environments (Darowicki *et al*., 2003; Torres-Acosta *et al*., 2004). However, certain polymers - such as hydrogel polymers – have been found to interact with moisture from the atmosphere in tropical environments (Torres-Acosta *et al*., 2004; Schwarz *et al.*, 2011). These interactions have been found to result in the disintegration of polymers; consequently leading to the loss of CP in the RC structure. Other polymers, such as conductor filled polymers, have been reported to suffer from poor wear resistance, high thermal expansion coefficient and high material cost (Hou and Chung, 1997).

3.4.5. Other anodes

Inert graphite has been used to protect buried structures while high-silicon cast iron or platinum coated titanium is used to protect immersed sea water structures. Magnesium, zinc, aluminium, and aluminium-zinc-indium alloy anodes, welded to buried and immersed structures, provide long-term CP (Montoya *et al*., 2009). Nickel is also a potential anode material. Nickel and its alloys behave optimally, even in the presence of chlorides and in carbonated concrete (Bertolini *et al*., 2004a).

3.5. Common anode configurations and systems used in SACP systems

The selection of an anode material and its configuration is paramount to the success of any CP system (USDoT, 2001). SACP systems can comprise single or multiple anodes that are used to distribute the cathodic current. In addition, most galvanic anode cathodic protection systems can be configured as surface-applied systems (in the form of an overlay) or embedded in concrete. They can also be in the form of encapsulated and non-encapsulated systems (Sohanghpurwala, 2009).

Surface-applied systems involve the application of the anode material over the entire surface or to selected areas where CP is needed. They are generally used on the underside of bridge decks and superstructure elements such as beams, girders, diaphragms and caps. On substructure elements, surface-applied systems have been used on piles (above tidal zone), struts, columns atop footers, etc. On one hand, there are encapsulated SACP systems that comprise a concrete encasement, spray applied shotcrete, concrete overlays, saw cutting for ribbon mesh in slots or drilling to insert discrete anodes or an expanded zinc mesh encapsulated in a cementitious material and contained in a fibreglass jacket that is installed around piles. Most encapsulated systems have been installed on marine piles in tidal zones (Sohanghpurwala, 2009). On the other hand, non-encapsulated systems comprise: perforated sheets, expanded mesh, and bulk anodes. In some instances, the perforated zinc or the expanded mesh anode is sandwiched between fibreglass or a recycled material panel and the concrete of the pile and held in place by compression bands. The bulk anode is generally installed under water to supplement the protection provided by any of the other zinc anode

configurations used on marine piles. Non-encapsulated systems have been widely used in the marine environment. The most commonly used SACP anode systems in RC structures are discussed below:

3.5.1. Thermal sprayed zinc systems

Thermal sprayed zinc systems have been widely used to protect RC structures that are exposed to marine environments. However, Sohanghpurwala and Scannell (1994) and Sohanghpurwala (2009) report that they cannot be used to protect marine structures that are located within the tidal zone. Most thermally sprayed zinc systems comprise thin layers of zinc – ranging between 300 and 500 micrometres – that are applied on the surface of concrete through flame or electric arc spraying (Spriestersbach *et al*., 1999; Broomfield, 2007). The flame spray process uses a hand-held gun while the arc spray process uses a high-voltage direct current (DC) arc to melt the zinc wire (Funahashi and Young, 1998).

The performance of sprayed zinc anodes on concrete is greatly influenced by the presence of moisture at the anode-concrete interface (Funahashi and Young, 1998) and the bond at the interface of the concrete and the anode. Thus, prior to their installation, the concrete surface ought to be preheated, cleaned and roughened. Pre-heating removes existing residual moisture from the surface rim zone. This moisture ought to be removed because it weakens the adhesion between concrete and zinc. Likewise, cleaning and roughening enhances the mechanical adhesion between zinc and concrete (Sohanghpurwala and Scannell, 1994; Spriestersbach *et al*., 1999). After the concrete surface has been prepared, a thin layer of zinc may be applied directly to the cleaned steel in areas where damaged concrete has been removed and to the adjacent concrete surfaces. Also, the areas where damaged concrete was removed may be left unrepaired in the event that the structure is aesthetically acceptable after repair. Occasionally, humectants of hygroscopic salts may be used in concrete in dry locations. Humectants are substances that penetrate into concrete thereby resulting in an increase in humidity. An increase in humidity helps in maintaining a low electrical resistance in concrete; thus facilitating the flow of current between the anode and the reinforcing steel (Darowicki *et al*., 2003; Torres-Acosta *et al*., 2004; Broomfield, 2007; Glass *et al*., 2010; Sohanghpurwala, 2009; Funahashi and Young, 1998).

Studies carried out in the US by Spriestersbach *et al* (1999) have shown that thermally sprayed zinc anodes can last up to 20 years. This service life, however, can be increased through the application of an organic top coating on the arc sprayed zinc anode. This is due to the fact that organic top coatings help to reduce the effects of self corrosion which may take place when zinc is in direct contact with the atmosphere. An example of a thermally sprayed zinc anode is shown in Figure 3.7.

Figure 3.7: Application of thermal sprayed zinc SACP (Source: http://www.efcweb.org/Working+Parties/WP+11/WP+11+History.html accessed on 18th November 2011)

3.5.2. Thermal sprayed Aluminium/Zinc/Indium systems

These anode systems are made from an alloy of Aluminium (Al), Zinc (Zn) and Indium (In) with the composition of aluminium in the alloy being at least 80% in some cases. The Al/Zn/In system is a consumable anode; that is, it results in the loss of zinc and aluminium while in operation.

The Al/Zn/In alloy is usually used to avoid or minimise the passivity of the aluminium alloy in the high alkalinity environment provided by concrete. It also helps to provide a more negative potential that can drive current through concrete that has a high resistivity (Cramer *et al*., 2002; Daily, 2003; Sohanghpurwala, 2009). Nevertheless, Al/Zn/In anode systems are similar to thermal sprayed zinc systems in many aspects except that they use Indium, instead of humectants, to keep the anode active while operating in drier environments (Broomfield, 2007; Young *et al*., 2009). A study, such as the one conducted by Young *et al* (2009), led to the development of an anode that could absorb stresses that emanate from corrosion products. Thus, modified Al/Zn/In anodes can be used to prevent the formation of cracks on concrete overlays. The same study also reported that Al/Zn/In anode are capable of providing sufficient CP current to steel that has been embedded in chloride contaminated concrete.

Al/Zn/In anode systems are normally applied onto the concrete surfaces using electric arc spray. They are thin, with a recommended coating of 12 mils. Thus, based on predictive consumption rates, they can be expected to provide 10 to 15 years based on the consumption of 0.5 mils per year (Young *et al*., 2009).

3.5.3. Zinc sheets

Zinc sheets, also called zinc adhesive anodes or zinc hydrogel anodes, are normally applied onto the surface of concrete. They are a proprietary anode system that consist of rolls of zinc whose dimensions are 0.25 mm thick and 25 mm wide with a hydrophilic, electrolytic conductive pressure-sensitive gel adhesive that has been laminated on one surface (Cramer *et al*., 2002; Raupach and Bruns, 2003; Broomfield, 2007). The adhesive is a 0.75 mm thick hygroscopic acrylate polymer that contains sulphonic acid and is charged with lithium chloride to form the electrolyte. The hydrophilic properties of the hydrogel layer help to prevent dehydration under low humidity environmental conditions. However, over time, the adhesive tends to dry out – thus limiting the ability of the zinc to provide CP (USDoT, 2001; Broomfield, 2007).

No special equipment or engineering skills are required for its installation. A liner is simply removed from the backing and the laminate is thereafter pressed onto the concrete surface. The surface of the concrete ought to be relatively smooth, clean and free of isolating layers before the anode is applied. In addition, the installed system can be painted. However, its edges must be sealed with silicon caulking (or any other convenient sealing material) against humidity exchange with the environment or water ingress. The ingress of water/humidity exchange with the environment may cause the gel to swell and/or leak, dissolve or dry out; thus leading to the loss of its ability to provide the bond and the much needed cathodic protection (USDoT, 2001; Raupach and Bruns, 2003). Nevertheless, these anodes are simple and easy to install. They also have a high initial current density. However, their long term durability is a concern, especially in wet conditions where adhesion of the hydrogel can be affected. To overcome this problem, sealing of the joint areas and perimeter of the sheeting is recommended to prevent moisture ingress.

3.5.4. Expanded zinc mesh

These systems consist of an expanded zinc mesh in a glass-reinforced plastic permanent form filled with a proprietary cementitious grout. Expanded zinc mesh anodes are usually applied to marine exposed piles in the splash and tidal zone of RC structures (Broomfield, 2007; Roberge, 2008). A typical example of a zinc mesh is the integral pile jacket which is designed to protect the tidal and splash zones of bridge pilings. It consists of snap-together fibreglass jackets with expanded zinc mesh which have been fastened together to the inside face of the jacket assembly. The annular space between the jacket and the pile is then filled with a cementitious grout. Since the system is pre-assembled, installation is quite simple. An expanded zinc mesh jacket is shown in Figure 3.8.

Figure 3.8: Sacrificial zinc jacket around a concrete pile (Source: http://www.pavementpreservation.org/wp-content/uploads/presentations/FDOT%20Cathodic%20Protection%20Practices%20in%20florida.pdf accessed on 22nd December 2011)

3.5.5. Compact discrete zinc disks and probe anodes

Discrete zinc disks (also known as 'point anodes' or 'hockey puck' anodes) have been widely used in patch repairs in buildings; and in the mitigation of corrosion in repaired bridge deck spalls and patches in inland as well as in marine substructure components (Dugarte *et al*., 2007; Roberge, 2008; Sohanghpurwala, 2009). These anodes can also be placed on a grid pattern in sound concrete to provide distributed protection. Nevertheless, these anodes are normally intended to provide corrosion protection for the top mat of reinforcing steel.

Common discrete zinc disks comprise zinc plugs/disks put in a cylinder of a proprietary high alkaline mortar for installation in cored holes in the concrete. They are usually installed in a manner that ensures that they are in contact with a purpose designed backfill in cavities within the concrete (Roberge, 2008; Glass *et al*., 2010). Figure 3.9 shows a typical zinc disk/plug. Small cylindrical zinc anodes encased in an enhanced cementitious mortar have also been installed in drilled holes for corrosion control in chloride contaminated concrete. They have a tie wire attached to fasten them to the reinforcing in repair areas. These anodes are wire tied to the exposed steel rebar around the perimeter of the repair at appropriate spacing/intervals. The spacing/interval between these anodes depends on the reinforcing steel density, concrete resistivity and chloride concentration of the concrete at the rebar.

Figure 3.9: Cylindrical zinc disk galvanic anode (Roberge, 2008)

Probe anodes come in two forms. The first type consists of proprietary anodes approximately 65 mm diameter by 30 mm high with four wires protruding for attachment to exposed steel in a concrete patch. They are usually used in patch repairs whereby they are installed at a maximum spacing of 750 mm around a patch. The second type consists of probe anodes in cored holes. These are potted up zinc anodes whose dimensions are approximately 45 mm diameter by 40 or 60 mm long. They are installed on 300 mm to 650 mm spacing in 50 mm diameter cored holes (Broomfield, 2007).

3.5.6. Bulk anodes, wires, ribbons and strips

Bulk anodes, anodes consist of blocks of cast zinc and aluminium. They are usually submerged adjacent to the concrete pilings of marine structures. They have been used successfully to cathodically protect portions of RC structures below the mean low water level and portions of the tidal zone. Wires and strip anode systems are characterised by a set of wires or strips placed in holes or slots and backfilled with a cementitious grout (Polder, 2005).

Galvanic anodes in the form of ribbons and strips (usually referred to as distributed anodes) have been used to provide cathodic protection to reinforced concrete. They are usually provided in various shapes and lengths up to 2.3m. Distributed anodes are placed across the surface of the concrete to be protected and thereafter embedded in a reinforced concrete jacket or overlay. Distributed anode systems are used to provide galvanic protection over a large area. However, they can be used to protect small structural elements such as a single pile, column or wall (Ball and Whitmore, 2009).

3.5.7. Conductive electro-active mesh and overlay systems

These systems consist of a conductive electro-active mesh which has been shaped to fit the surface of the structure. The mesh is usually covered with a cementitious overlay (Polder, 2005; Ball and Whitmore, 2009). Conductive overlay systems are widely used for bridge decks without waterproofing membranes. The Concrete Society (1989) has, however, reported that these systems are inappropriate for vertical soffit surfaces.

A summary of common SACP systems and configurations is given in Table 3.1.

Table 3.1: Common types of anodes used in SACP systems (Broomfield, 2007)

Anode	Environment	Application	Durability life	Comments
Thermal sprayed zinc	Marine or anywhere (with humectants)	Requires bulky spray equipment and skilled operator	Greater than 10 year life expected. Reduces with severe exposure	Colour change to concrete is the only effect
Thermal sprayed Al/Zn/In	Marine and de-icing	Requires bulky spray equipment and skilled operator	10-15 year marine exposure. 15-20 year in northern de-icing	Colour change to concrete is the only effect
Adhesive zinc sheet	Not for very high wetting, will work in very dry	No special skills apart from soldering connections conditions	Design life is 25-50 years. Gel deteriorates in very wet conditions	Anode is 1 – 2 mm thick. Either leaves zinc metal finish or painted metal finish
Encasement jacket	Marine only. Mainly columns and piles	Special skills required for grouting up jacket	Very durable life up to 5 year	Pile column section enlarged by 25 mm or more. Load increase. Repair can be structural
Probe anodes in cored holes	Anywhere	No special skills	15-25 year life. Very durable	Core holes at 330 to 650 mm centres. Requires good design
Probe hole in patch repairs	Any patch repairs	No special skills	10-15 year life. Very durable	Only area around patch repairs protected

3.6. Factors that influence the effectiveness of SACP systems in RC structures

The effectiveness of any SACP system depends on the properties of the concrete and anode material (intrinsic factors) as well as the interaction between the anode-concrete system and external environment (extrinsic factors). Thus, to ensure that SACP systems perform effectively, the complex interactions that exist between the electrochemical properties of the

anode as well as the environmental factors that affect the anodes ought to be understood. The main factors that influence the effectiveness of SACP systems are discussed below.

3.6.1. Electrochemical properties of the anode material

The electrochemical properties of an anode material dictate its ability to provide cathodic protection to steel in concrete (Funahashi and Young, 1998; USDoT, 2001). The main electrochemical properties that define the effectiveness of an anode are its potential and the magnitude of anodic polarisation. Polarisation refers to the change in the effective potential of the anode (or cathode). It results from the circulation of current between the anode and the cathode. The magnitude of polarisation for the same level of current delivery differs from one material to the other. In addition, the polarisation of the anode governs the level of current delivery above which harmful by-products such as chlorine gas are generated. Such by-products can result in damage of the concrete adjacent to the anode and/or the anode itself (USDoT, 2001). Essentially, an anode is expected to provide an adequate current density that can polarise the steel sufficiently to levels where it will either corrode at an acceptable rate in a cost-effective manner or not corrode at all (Pedeferri, 1996; Funahashi and Young, 1998; Bertolini *et al*., 2004a; Sohanghpurwala, 2009). An anode's ability to polarise steel depends, to a large extent, on the electrochemical properties of its material(s) and the surrounding environment.

Other electrochemical parameters that dictate the efficiency with which an anode material operates include: its electrochemical equivalent and efficiency (Roberge, 2008), its throwing power (Sekar *et al*., 2007) and its electrochemical age/aging rate (Cramer *et al*., 2002). The efficiency of a galvanic anode material refers to the ratio of an anode weight sacrificed for CP purposes divided by the total theoretical ampere hours or capacity of the actual material consumed. Thus, an efficient anode material is one that generates a lot of current while undergoing minimal consumption. The throwing power of an anode is the maximum distance that can be reached by the cathodic protection current. It determines whether points further from the anode will receive cathodic protection or not. In addition, the throwing power depends on the anode as well as concrete properties. The anode aging rate is directly proportional to the anode current density and the operating conditions such as temperature and the presence of rainfall (Cramer *et al*., 2002; Holmes *et al.,* 2011). Thus, an anode that delivers a high current density ages at a faster rate compared to one that delivers a low current density. The electrochemical age also affects the anode bond strength.

The type of products formed when an anode operates within a concrete medium/environment has been reported to affect the efficiency with which an anode operates (Holmes *et al.,* 2011; Schwarz *et al.*, 2011). These products, for example, have also been postulated to determine the response of an anode to external perturbations such as temperature fluctuation. Similarly, the deposition and agglomeration of zinc hydroxide and zinc hydroxychlorides or contact with the calcium hydroxide in the pore solution may passivate the surface of a zinc anode. Thus, SACP systems ought to incorporate materials that are inert.

3.6.2. Location, size and orientation of the anode

Anodes used in SACP systems are usually placed on the surface of concrete or embedded in RC structures in a way that ensures that uniform distribution of current is achieved. Thus, the thickness of the anode material is important to its installation and operation. Anodes which are very thin fail to achieve their desired service life while those that are thick lead to material wastage (Covino *et al*., 2002). Therefore, an adequate amount of anode material that can ensure that optimal protection is achieved throughout the design life of the structure ought to be provided. While the location of an anode helps in achieving a sufficient and durable protection (de Rincón *et al*., 1992, Bertolini *et al*., 2004a; Raupach and Bruns, 2003); the orientation of a sacrificial anode contributes towards its effective performance. Certain anode systems, titanium for example, perform exceptionally well on horizontal surfaces – i.e., where the anode mesh can be encapsulated in a concrete overlay.

3.6.3. The conductivity of concrete and repair materials

The conductivity of concrete dictates the effectiveness with which SACP systems operate. This is due to the fact that conductivity influences the way current flows through concrete. Conductive concrete enhances the ease with which ions move within it. Thus, concrete with a high conductivity is capable of offering an effective protection. The conductivity of concrete is affected by: temperature, moisture content and amount of chloride ions and the water to binder (w/b) ratio (Talavera *et al*., 2002, Daily, 2003; Parthiban *et al*., 2008a; Sohanghpurwala, 2009). Specifically, Schwarz (2003), Torres-Acosta *et al* (2004) and Schwarz *et al* (2011) reported that hydrogel polymers disintegrate when exposed to high humidity environments. This is due to the fact that the high humidity, coupled with the ingress of liquid water, leads to the expansion of the hydrogel and the subsequent delamination of the anodes. Thus, these anodes are not ideal for structures that are exposed to environments with high humidity. Despite the fact that carbon fibres increase the conductivity of concrete (Hou and Chung, 1997), information regarding their use in sacrificial anode cathodic protection has not been obtained from a review of available literature. In addition, the code of practice used for cathodic protection of concrete, BS EN 12696:2000, has prohibited the use of such materials in CP systems.

The resistivity of concrete (Sekar *et al*., 2007; de Rincón *et al*., 2008; Polder, 2001) affects the performance of SACP systems. Concrete of particularly low resistivity creates a local high current demand, or 'current' dumping'- a phenomenon that affects the performance of an SACP system (Concrete Society, 1989). Particularly, resistivity becomes a critical factor when submerged anodes are used to protect reinforcement above the water-line in maritime structures. Concrete is normally repaired with standard repair mortars or shotcrete prior to surface preparation and thermal spray application. Thus, repair materials that are used in conjunction with SACP systems ought to be cementitious in nature as well as have resistivity values that are less than 50kΩ-cm. Most normal portland cement mortars and concrete meet this criterion. If higher resistance materials are to be used, then the anodes can be embedded in a normal resistance grout prior to application of the repair material to provide a conductive

path between the anode and the reinforcing steel in the concrete adjacent to the patch (Ball and Whitmore, 2009). In addition, Daily (2003) reported that materials that provide a dielectric barrier to current flow, such as epoxy bonding agents, should be avoided.

3.6.4. Durability of the anode material

The durability of an anode refers to its ability to resist weathering while operating within the application environment (USDoT, 2001). The components of SACP systems' ought to be durable. That is, they have to realise their design life under ambient conditions (Chess, 1998b; Sohanghpurwala, 2009). In addition, they have to withstand mechanical wear from the forces that are impacted on them during their installation in structures, or when the structures are in operation. They should neither fail nor deteriorate to an unacceptable level, even under reasonably varying environmental conditions before their anticipated service life (Chess, 1998a; Bertolini *et al*., 2004a; Sohanghpurwala, 2009).

3.6.5. Concrete surface preparation and bonding between the anode and the concrete substrate

Concrete surface preparation is an important parameter that dictates the feasibility and effectiveness with which SACP systems work (Daily, 2003). It helps to provide a clean, sound, dust free surface of suitable roughness and exposure of aggregate for the various anodes or overlay types. Proper concrete surface preparation for SACP systems may involve the use of abrasive blasting equipment or a high pressure water jet. In addition, this process (concrete surface preparation) helps to ensure that an adequate bond exists between the concrete and the anode and/or its overlay. The nature of the bond that exists between an anode and the concrete substrate, and its associated bond strength, is crucial for the performance of surface-applied anodes and/or their associated overlays. While Covino *et al* (2002) reported that durable/long-lasting bonding, among many other factors, can guarantee adequate CP over a long time; Pedeferri (1996) and Bertolini *et al* (2004a) reported that poor bonding result in the loss in efficiency of cathodic protection. This loss in efficiency is attributed to the delamination of overlays that are used in SACP systems. Thus, the surface of concrete upon which surface-mounted anodes and their associated overlays are installed should be prepared in a manner that guarantees a good physical bond exists between concrete and the anode or its overlay (Highways Agency, 2002).

3.6.6. Monitoring system

Monitoring systems for SACP systems ought to give accurate and reliable information since the information that is gathered from them helps in decision making. This information, for example, can be used to determine the level of protection that has been achieved, check for problems in connection, as well as evaluate the effectiveness of CP. The location of the monitoring system should be considered. Bertolini *et al* (2004a), for example, reported that monitoring equipment, such as reference electrodes and monitoring probes, ought to be fixed into the most critical areas or where the control of the potential is most important. Furthermore, most monitoring systems ought to be sheltered from the vagaries of weather.

They should be installed in areas that are not prone to vandalism. The monitoring systems ought to be easy to operate. Complex systems tend to provide challenges, especially when the staff/people that are responsible for the maintenance are not highly trained with respect to their installation, troubleshooting and interpretation of results. Finally, these systems ought to be installed in areas that can be accessed easily by technical personnel.

3.6.7. Electrical continuity

A closed electrical circuit (unbroken electrical path) between all reinforcing steel is required for proper functioning of CP systems (Weyers and Cady, 1984; Scannell and Sohanghpurwala, 1993; Beamish and El-Belbol, 1998; Daily, 2003). This is due to the fact that electrical continuity ensures that protective current returns to its source. The risk of corrosion at a local anodic site can be increased in areas where the reinforcement is discontinuous or there is electrolytic conduction. Thus, it is important to ensure that the reinforcement is electrically continuous. In addition, discontinuous/isolated elements usually receive little or no protective current; thus they are prone to corrode from stray current. To provide a precaution against electrical discontinuity, The Highways Agency (2002) states that the spacing of reinforcement and the possibility of electrical short-circuits ought to be checked and corrected during installation of anodes. Finally, electrical continuity, in practice, can be provided by tie wires which are used to mechanically connect the steel reinforcing bars during construction (Weyers and Cady, 1984).

3.6.8. Miscellaneous

The ease with which SACP systems can be installed and the effects that they induce on other components of structures (e.g. additional weight) have an influence, though to a small extent, on the effectiveness of SACP systems. The installed system should, on completion, be aesthetically acceptable. Furthermore, it should be relatively easy to operate. Its design should adequately address inspection and maintenance requirements (Chess, 1998a and b). SACP systems must be structurally, mechanically, chemically and electrochemically compatible with the existing substrate in the RC structure being protected (Morgan, 1996; Vaysburd, 2006; Mailvaganam and Zhang, 2006; Vaysburd *et al*., 2009). The reinforcement content and the ratio between the steel reinforcement and the anode area play a critical role towards the effective performance of SACP systems. A study by Raupach and Bruns (2003), for example, reports that an increase in reinforcement content results in a decrease in protection current density as well as potential decay. Similarly, a study by Dugarte *et al* (2007) concluded that a high anode-to-steel placement density could ensure potentials that are necessary for effective cathodic protection/prevention.

Prior to the installation of SACP systems, areas that have loose overlays (i.e., where there is a loss of connection between concrete and the rebars) ought to be repaired with a repair mortar suitable for CP according to EN 12696. Similarly, cracks in concrete ought to be filled by injecting cement grout (Schwarz, 2010). The susceptibility of the anode material to self-pitting also ought to be considered. Schwarz *et al* (2011), for example, reports that self-

corrosion/self-pitting can reduce the service life of anodes. More specifically, he reports that the service time of a zinc anode may be limited by self pitting that increases with the activation of the zinc anode and may reach up to 70% of the zinc consumed during operation. The local weather conditions that prevail within the structure's environment ought to be factored. Holmes *et al* (2011), for example, report that the current passed by galvanic anodes on site and in laboratory installations fluctuates depending on the level of corrosion (e.g., a high corrosion risk will be accompanied by the generation of a high protection current output), local weather conditions and the time of the year.

3.7. Factors to consider when selecting an SACP system

SACP systems are designed to provide long-term corrosion control. On one hand, they have to satisfy the expectations of the owners and designers as well as guarantee cost-effectiveness, sustainability and aesthetics, among other factors, on the other hand. These 'conflicting expectations', thus, deems it necessary to consider a number of factors when selecting an SACP system. Such factors include:

3.7.1. Remaining service life of the structure and the design service life of the SACP system

The service life (or life expectancy) of an anode refers to the time it takes the anode to continue to provide the required CP current. All anodes are consumed while in operation. Thus, the rate at which this consumption takes place determines the service life of the anode. The theoretical upper limit of service life of metallic anodes, such as zinc, can be calculated using Faraday's law as well as the amount of material that has been used as an anode. However, the service life of other materials, such as conductive paint, requires complex calculations (USDoT, 2001).

Typical galvanic anodes have a service life that ranges between 10 to 30 years. The service life of anodes varies with the amount of current that is drawn while it is in operation (Schwarz, 2003), the type of by-products generated while in operation (USDoT, 2001), the type of material used to make it as well as the level of chloride contamination within the parent concrete. The service life of the anode to be installed as well as the remaining service life of the structure to be protected ought to be factored while selecting an anode. Structures that incorporate SACP systems earlier in their service life require anodes that can last longer. Thus, it would be inappropriate to install anodes that last long in structures whose remaining service life is short. The mode of application of SACP systems determines, though to a smaller extent, the service life/life expectancy of the system. Surface-applied systems, for example, have a shorter life expectancy in comparison with encapsulated systems. The reduced life expectancy is attributed to the fact that surface-applied systems face direct/mechanical contact with aggressive agents such as human beings as well as elements of weather. These interactions render them prone to disintegration.

3.7.2. Concrete surface preparation, reinforcement density and operator skills

Concrete surface preparation is a parameter of utmost consideration in surface-applied SACP systems. The type and quality of concrete surface preparation dictates the quality of bond that will be developed between the concrete substrate and the anode and/or its overlay (Bertolini *et al*., 2004a). Poor surface preparation may result in the development of a poor bond. A poor bond may cause the anode and/or overlay to debond; consequently resulting in a decrease in the efficiency of CP.

The reinforcement density within a structure affects the amount of protection current density delivered by the SACP system and the polarisation. The ratio between the reinforcement and the anode also ought to be considered. More specifically, a study by Raupach and Bruns (2003), for example, reported that reinforcement contents that have reinforcement to anode ratios that range between 0.5 and 1.0 can help to achieve good and efficient CP. Structures that have a high reinforcement density experience a reduced protection current density as well as a decrease in potential decay as explained in Section 3.6.8.

Surface-applied SACP systems require that adequate surface preparation as well as correct application procedures be undertaken. Thus, a good thermal spray operator (or nozzle man), for example, may help in ensuring that an SACP system is installed properly. Furthermore, Daily (2003) reports that SACP system operators should demonstrate that they posses adequate technical training and field experience to safely and proficiently apply anode coatings on concrete surfaces. In addition, it is usually recommended that thermal spray operators ought to be pre-qualified on concrete test panels prior to commencing field operations.

3.7.3. Environmental conditions

Galvanic systems, such as thermally sprayed Al/Zn/In anodes, are self-regulating. That is, the CP levels and current densities vary with time based on environmental conditions such as moisture and temperature. The protection current of most SACP systems is greatly influenced by the moisture at the anode/concrete interface (Funahashi and Young, 1998; Daily, 2003). This is due to the fact that fluctuations in moisture content result in a corresponding fluctuation in cathodic current delivery due to changes in concrete resistivity. Conversely, a decrease in moisture may result in a decrease in protection current to a point where protection levels are reduced. An increase in current delivery may also result in a decrease in the service life of the anode because the anode will be consumed at a high rate. In dry concrete conditions, current may drop off to a point where protection levels are reduced. Thus, efficient SACP will be realised in structures that are situated in places where moisture is present and relative humidity is high.

Temperature also affects the CP current output (Funahashi and Young, 1998; Daily, 2003). Daily (2003), for example, reports that the protection current may be reduced to negligible values in extremely cold climates, that is, below freezing. Some anode materials affect the

ecosystem. Zinc in sufficient quantities, for example, is known to be toxic to aquatic life. Thus, the use of thermally sprayed zinc systems in marine structures, for example, may require enclosures to contain zinc dust and rebound.

3.7.4. Type of structure and morphology

SACP systems must satisfy the functional as well as the aesthetic requirements of a structure. The type of reinforcement used in a structure, for example, may help while determining the best materials for anodes. While ICCP systems have been reported to polarise steel to hydrogen evolution potentials, SACP systems do not pose such challenges (Sohanghpurwala, 2009). Thus, SACP systems can be used in most buildings. However, alloys of metals that have the capacity to polarise steel to very negative values should be used with caution on prestressed concrete structures.

The morphology of the structure also ought to be considered. Bridge substructures, for example, consist of vertical and overhead surfaces as well as irregularly shaped surfaces. Thus, thermally sprayed anodes may be the most preferred anode type. Also, surface-applied systems are easier to apply to vertical and overhead surfaces as compared to encapsulated systems. Funahashi and Young (1998) have also reported that SACP systems have to fulfil the aesthetical requirements of a structure since most protected parts of structures are visible.

3.7.5. Type of material, costs, operation and maintenance

The type of material used to make the anode determines its service life and level of protection. Furthermore, Funahashi and Young (1998) reported that an appropriate material for use in SACP systems ought to be selected on the basis of its composition, potential (oxidation), current output, anode efficiency and polarisation characteristics with time. Also, different materials yield different CP output in different environments. Zinc, for example, has been reported to give high current under high temperature and humidity. Aluminium has also been reported to perform equally well in high and low temperature and humidity environments (Funahashi and Young, 1998). Other materials, e.g., zinc, have been reported to be unfriendly to the environment; thus, they may not be preferred for use in certain environments. Furthermore, a particular application may preclude the use of some of the available anode materials (Sohanghpurwala, 2009). The selection of an anode material and configuration should not impact the overall durability or the operating capacity of the structure. Anodes, for example, should not cause acid attack of the concrete or aggravate freeze-thaw damage. In addition, they should not add additional dead load which would reduce a structure's overall live load capacity or operating clearances.

The installation and life cycle costs of SACP systems ought to be factored in the selection of SACP systems. A good SACP system should be designed in a manner that ensures that optimal protection at the least cost possible is provided. In addition, the complexity of the system that is used to monitor an SACP system as well as its associated costs ought to be factored. The operation and maintenance requirements of any SACP dictates, to a large extent, the amount of resources that will be needed to ensure the proper functioning of SACP

system. Also, Sohanghpurwala (2009) reports that system maintenance helps in ensuring that SACP systems have a long service life. Operation and maintenance involves the following: monitoring the CP current, depolarization and half-cell potentials, checking for the presence of cracks, rust spots and loss of adhesion amongst other parameters.

3.7.7. Miscellaneous

Other factors that ought to be considered when selecting an SACP system include: design requirements, tidal variations (in the case of marine structures), maintainability, reliability and effectiveness of the protective measures, the risk of unintended side effects, the possibility of future installation of other control measures and any impact on the cost of other elements within the structure (Chess, 1998b; Smith and Virmani, 2000). The availability of qualified personnel who can monitor the day-to-day operations of an SACP system also ought to be factored. Furthermore, the methods of installation and the feasibility of incorporating the system into the repair scheme, the electrical continuity of the embedded steel as well as other costs associated with the installation (e.g., traffic control, enclosures, scaffolding, etc) should be factored. Finally, utmost consideration ought to be put on the sustainability of the SACP system.

3.8. Application of SACP systems to new RC structures

Cathodic prevention (CPrev) is an electrochemical technique of preventive maintenance of new RC structures that are expected to become affected by chloride contamination. It is applied in the same way as CP; however, it is applied when chloride ions have not reached the critical threshold at the reinforcement and the steel is still passive (Bertolini *et al*., 2002; Chaudhary, 2002; Highways Agency, 2002; Bertolini *et al*., 2004a; Bertolini *et al*., 2009).

Cathodic prevention relies on the principle that cathodic polarisation of steel results in an increase in the chloride threshold required to initiate corrosion. Thus, an increase in chloride threshold, due to cathodic polarisation of steel, prevents the initiation of corrosion throughout the entire service life of an RC structure (Bertolini *et al*., 2004a; Sekar *et al*., 2007; Bertolini and Redaeli, 2009). Cathodic reactions also lead to the production of alkalinity as discussed in Section 3.2. This alkalinity helps to avoid a decrease in local pH - a phenomenon which is involved with pitting corrosion. Moreover, cathodic prevention causes the migration of ions within concrete. Ion migration helps in reducing the concentration of chlorides in the vicinity of reinforcing steel (Presuel *et al*., 2002; Bertolini *et al*., 2009). Thus, cathodic prevention helps in extending the service life of RC structures because it prolongs the corrosion initiation phase. Figure 3.10 shows the principles and mechanisms on which cathodic prevention is based.

Zone A is the corrosion zone, above the pitting potential, where corrosion can initiate and propagate; Zone B is the imperfect passivity zone, where corrosion does not initiate but can propagate (as the potential moves from E_{pit} to E_{pro}, the corrosion rate decreases and in the lowest part of zone B it is greatly reduced); Zone C is the perfect passivity zone, where corrosion neither initiates nor propagates; Zones D and E are hydrogen evolution zones,

where hydrogen embrittlement of high strength steel can take place; in Zone E loss of adhesion between concrete and reinforcement can occur (Bertolini *et al*., 2004a; Bertolini *et al*., 2009).

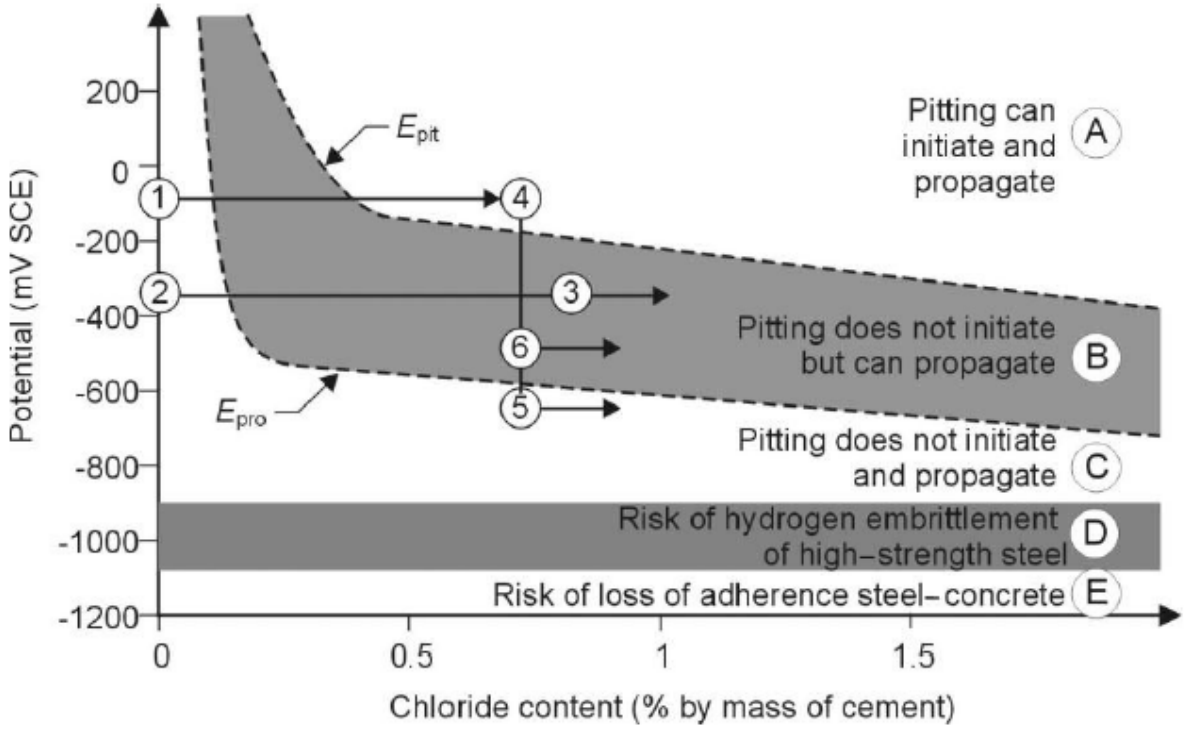


Figure 3.10: Steel potential versus chloride content in concrete (Bertolini *et al*., 2009)

Rebars embedded in concrete that is exposed to chloride-bearing environments usually follow path 1→4: as the chloride content in concrete increases they move from zone C towards zone A and, when the critical threshold is reached, pitting corrosion initiates. After corrosion has initiated, it can be stopped only by lowering the steel potential to values in or close to zone C, following path 4→5 or 4→6, i.e. applying CP, which is a typical repair technique for structures suffering chloride-induced corrosion. Cathodic prevention induces a decrease of steel potential (1→2); also in this case the chloride content in concrete increases in time due to chloride penetration (2→3), but the chloride threshold for the initiation of pitting corrosion (which is again represented by the ingress in zone A) is much higher. This translates into a longer time taken to reach corrosion conditions and in an increase in the service life of the RC structure (Bertolini *et al*., 2009).

Cathodic prevention differs from CP with regards to its throwing power. It has been shown that, in spite of the high resistivity of concrete, the beneficial effects of cathodic prevention can extend to rebars at remarkable distances from the anode. This is a consequence of the higher cathodic polarizability of passive steel compared with corroding steel (Bertolini *et al*., 2002; Sekar *et al*., 2007; Bertolini and Redaelli, 2009). Cathodic prevention is usually applied to the tidal zones of marine highway support structures or tunnels where the external ground water or estuarine conditions are saline. Finally, the Highways Agency (2002) has noted that cathodic prevention, particularly if installed at the time of construction, is likely to

be significantly lower in cost than cathodic protection installed during the service life of a structure.

3.9. Application of SACP systems to deteriorating RC structures

CP is mainly aimed at arresting corrosion while removing the hazards of spalling concrete rather than restoring strength. Thus, provided that the structure has sufficient reserves of strength, CP can, in many cases, be the most appropriate and cost-effective repair option (Chess, 1998a; Beamish and El-Belbol, 1998). The methods through which SACP systems are used to repair RC structures that have started to deteriorate are discussed below.

3.9.1. Concrete subjected to patch repairs

The fundamental objective of repairing deteriorating RC structures is to stop or reduce the rate of further corrosion of reinforcement. Conventionally, patch repair involves the removal of concrete that has been contaminated by chlorides around the reinforcing steel in areas where corrosion is detectable, cleaning of the exposed metal and reinstatement with fresh alkaline concrete or mortar (Cleland *et al*., 1997; Sergi and Page, 1999; Soleimani *et al*., 2010). While these actions address the immediate serviceability requirements, they do not always satisfy long-term durability needs. In fact, they lead to an increase in the driving voltage of the resulting concrete cell because of difference in chloride ion concentration, pH, moisture, and electrical conductivity between the repair patch and the old concrete. The increase in driving potential causes accelerated corrosion of the reinforcing reinforcement adjacent to the patch. Therefore, patch repairs that are limited to sections that have been damaged fail prematurely in chloride contaminated, and exposed, RC structures (Vaysburd and Emmons, 2004; ACI, 2005; Qian *et al*., 2006; Wang *et al*., 2006; Sergi, 2010).

The major challenge that has been encountered while using patch repairs is that unless stringent measures are taken to remove all chloride-contaminated concrete around corroding areas, corrosion may reappear in three areas, namely: the repaired area, the unrepaired concrete adjacent to the repair and at the interface between these two areas (Sergi and Page, 1999; Sagüés and Powers, 1996; Soleimani *et al*., 2010). The latter phenomenon is known as the ring anode (or incipient anode or ‘halo’) effect and its cause has been attributed to macrocell corrosion formed between the steel in the repaired patch (macrocell cathode) and the steel in the substrate (macrocell anode) (Qian *et al*., 2006; Soleimani *et al*., 2010). The mechanism through which incipient (or ‘ring’) anodes form is shown in Figure 3.11.

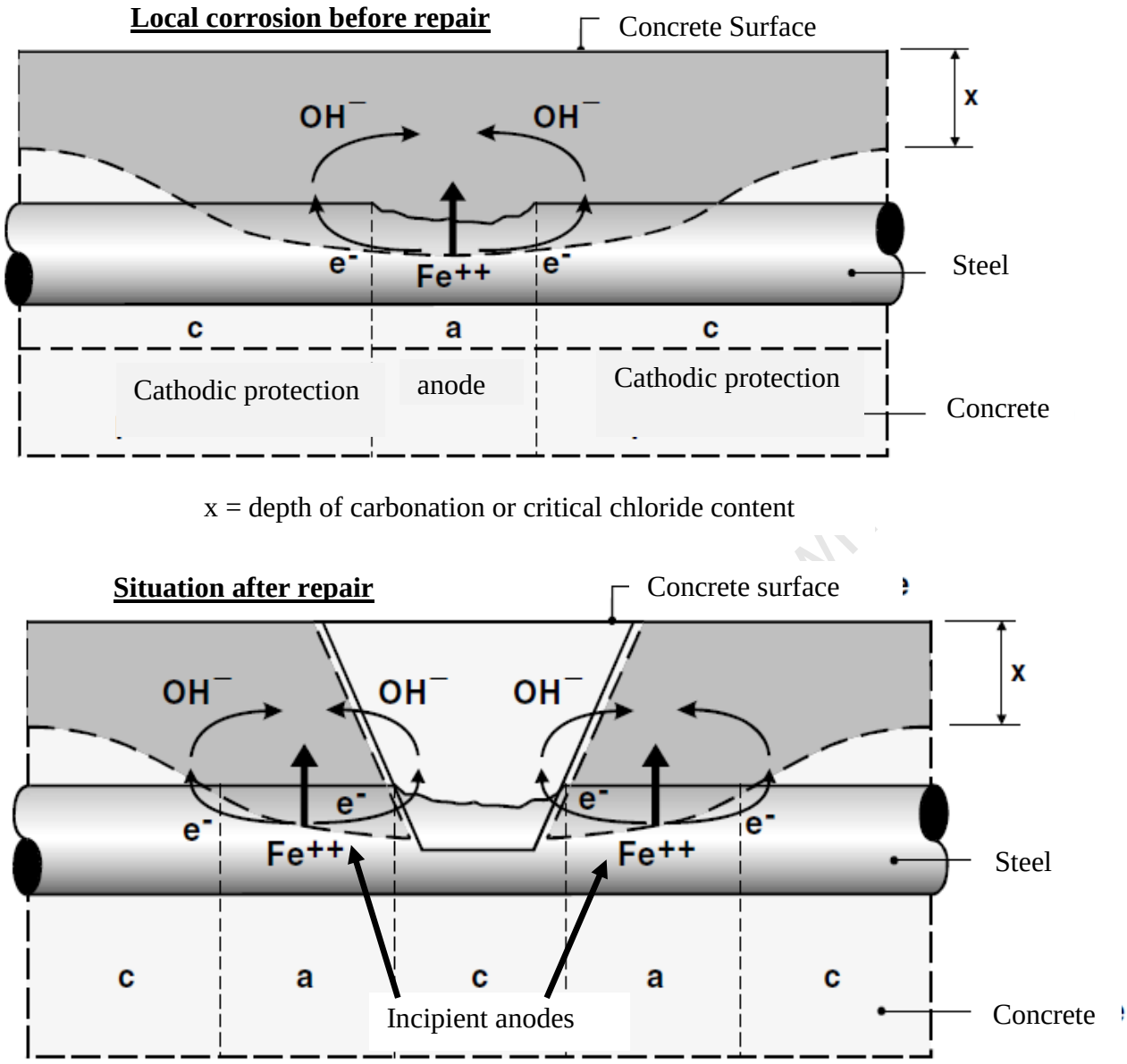
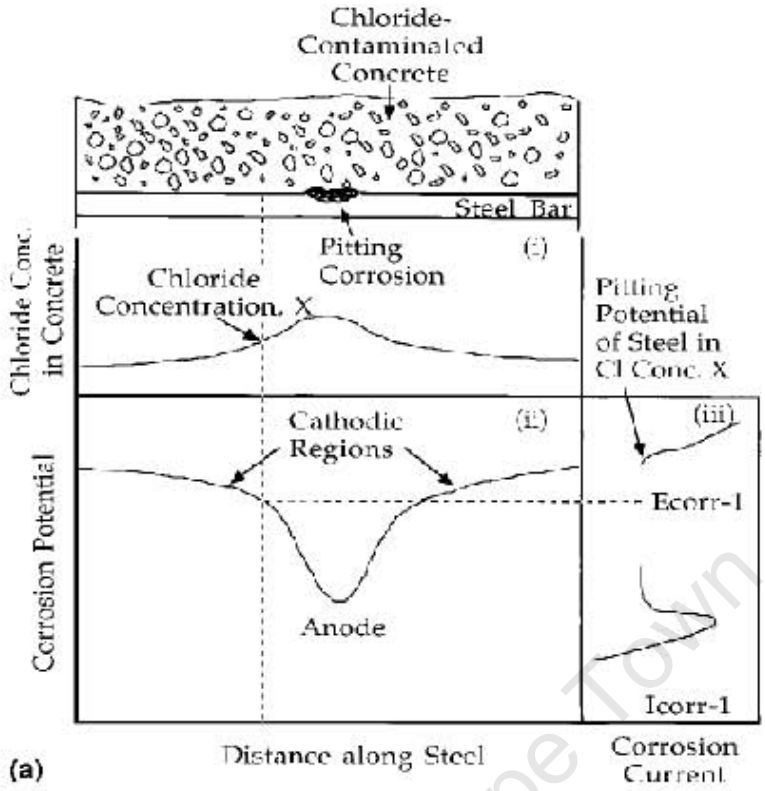
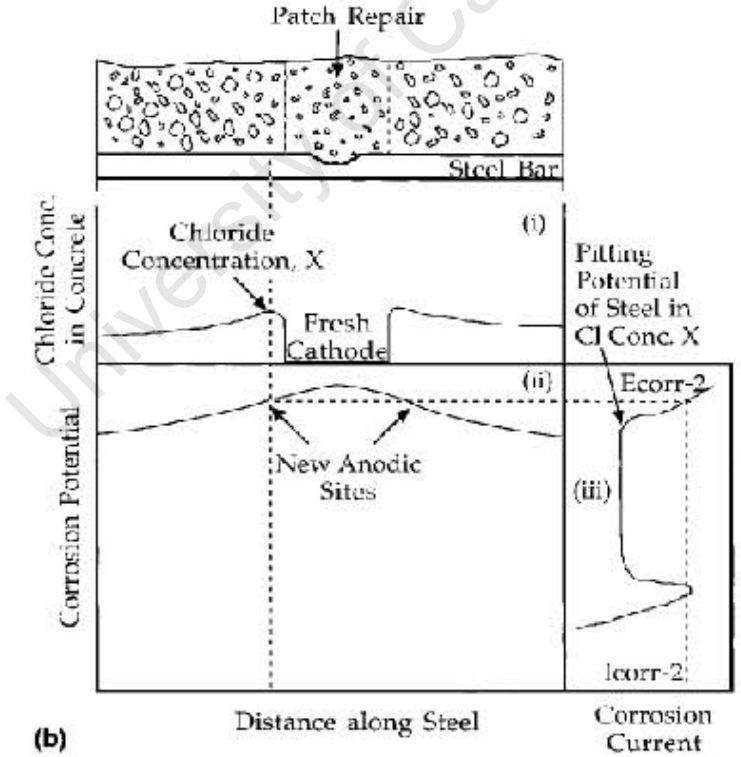


Figure 3.11: Corrosion mechanism due to macrocell action before and after patch repair (schematic) (adapted from Raupach, 2006)

Incipient anodes are attributed to the fact that the replacement of the most intensely anodic regions of the reinforcement with passive steel in the repaired zones effectively removes the adventitious form of SACP that was formerly being applied in the vicinity of steel (Figure 3.12a). Thus, a rise in steel potential is experienced in areas that are not contaminated severely. The rise in potential may approach a value that is liable to initiate pitting (Figure 3.12b) (Sergi and Page, 1999). This phenomenon is represented in Figure 3.12 (a-c).





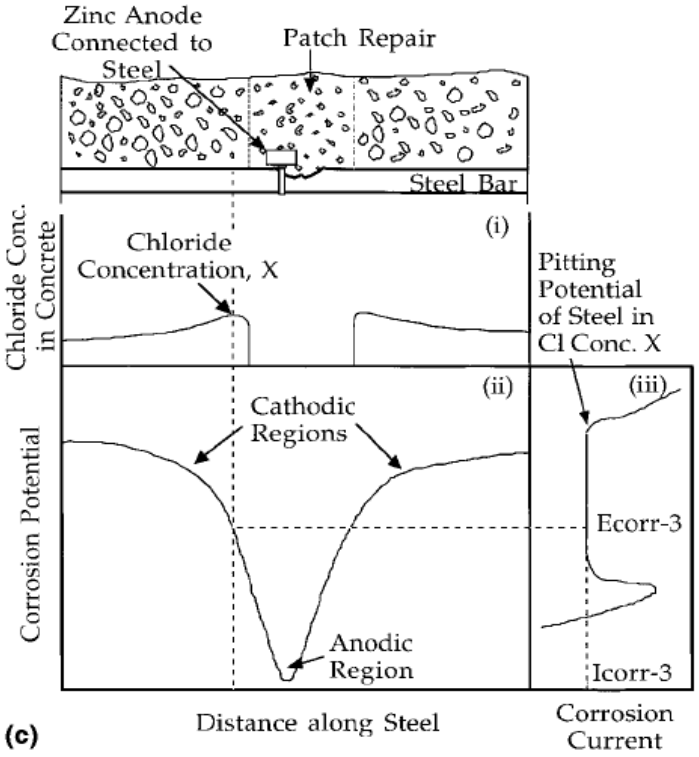


Figure 3.12: (a) Conditions for Localised Corrosion of Steel in Chloride-Contaminated Concrete. (b) Conditions for Incipient Anode Formation on Steel in "Patch-Repaired" Concrete. (c) Protection of Steel in "Patch-Repair "from Zinc Sacrificial Anode (Page and Sergi, 2000)

The problem of corrosion at incipient anodes can be avoided by the use of a form of 'intentional cathodic prevention' which is accomplished with sacrificial anodes of an appropriate design. This method is intended to combat the underlying corrosion rather than simply repairing the physical damage. These anodes are embedded near the periphery of the repair patches as shown in Figure 3.13 (Sergi and Page, 1999; Page and Segi, 2000; Glass, 2001; ACI, 2005; Whitmore and Ball, 2005; Sergi, 2010).

Embedded discrete anodes are installed around the perimeter of the concrete repair as close as possible to the patch edge where the anode units are pre-wetted and tied directly to the reinforcing steel (Ball and Whitmore, 2009). A form of sacrificial anode, that has been found to be successful under laboratory conditions, consists of zinc encased in a high alkalinity mortar (Sergi and Page, 1999; Dugarte *et al*., 2007; Christodoulou, 2009; Sergi, 2010). The mortar used to embed these anodes has an admixture that promotes high pH or otherwise activating the zinc. Also the mortar may contain humectants that may promote activity of the zinc (Dugarte *et al*., 2007). This system is illustrated schematically in Figure 3.13.

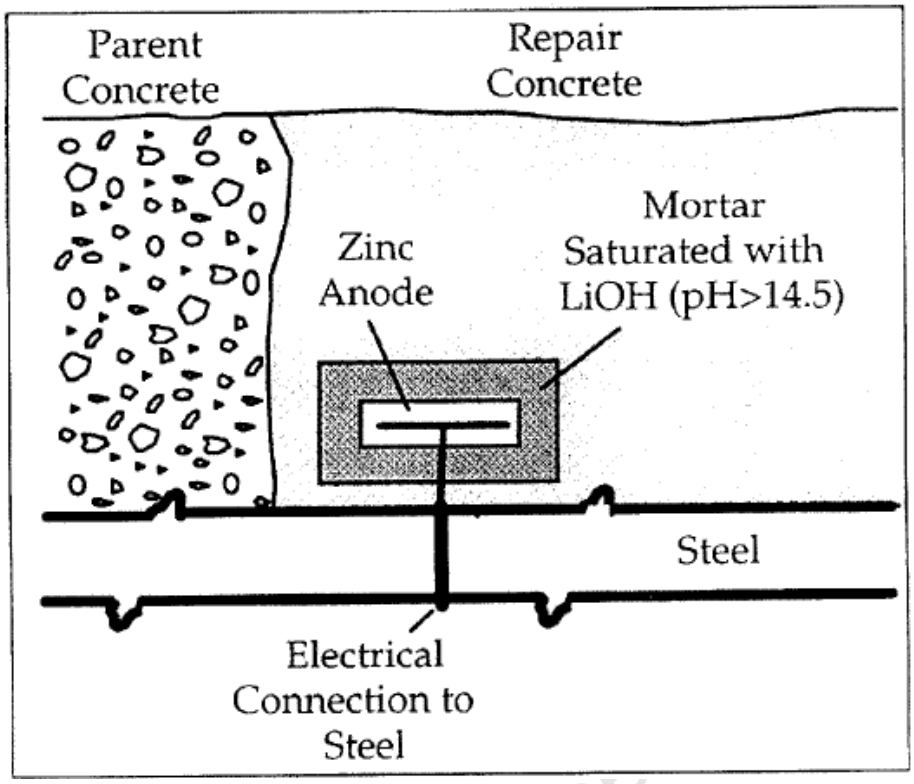


Figure 3.13: Zinc sacrificial anode contained in a specially formulated mortar and connected to the steel reinforcement in a patch repair (Sergi and Page, 1999)

The materials used for patch repairs ought to be electrochemically compatible with the existing concrete substrate (Gu *et al*., 1997; Vaysburd and Emmons, 2000; Qian *et al*., 2006). That is, they should be ionically conductive and, in order to achieve reasonably uniform current distribution; they should have resistivities of the same order as the parent concrete (Beamish and El-Belbol, 1998). Electrochemical compatibility of materials used for patch repairs incorporating sacrificial anodes is important because it determines the efficiency with which the anode will operate. Moreover, embedded galvanic anodes should be used in conjunction with cementitious or cementitious-polymer repair materials which have low resistivity. The resistivity of repair materials for use with embedded galvanic anodes should be less than 15kΩ-cm. High-resistivity materials such as epoxies or high polymer modified repair mortars greatly reduce the available galvanic current or prevent the anodes from functioning properly. If a low-resistivity material is not available for the full repair, anodes can be embedded in individual pockets of low-resistivity material. These pockets should completely encapsulate the anode and completely fill the space between the anode and the concrete substrate (ACI, 2005). Finally, materials used as bonding agents for patch materials ought to be used with caution. This is due to the fact that the bonding layer may prevent the passage of current to the reinforcement within the repaired zone (ACI, 2005; Broomfield, 2007; Beushausen and Alexander, 2009).

3.9.2. Other types of application

Sacrificial anodes can also be used to protect corroded epoxy coated steel reinforcement (Sagüés and Powers, 1996); or in expansion joint repairs, bridge widening, and galvanic encasement using concrete jackets and overlays (Ball and Whitmore, 2009). In the case of epoxy-coated reinforcement, an initial blasting ought to be done on the reinforcement steel. Blasting is important because it removes the polymer coat as well as the underlying corrosion products (Sagüés and Powers, 1996). The main challenges faced while applying CP to epoxy-coated reinforcement are establishing the continuity and the risk of undercutting corrosion behind the epoxy coating (Broomfield, 2007). Continuity in structures that incorporate SACP systems can be realised through the welding of extra bars. Multiple continuity systems can also be established by the use of sprayed zinc.

3.10. Operation and maintenance of SACP systems

SACP systems are designed and installed to remain in constant operation during their service life. Thus, effective operation and maintenance is vital for the continuous service that is required of sacrificial anodes in extending the service life of a structure.

The voltage, current and depolarisation of SACP systems must be routinely monitored, adjusted, and repaired as required; failure to do so may lead to lack of protection of reinforcing steel (ELTECH Research Corporation, 1993; ACI, 2005). However, it is important to note that the intervals and procedures for routine inspection and testing vary from one cathodic protection system to another depending on the structure, the cathodic protection system, the reliability of power supplies, the environment and the vulnerability to damage (Highways Agency, 2002).

In addition, sacrificial anodes should be visually inspected for cracks, rust spots, loss of adhesion, cable defects, etc., which should all be corrected or further investigated (Isecke, 1997; Bertolini *et al*., 2004a). General data such as the structure description – that is the location and name of the structure, weather condition, date and time of inspection; temperature and condition of the anode ought to be collected during maintenance. To evaluate the performance of an SACP system and make adjustments for optimum service, depolarisation tests, E-Log i tests, and electrical resistance measurements should be performed at start-up and thereafter annually (ELTECH Research Corporation, 1993).

The bond between the sacrificial anode and the concrete ought to be checked regularly (Covino *et al*., 2002). This is due to the fact that anode reactions result in materials which may interfere with the performance and efficiency of the anode and the bond between the anode and the concrete. Finally, inasmuch as the SACP is operating, it is always important to consider and establish health and safety practices that are appropriate to the specific circumstances that are associated with the use of SACP systems (ACI, 2005).

3.11. Complexities that influence CP system performance

The most important prerequisites that an SACP system design engineer ought to possess include: a proper understanding of the principles of CP, a general knowledge of electricity and electrochemical theory as well as the knowledge obtained from field experience with CP systems. In spite of the above prerequisites, the design of SACP systems in field structures offers complexities such as:

3.11.1. Exposure to a dynamic environment and variations in concrete cover

The environment under which SACP systems operate is dynamic. Consequently, there is a constant change in the circuit resistance between the sacrificial anode and the reinforcing steel. Localised wet conditions on large structures, for example, may influence the current output from an anode; thus resulting in an unbalanced current distribution. The presence of cracks that emanate from the concrete to the surface of the anode also contributes towards an unbalanced current distribution. Similarly, the depth of cover of most field structures is not uniform - it varies from location to location. Thus, localised high current discharge may occur in areas where the anode and reinforcing steel are extremely close. Also, uneven current distribution to the reinforcing steel as well as an electrolytic short circuit may develop in areas that have a low cover (less than 5 mm). This effect is significant in surface-applied SACP systems.

3.11.2. Variations in chloride contamination and reinforcing steel density

Structures that are/have been exposed to marine and/or de-icing salt environments are characterised by a spatial variation in the levels of chloride contamination. The variability in chloride concentration poses a unique challenge because a higher current density is required in steel that is exposed to higher chloride concentrations. The variability in the concentration of chloride presents a challenge during the design of SACP systems. In addition, the structural design of marine structures, such as jetties, intake structures and piers, are characterised with a complex reinforcing steel density. This complexity may result in shielding and uneven current distribution to the reinforcing steel. Thus, to ensure that uniform distribution of current is provided; a design engineer ought to consider the variations in steel density. Similarly, this complexity can be overcome by possessing an extensive field experience with theoretical background in current distribution modelling.

3.11.3. Miscellaneous

A low chloride concentration (< 1500ppm) and rebar density, the desire to protect one mat of steel as well as have a low maintenance regime are the some of the main parameters that present challenges to SACP system designers. The installation methods may also create a major challenge in anode selection. When a large amount of concrete repair is required, for example, an anode directly attached to the exposed reinforcing steel and installed in the repair or overlay may be preferable. Where access to the lower surface of a structural component is

limited, such as the soffit of a beam or pile cap in a marine jetty, a more appropriate approach may involve the installation of a discrete anode system in drilled holes.

3.12. Advantages and disadvantages of SACP systems

Associated with the use of SACP systems to control the corrosion of steel in RC structures are the following advantages and disadvantages:

3.12.1. Advantages of SACP systems

SACP systems are relatively inexpensive, easy to install, and in contrast to impressed current systems, they can be used in places where there is no power supply (Sergi and Page, 1999; Torres-Acosta *et al*., 2004; Gurrappa, 2005). The technique also has an added advantage of low monitoring requirements which is attributed to the fact that they do not need expensive electrical equipment (Spriestersbach *et al*., 1999; ASM, 2000; Roberge, 2008; Christodoulou, 2009; Young *et al*., 2009). Sacrificial anodes are suitable for small-scale applications, though they are also used extensively and with equal effect on large structures. This is due to the fact that they are capable of providing localised protection, i.e., they provide protective current to areas on the steel surface which need it the most (Scannel and Sohanghpurwala, 1993; Roberge, 2008). Moreover, sacrificial anodes have been used as an adjunct to conventional patch repairs; with small galvanic anodes being produced for patch repairs.

The low driving voltage of these systems makes them attractive for protecting prestressed structures (which are liable to hydrogen embrittlement); and epoxy coated reinforcement (where electrical continuity cannot be guaranteed and the consequences of discontinuity are small) (Daily, 2003; Parthiban *et al*., 2008a; Christodoulou, 2009). Since current used in these systems is self-regulated from the anode, SACP systems can be directly connected to the cathode without the need of any complex wiring (Christodoulou, 2009). Moreover, the direct connection to the reinforcement minimises electrical short-circuiting (Parthiban *et al*., 2008a). SACP systems have an inherent advantage of freedom from regular maintenance (Jordan and Page, 2003) and the fact that the protection current that is generated while operating cannot be supplied in the wrong direction.

Although the installation of SACP systems is capital intensive, its life-cycle costs, when compared with other corrosion mitigation systems, are generally lower (Sohanghpurwala, 2009). SACP systems help in saving costs during repair because they only require the removal of the spalled and delaminated concrete, while mechanically sound but chloride-contaminated concrete or carbonated concrete is left in place (ELTECH Research Corporation, 1993; Chess, 1998a; Naish and McKenzie, 1998; Polder, 2005; Schwarz, 2010). Consequently, less material has to be removed in comparison with conventional repair, causing less noise and dust to be produced and possibly resulting in shorter execution times. Moreover, cost reduction can be attributed to the fact that less concrete cross-sections are removed during repairs and the fact that temporary structural support (falsework) is required during concrete removal and repair (Chess, 1998a; Polder, 2005). The Highways Agency (2002) reports that the reduction of large scale concrete removal and reinstatement can result

in contract cost reductions of between 20% and 80%. From a structural mechanics and stress analysis viewpoint, the minimisation of concrete breakout reduces uncertainties over structural behaviour which results from the redistribution of stresses. SACP systems minimise stray current effects which arise from the presence of discontinuous steel (Christodoulou, 2009).

New anode types, such as the ones described by Bänziger *et al* (2009), have been developed. These anodes are not only capable of supporting loads and transmitting them to the surrounding mortar in a structure but also capable of being modelled to almost any shape. Finally, SACP systems help in the preservation of history and cultural heritage. This is due to the fact that SACP systems have been used to rehabilitate deteriorating antiquities as well as historic buildings.

3.12.2. Disadvantages of SACP systems

The passage of current through RC structures has no detrimental effects on the concrete itself; however, in the cathodic case, sodium and potassium hydroxides tend to accumulate at cathodic metal surfaces. A high concentration of sodium and potassium hydroxides near the surface of reinforcing steel reacts with calcium silicate hydrate binder in the cement system. These reactions lead to the production of soluble silicates which may result in softening of concrete that is adjacent to reinforcing steel (Hausmann, 1964; Weyers and Cady, 1984; Rasheeduzzafar *et al*., 1993). Moreover, the reactions that take place at the reinforcement surface result in a reduction in bond strength and durability (Rasheeduzzafar *et al*., 1993; Chess, 1998a; Chang, 2002; Koleva *et al*., 2006). Likewise, electrochemical reactions at the anode surface generate acid products and chlorine gas (chlorine evolution). Excessive evolution of acid products causes softening of the concrete surface which leads to failure of the anode system (Concrete Society, 1989; Chess, 1998a; Covino *et al*., 2002; Bertolini *et al*., 2004a). The evolution of chlorine gas may lead to the deterioration and disintegration of the anode, especially if the anode is in the form of a conductive paint. Though to a smaller extent, the chlorine gas may pose health problems to human beings.

Cathodic polarisation enhances the potential for alkali-silica reaction (ASR). This effect is pronounced when CP is applied over a long duration (Ali and Rasheeduzzafar, 1993; Pedeferri, 1996; Bertolini *et al*., 2004a). Cathodic polarisation generates hydroxide ions whose concentration in the pore fluid may rise to a level where silica in reactive aggregates, if present in concrete, could readily react. CP current also imparts brittleness to the reinforcing steel (Ali and Rasheeduzzafar, 1993). Moreover, improper choice and design of the anode material and anode, respectively, may result in undesirable consequences like cracking that result from the expansive oxidation products (de Rincón *et al*., 1992; Scannell and Sohanghpurwala, 1993). Despite the fact that these effects are associated with the passage of current within concrete, it is worthwhile to know that these effects are more pronounced in ICCP systems than in SACP systems. This is due to the low current that is associated with SACP. Thus, a considerable amount of time has to be taken before the effects described are realised.

The throwing power of commonly used sacrificial anode materials such as zinc, aluminium or magnesium alloys is very limited except in sea-water or very saline ground water condition (Chess 1998a). Thus, most SACP systems are limited to submerged RC structures. Similarly, the problem of limited throwing power is experienced in big RC structures whereby areas which are remote from sacrificial anodes end up receiving inadequate protection. Moreover, Hunkeler (2005) reported that SACP systems do not provide a means of increasing the current or voltage in cases where the protection is below the criterion, for example, 100 mV depolarisation criterion.

A major setback towards the use of SACP systems lies in the fact that their service life depends on the amount of sacrificial material provided and the current density (Sergi and Page, 1999; Gurrappa, 2005; Polder, 2005). Due to their limited service life, which results from the consumption of anodes, most sacrificial anodes need replacement at intervals that can be difficult to define (Broomfield, 2007; Roberge, 2008). Furthermore, sacrificial anodes applied directly to the concrete surface often exhibit adhesion problems (Glass *et al*., 2010). Most metal-activated sacrificial anodes undergo self/pitting corrosion in chloride environments. This phenomenon reduces the anodes' capacity, thus the efficiency of most anodes is always less than 100 percent (Kim *et al*., 2000; Roberge, 2008). It is also difficult to achieve a uniform current while using sacrificial anode systems to protect of corroding reinforcement. This is due to the high electrical resistivity of concrete, the small distance between anode and reinforcement, the low polarizability of corroding steel and the complex geometry of reinforcement (Scannell and Sohanghpurwala, 1993; Al-Idi and Al-Mehthel, 1995; Sohanghpurwala, 2009; Glass *et al*., 2010). Moreover, SACP systems do not work in dry concrete (Broomfield, 2007).

The simplicity offered by SACP systems means that there is no way of controlling them to ensure complete corrosion control (Gurrappa, 2005; Broomfield, 2007; Roberge, 2008). The driving voltage of sacrificial anode systems is low; thus, sacrificial anodes are ineffective when used in structures already subjected to pitting corrosion or those that are highly-contaminated with chlorides (Concrete Society, 1989; Brousseau and Pye, 1997; Broomfield, 2007). SACP requires good electrical continuity of the reinforcement. Thus, fusion bonded epoxy coated reinforcement is difficult to cathodically protect unless the reinforcement cage has been made electrically continuous prior to casting. Any other organic coatings would have similar problems (Highways Agency, 2002).

The initial cost of SACP (in comparison with other commonly used methods like corrosion inhibitors, patch repairs etc) is normally high (Al-Idi and Al-Mehthel, 1995). CP systems for garage-type structures are not only difficult to install but also lead to an increase in dead loading and reduced clearances (Al-Idi and Al-Mehthel, 1995). Moreover, conductive overlay systems have been reported to impose a substantial dead weight and load penalty on bridge decks (Concrete Society, 1989). Finally, it has been reported by Broomfield (2007), that the automatic flow of information regarding the performance of SACP systems is not available in the field.

3.13. Summary

The chapter discusses SACP systems critically. Specifically, the principles of cathodic protection, common sacrificial anode materials and system configurations, factors that influence the effectiveness of SAC systems as well as their advantages and disadvantages have been presented.

Sacrificial anodes rely on the cathodic polarisation of steel. This polarisation hinders the anodic dissolution of steel by moving the potential of steel into the range of passivity. Common sacrificial anodes are manufactured from Zinc, Aluminium, Magnesium, or their alloys. SACP system configurations comprise thermal sprayed Zinc and Aluminium/Zinc/Indium systems, zinc sheets, expanded zinc mesh, conductive electro-active mesh and overlays, bulk anodes and compact zinc anodes.

The effectiveness of SACP systems is influenced by: the electrochemical properties of the anode material, the conductivity of the concrete and the repair materials, the durability of the anode material, the concrete surface preparation and the bonding between the anode and the concrete substrate, as well as the location, size and orientation of the anode. The factors that ought to be considered when selecting an SACP system include: the remaining service life of the structure, the design service life of the SACP system, environmental conditions, the concrete surface preparation, reinforcement density and operator skills, the type of structure and morphology and operation and maintenance. SACP systems can be used to protect new RC structures (cathodic prevention) or structures that are corroding (cathodic protection).

From the review of available literature, it is clear that previous studies have failed to provide information regarding the performance of SACP systems with respect to parameters such as binder types, water/binder ratios, binder proportions and corrosion rates. Also, it can be inferred that a lot of work ought to be done with respect to the effectiveness of various SACP systems as well as investigations into their long-term performance. Finally, the incorporation of SACP systems into service life models has not been attempted.

CHAPTER 4: EXPERIMENTAL METHODOLOGY

4.1. Aims of research

This study evaluates the effectiveness of discrete sacrificial anodes as a potential concrete repair technique for use in South Africa. The main objective of this study is to investigate the effect of binder type, the level of chloride contamination and corrosion rate on the performance of a locally available discrete SACP system.

4.2. Testing philosophy

The extent to which SACP systems can be adopted in South Africa for use to repair structures that are deteriorating as a result of rebar corrosion depends, to a large extent, on their effectiveness with respect to binder type, the level of chloride contamination, and corrosion rate. Two binder types, PC CEM 1, 42.5N (hereafter referred to as PC) and a 70/30 blend of PC CEM 1, 42.5N and FA, were used to cast eight RC slabs. These binders were chosen because they are among the most common types of binders that are used for construction in South Africa. The cover to the outer reinforcement in each of the eight slabs was 20 mm. Each slab was divided into two equal sub-sections. One of the sub-sections (50% of each slab) was admixed with 1.0% sodium chloride (i.e. 0.6% chloride) by mass of binder during casting, and the other half with 3.0% sodium chloride (i.e. 1.8% chloride). The rationale behind admixing 0.6% and 1.8% chlorides, on a macroscopic scale, was the creation of an artificial macro-element (also called galvanic corrosion cell) in each slab as well as to accelerate the corrosion process (Fang *et al*., 2004; Poursaee and Hansson, 2009). The quantities of chlorides that were admixed in concrete during casting were also used to evaluate the effectiveness of the discrete sacrificial anodes with respect to the concentration of admixed chlorides.

The deterioration of RC structures as a result of rebar corrosion requires urgent attention. According to international literature, most structures that are deteriorating exhibit corrosion rate levels that can be classified as 'low to moderate' ($0.1\mu A/cm^2 \leq$ corrosion rate $\geq 0.5\mu A/cm^2$), 'moderate to high' ($0.5\mu A/cm^2 \leq$ corrosion rate $\geq 1\mu A/cm^2$) and 'high' (corrosion rate $\geq 1\mu A/cm^2$) (Gu *et al*., 2001; Grantham, 2003; Broomfield, 2007). In RC structures, the most critical levels of corrosion rate that require urgent attention are 'moderate to high' and 'high' corrosion rate. Thus, the effectiveness of discrete sacrificial anodes was investigated with respect to the two critical corrosion rate levels. Corrosion rates of $0.75\mu A/cm^2$ (i.e., an average of $0.5\mu A/cm^2$ and $1.0\ \mu A/cm^2$) and $2.2\mu A/cm^2$ were considered for investigation.

Corrosion was induced and accelerated in the slabs using direct anodic current (DC). Two direct anodic currents - 14.14µA and 41.48µA - were used to induce and accelerate corrosion. The applied currents of 14.14µA and 41.48µA were intended to induce corrosion rate values that correspond to 'moderate-to-high corrosion' conditions and 'high corrosion' conditions respectively (see the corrosion rate calculations in Appendix B.1). The preference of initiating and accelerating corrosion through the use of a direct anodic current (DC) over

natural corrosion is based on the fact that natural corrosion of steel in concrete is a slow process that takes a long time to initiate and progress - even in the case of severe corrosive exposure conditions. Thus, unless corrosion is either induced or accelerated, using DC or otherwise, it may be difficult to achieve a significant degree of corrosion in a limited duration available for performing research studies (Ahmad, 2009). Moreover, Poursaee and Hansson (2009) reported that the use of DC to induce corrosion is advantageous because it can help in achieving a high degree of corrosion within a short period as well as enhance the ease with which the desired degree of corrosion can be controlled. Manual measurements of concrete resistivity, temperature and moisture content, corrosion rate, and half-cell potentials were made. A commercially available discrete sacrificial anode (Sika Galvashield CC65) was used in this study. The data sheets of this anode are given in Appendix F.

This study comprised four control slabs and four test slabs. Sacrificial anodes were not installed in the control slabs. The slabs were exposed to an outdoor environment in Cape Town, South Africa over a five-month period (October 2011-March 2012). Three ambient temperature and relative humidity measurements (RH) were taken alongside corrosion rate, half-cell potential and concrete resistivity measurements at three o'clock in the afternoon. The average temperature and RH during the entire period of study were measured as 21.5^{o}C and 53.7% respectively. Variations in temperature and relative humidity are shown in Figure 4.1 and Figure 4.2 respectively. The measured data are presented in Appendix D (Table D.1).

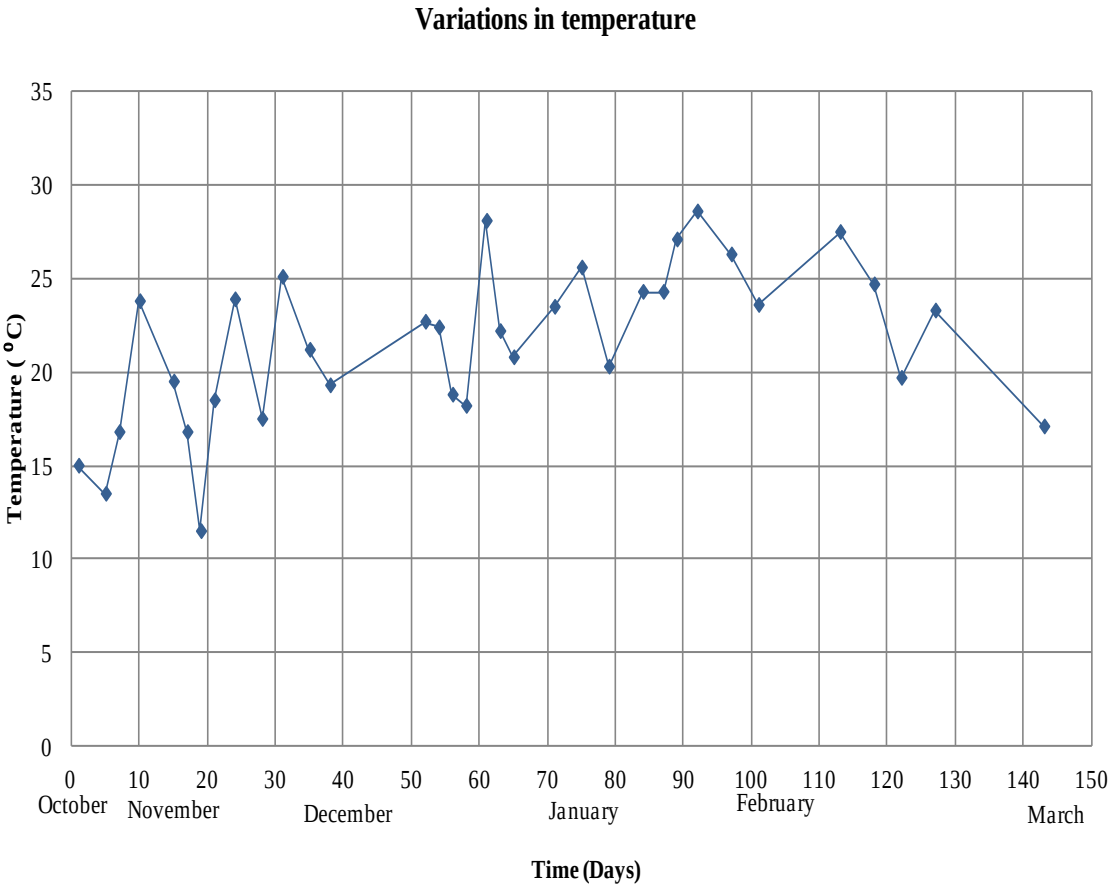


Figure 4.1: Variations in temperature during the study period

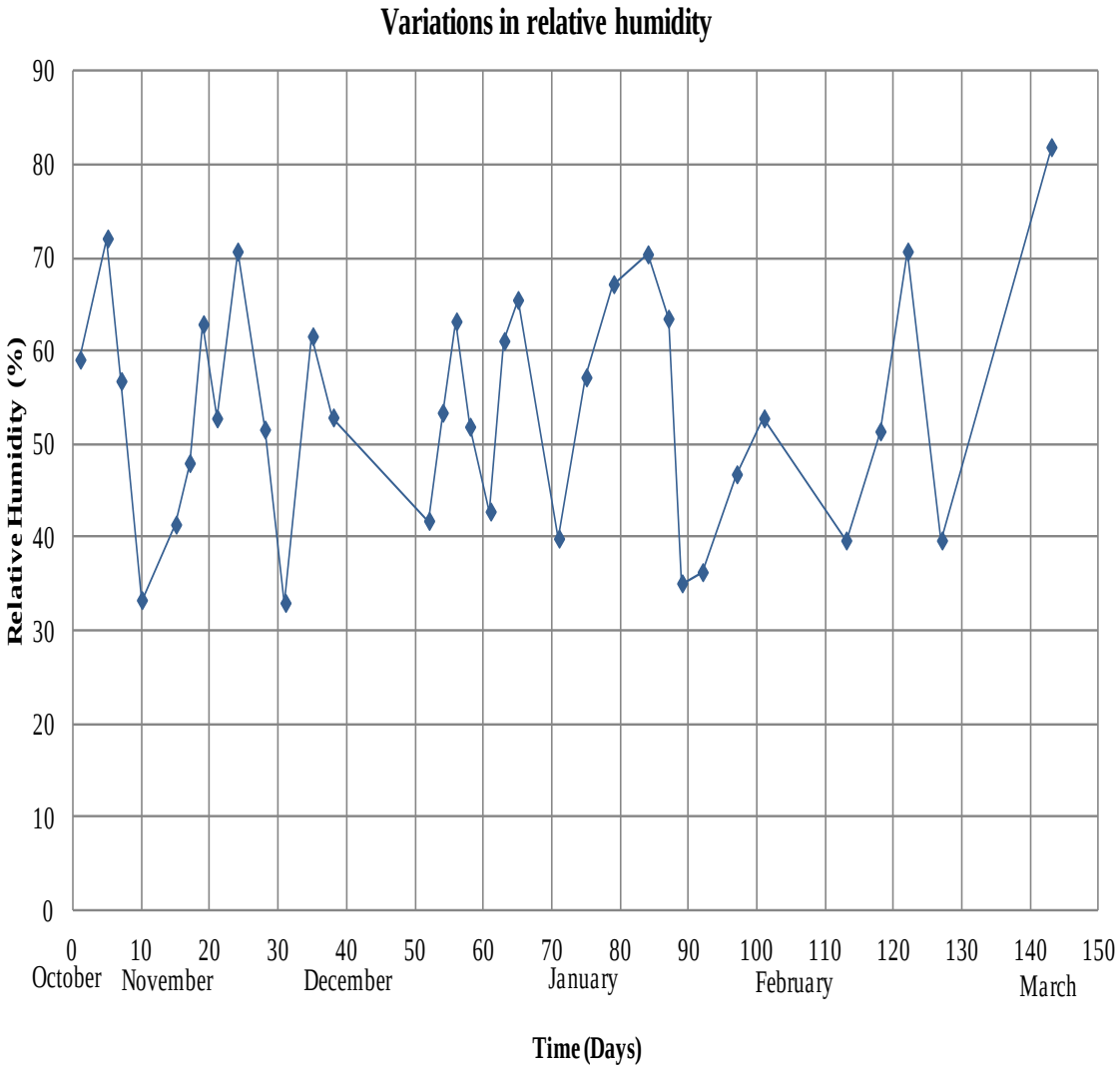


Figure 4.2: Variations in relative humidity during the study period

4.3. Specimen types and sizes

The RC slabs were made using the materials described in Section 4.2. High yield reinforcing steel bars - 10 mm diameter and at a spacing of 100 mm centres in each direction - were embedded in each of the 8 No. 1500 x 1000 x 100 mm thick slabs (Figure 4.3). The adopted specimen sizes and configuration of reinforcement was based on the technical advice that was given by Dr. Wolfgang Schwarz of Composite Anode Systems as well as the personnel from the technical department of Sika. The slabs were initially designed for a surface-applied SACP system. However, the designed SACP system was not delivered. Discrete sacrificial anodes were used eventually. The division of the slab into two sub-sections – that is, with 1.8% and 0.6% admixed chlorides - was to create a macro-element as explained in Section 4.2. The cover to the outer reinforcing steel was set at 20 mm from the top surface and sides of the RC slabs. The discrete sacrificial anodes were installed after corrosion had been induced in the slabs. Corrosion was induced through the use of direct anodic current as discussed earlier.

The reinforcement layout of a typical slab is shown in Figure 4.3 (a - b).

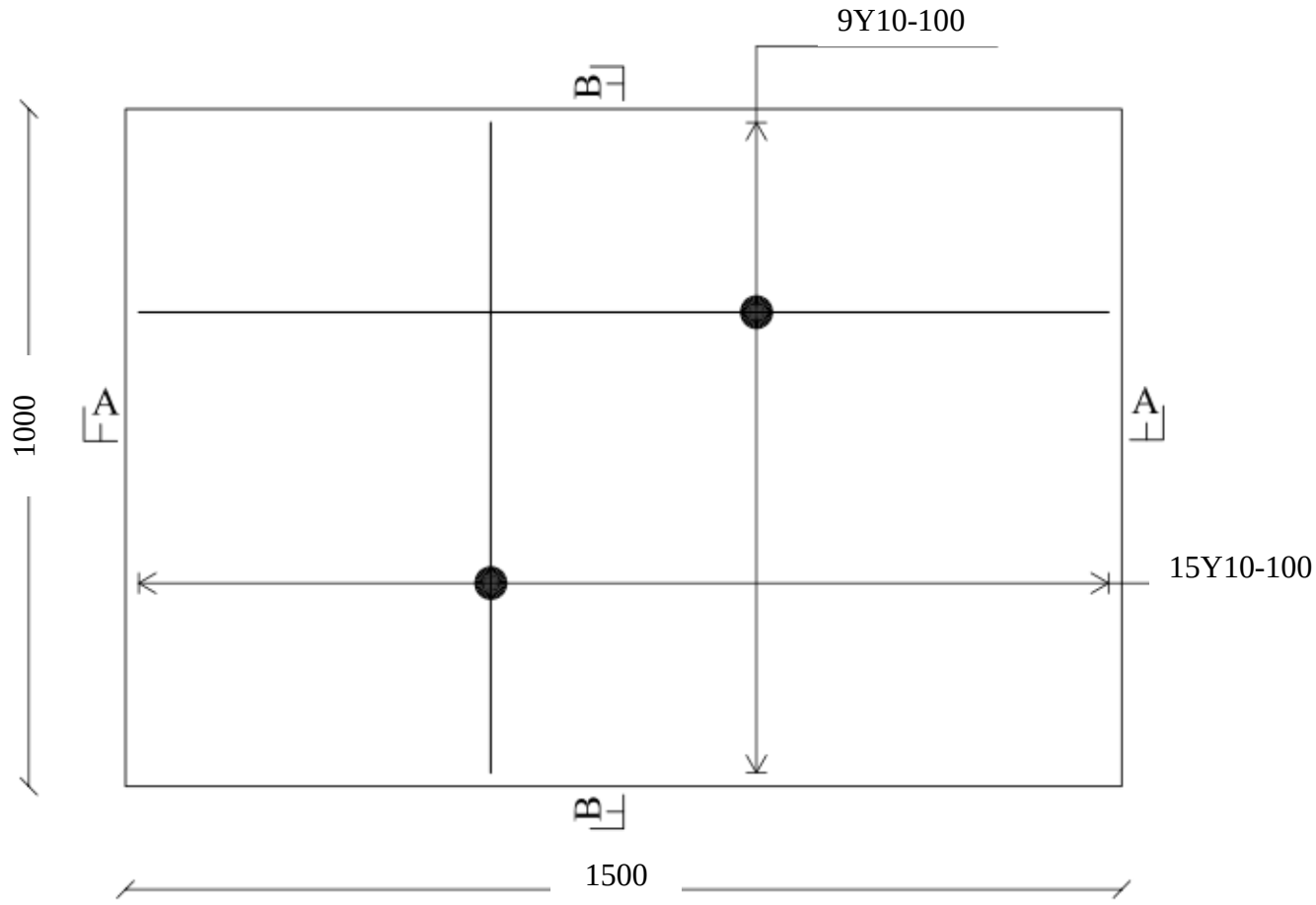
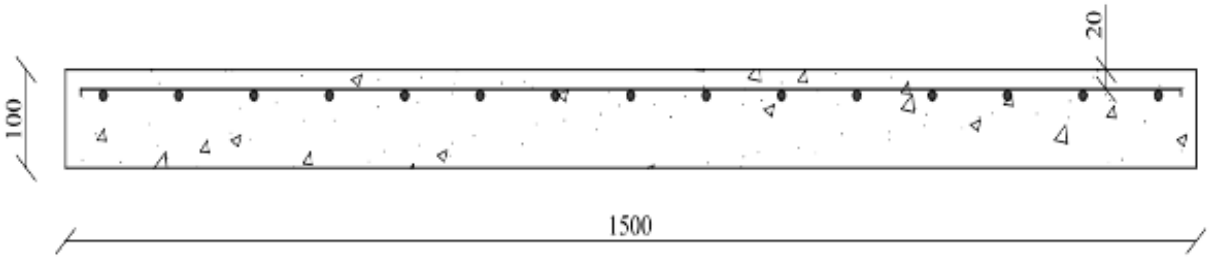
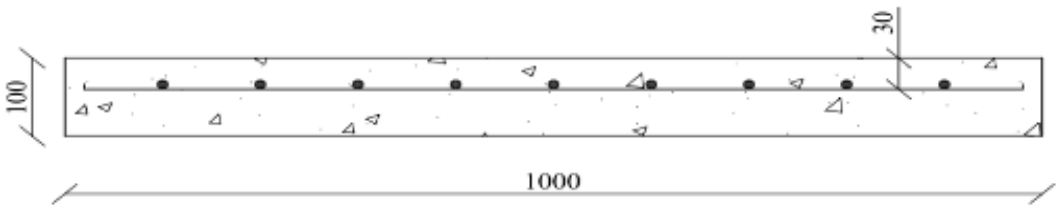


Fig. 4.3a: Slab reinforcement layout (Dimensions in mm)



Section A-A



Section B-B

Figure 4.3b: Typical RC slab section details (Dimensions in mm)

The configuration of the RC slab as well as the concentration of chloride that was admixed in the concrete while casting the RC slabs is shown in Figure 4.4.

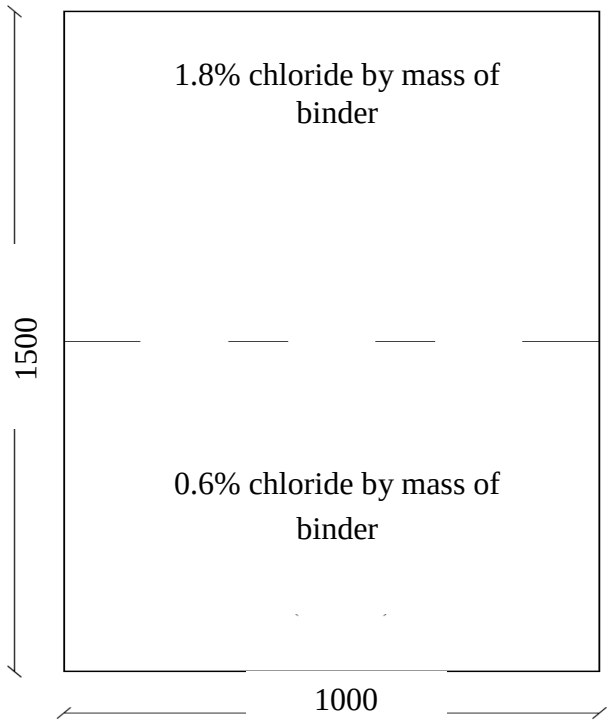


Figure 4.4: Slab configuration showing the concentration of chloride (Dimensions in mm)

4.4. Experimental variables

The effectiveness of discrete SACP systems was evaluated with respect to the following variables:

i. Corrosion rates: using the corrosion rates that were achieved in the slabs after corrosion was induced using the two direct anodic currents that were described in Section 4.2

ii. Binder type: using the binders described earlier (Section 4.2)

iii. Chloride content: using the chloride contents of 1.8% and 0.6% by mass of binder that were admixed into concrete during casting

4.5. Test equipment for corrosion assessment

The effectiveness of SACP systems was evaluated through: half-cell potential (HCP) and corrosion rate measurements. Therefore, the equipment for this study comprised the following:

i. Corrosion rate measurement device: Gecor-6 device with a guard ring and a sensor

ii. Silver/Silver chloride reference electrode (RE): to measure half-cell potentials

iii. Resipod Resistivity Meter: to measure the resistivity of concrete

iv. Temperature and relative humidity probe: to measure the RH and temperature of the atmosphere

4.6. Test materials

The following materials were used to cast slabs for this study:

i. Ordinary Portland cement, (PC CEM 1 42.5 N, supplied by PPC)

ii. Fly ash (Class F, supplied by Ash Resources)

iii. 13 mm greywacke stone as coarse aggregate (CBD = 1570kg/m^3; relative density 2.7)

iv. Klipheuwel sand as fine aggregate (Fineness modulus = 2.7; relative density = 2.65)

v. A commercially available sacrificial anode (Sika Galvashield CC65): This system comprises discrete zinc disks put in a cylinder of a proprietary high alkaline mortar with a tie wire that was used to attach it to the reinforcement cage

vi. 10 mm diameter high yield steel bars as reinforcement

vii. Sodium chloride as the source of chloride ions for these experiments

4.7. Test preparation

2 No. 1500 x 1000 x 100 mm timber moulds were fabricated for casting RC slabs. Owing to the fact that the mechanical and electrochemical behaviour of steel in RC applications depends on its surface condition (Al-Dulaijan *et al.*, 2002); all reinforcing steel were cut into their appropriate lengths (as per the drawing on Figure 4.3a) and then cleaned prior to fixing using a wire brush to remove mill scale. Cleaning of the steel helps in reducing disruptions in electrochemical corrosion rate measurements (Dehghanian, 2003). 70 mm thick cover blocks were made using mortar. The cover blocks were connected to the outer reinforcement. The ratio of the weights of water to cement to sand that was used to make the mortar for cover blocks was 1:2:4. The cover blocks were cured in a curing tank for 7 days. An insulated copper cable was connected to the reinforcement cage. The cable enabled the connectivity to the rebar cage during corrosion rate and half-cell potential measurements. The mould assembly and rebar arrangement of a typical slab is shown in Figure 4.5.

4.8. Test procedure

The following testing regime and sequence was adopted:

i. Concrete batching and mixing

ii. Casting and curing of concrete: the slabs were cured using water from a hose pipe

iii. 28-day concrete cube compressive strength and chloride conductivity index (CCI) tests

iv. Inducing corrosion in the slabs using direct anodic current: was done 8 weeks after the last date of casting

v. Preliminary corrosion rate measurements: began immediately after corrosion in the slabs was induced (i.e., 9 weeks after the last date of casting) and lasted for 7 weeks after the withdrawal of the direct anodic current

vi. Installation of discrete sacrificial anodes: began 22 weeks after the last date of casting)

vii. Corrosion rate, half-cell potential and concrete resistivity monitoring: began 16 weeks after the last date of casting

The slabs within which sacrificial anodes had been installed were wetted three times a week, namely: on Mondays, Wednesdays and Fridays. Wetting ensures that adequate current is delivered (Sagüés and Powers, 1996). The wetting of the slab was done using tap water that was applied using a hose pipe for a period of two minutes. The operation of SACP systems is affected by ambient moisture content/relative humidity (RH), atmospheric temperature, concrete resistivity and corrosion rate. Thus, these parameters were measured and recorded simultaneously during the measurements for half-cell potential, concrete resistivity and corrosion rate.

Figure 4.5: Typical mould assembly and rebar arrangement

4.8.1. Mix design, casting and curing of concrete slabs

The concretes were designed according to the Cement and Concrete Institute (C&CI) method (Addis and Goodman, 2009). The constituents of the concrete mixes are summarised in Table 4.1. Admixtures were not used.

Table 4.1: A summary of the concrete mix design

Material (kg/m^3)	**100 % PC**	**70/30 PC/FA**
PC (CEM 1 42.5N)	405	283
FA	-	121
Klipheuwel sand	850	767
Greywacke (13 mm)	990	1039
Water content (ℓ/m^3)	190	190
Water/binder ratio	0.47	0.47

Galvanic systems usually require concrete with a good electrolytical conductivity. Thus, considering a water demand of concrete of 0.38 and the need to design a concrete mix with a high capillary porosity that facilitates the easy diffusion of chloride ions; a water-to-binder ratio of 0.47 was used. Slump values of 120 mm and 150 mm were achieved in the 100% PC and 70/30 PC/FA concrete mixes respectively. 0.6% and 1.8% chloride (by mass of binder) were admixed with concrete during casting as shown previously (Figure 4.4). The concrete was thereafter placed in the formwork containing the reinforcement cage (Figure 4.5), compacted using a poker vibrator and its surface levelled using a trowel. A temporary wooden barrier was used to separate the mixing of concrete that was admixed with 0.6% chloride from the one that was admixed with 1.8% chloride. The temporary barrier was removed after the concrete was compacted with a poker vibrator. The freshly cast slab was thereafter covered using a plastic sheet for four days. The plastic sheets protected the slabs from raindrops as well as the effects of exposure to direct sunshine. The cast RC slabs at the exposure site are shown in Figure 4.6.

The formwork was stripped from the concrete after three days. Thereafter, an epoxy coating (Sikagard - 63 N) was applied on the sides of the slabs using a roller brush according to the manufacturer's instructions. The epoxy coating was used to prevent the ingress of aggressive species (carbon dioxide and chlorides) from the sides of the slabs as well as to simulate the specimens as parts of a large slab. The cast RC slabs were cured for 4 weeks using water from a hose pipe. Cold tap water was applied for 2 minutes daily. Nevertheless, owing to the fact that the slabs were cast during the rainy season, the slabs were also wetted with rain water. After 8 weeks from the last date of casting, corrosion was induced in the slabs over a period of one week using direct anodic current (DC).

Figure 4.6: Cast RC slabs at the exposure site

Direct anodic currents of 14.14 and 41.48μA were applied to the slabs over a period of one week with the aim of inducing corrosion rates that correspond to 'moderate-to-high' corrosion conditions (0.75μA/cm^2) and 'high' corrosion conditions (2.2μA/cm^2) respectively as presented in the calculations in Appendix B.1. These target corrosion rates, however, were not achieved. A pond containing a solution of 3% NaCl was placed on top of each slab. The reinforcement in the slab was connected to the positive terminal of the power supply while a stainless steel cathode was put in the pond containing the salt solution. Figure 4.7 shows a typical circuit diagram that was used to induce corrosion of reinforcing steel in the slabs. The slabs upon which a specific direct anodic current was applied (14.14μA or 41.48μA) were connected in series. The induced corrosion rates were monitored regularly, over a period of 7 weeks after the withdrawal of the current that was used to induce corrosion. A Gecor 6 device with a guard ring and sensors was used to measure the induced corrosion rates. Specific details on corrosion rate measurements are discussed in Section 4.8.2.1. Owing to a delay in delivery, the discrete sacrificial anodes were installed in the slabs 22 weeks from the last date of casting.

Corrosion rate, half-cell potential (HCP) and concrete resistivity measurements were made at the locations shown in Figure 4.8. Prior to the installation of the sacrificial anodes, appropriate anode locations were marked on the slabs as shown in Figure 4.8. The location of the anodes was informed by the manufacturer's instructions regarding the spacing of anodes. The concrete that was enclosed within the marked areas was excavated as shown in Figure

4.9. Six 100 x 100 x 60 mm deep cavities (labelled 1-6 in Figure 4.8) on a square grid of 450 x 450 mm were excavated in each slab.

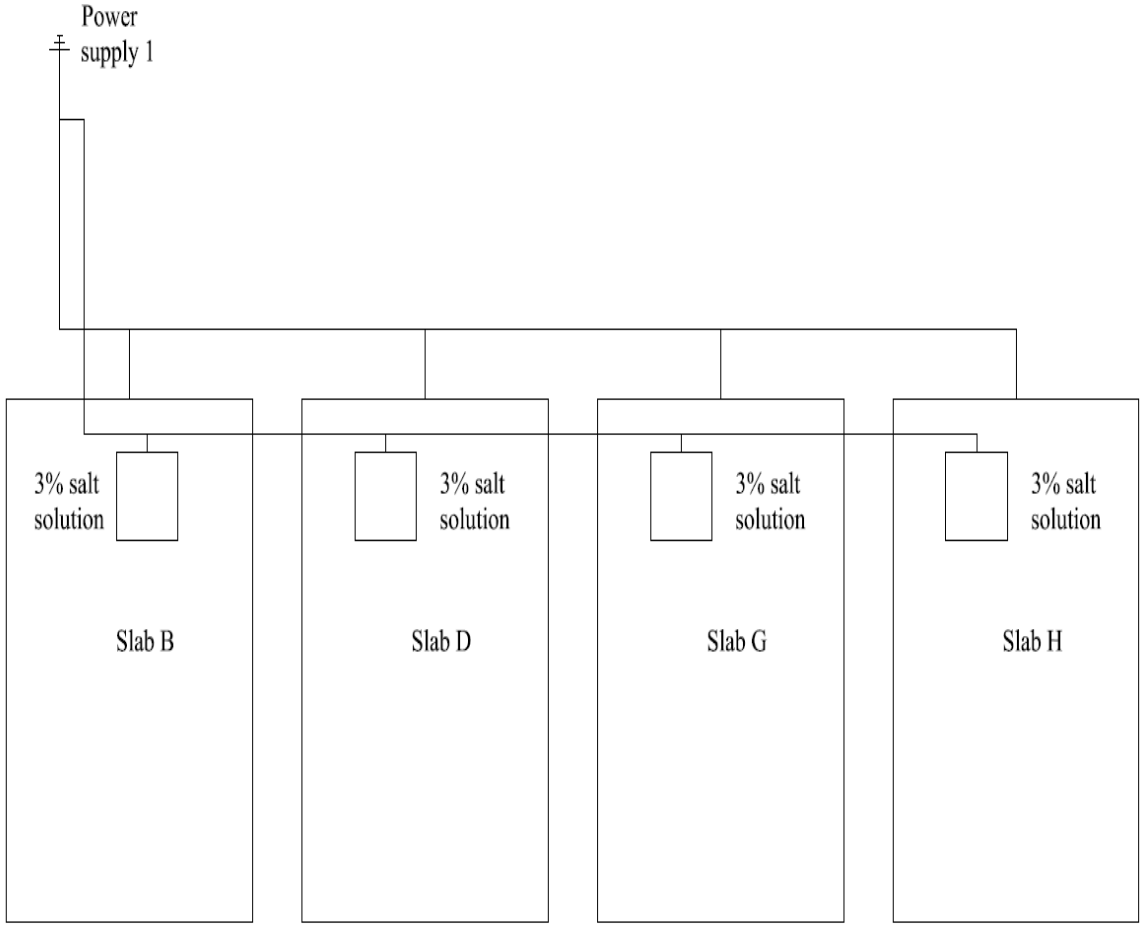


Figure 4.7: A typical circuit diagram of slabs that were subjected to corrosion rate of $0.75\mu A/cm^2$

The choice of the anode configuration was based on the reinforcement density ratio as provided in the manufacturer's installation instructions. The excavated concrete cavities were cleaned thoroughly (Figure 4.10) using a high-pressure water jet and then left to dry for 48 hours. Compressed air was also used to remove any dust particles within the cavity, so as not to compromise the quality of the bond between the mortar and the concrete substrate, which would consequently reduce the lifespan of the repair. Thereafter, the discrete sacrificial anodes were installed according to the manufacturer's instructions. A repair mortar, Sika-Rep LW, was used as the backfill during the anode installation process. The product data sheet of this mortar is contained in Appendix G. The mortar was mixed mechanically using a pan mixer, and then placed in the excavated holes containing the anodes. The mortar was water cured for seven days using wet hessian. To minimise the rapid evaporation of water from the wet hessian, a plastic sheet was used to cover the wet hessian.

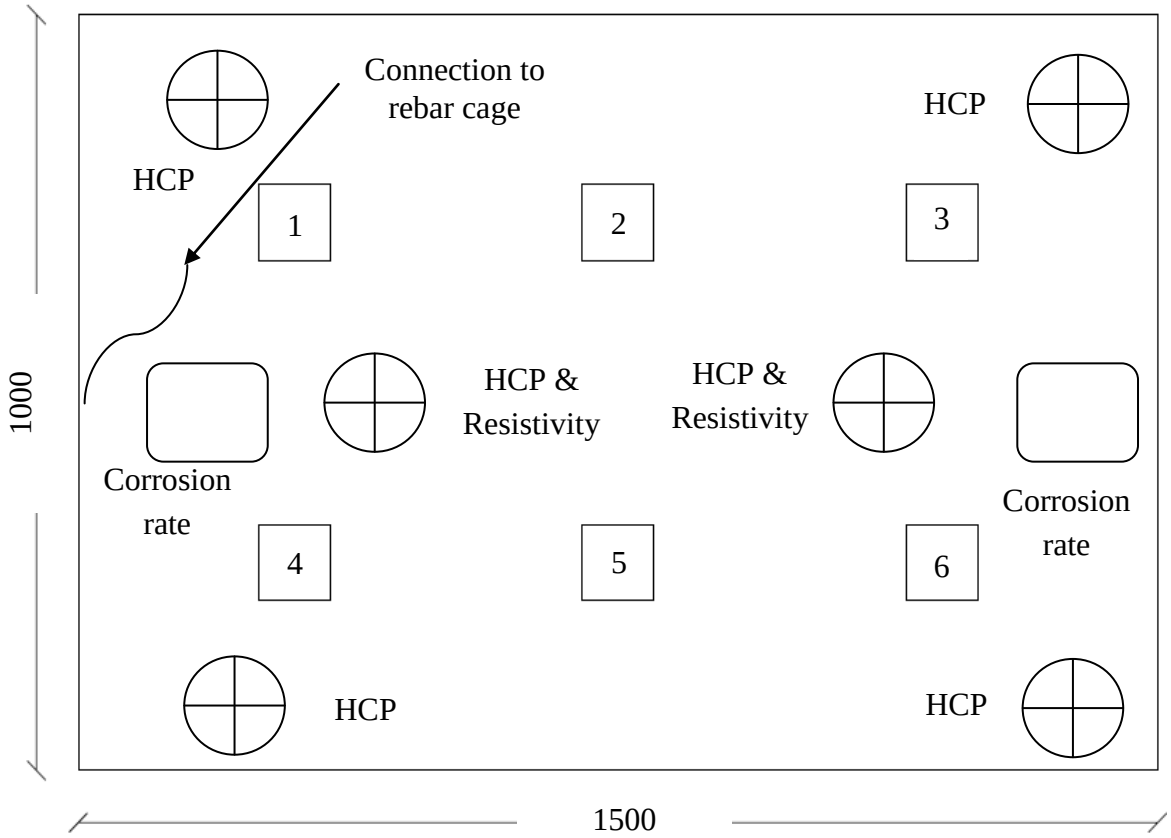


Figure 4.8: Slab layout showing location of anodes and measurement positions (Dimensions in mm)

Figure 4.9: Excavation of concrete in preparation for the installation of anodes

Figure 4.10: Concrete cavities after cleaning with a high-pressure water jet

4.8.2. Corrosion monitoring of steel in sacrificial anode cathodically protected slabs

Corrosion monitoring is a vital process. Its importance is attributed to the fact that it gives quantitative information about the development of corrosion once aggressive species penetrate into concrete. It is also used to assess the performance and effectiveness of rehabilitation systems such as coatings, corrosion inhibitors, and cathodic protection systems. Thus, corrosion monitoring plays a vital role in the planned maintenance and service life prediction of RC structures (Song and Saraswathy, 2007; Duffó and Farina, 2009).

Corrosion of steel in concrete is an electrochemical process. Thus, electrochemical techniques are well suited for measuring the corrosion rate, monitoring and assessing the state of corrosion of steel in concrete as well as pinpointing corrosion sites (Andrade and Alonso, 1996; Bertolini *et al*., 2004a). Electrochemical corrosion monitoring techniques are preferred to other methods because: they are fast and can account for very low values of corrosion rate (Andrade and Alonso, 1996; Koleva *et al*., 2006). Corrosion monitoring, in RC structures, makes use of electrochemical parameters such as polarisation resistance (R_p), corrosion potential (E_{corr}), corrosion current density (I_{corr}), half-cell potentials and concrete resistivity. One of the recent developments in corrosion monitoring is the use of sensors that can facilitate the continuous monitoring of the corrosion process. Specific details on the types and uses of such sensors, and their benefits have been reported by McCarter *et al* (2001), Schwarz (2003 and 2010), Song and Saraswathy (2007) and Yu and Caseres (2011). Finally, it is important to appreciate the fact that corrosion monitoring in concrete is difficult because it involves the monitoring of parameters which fluctuate with time (Duffó and Farina, 2009).

This fluctuation is due to variations in ambient temperature and relative humidity, the concentration of aggressive species, moisture content, etc in concrete.

4.8.2.1. Corrosion rate measurements

Corrosion rate refers to the amount of metal wastage produced by a unit surface area of metal when referred to a specific period of time (Andrade and Alonso, 1996; Andrade *et al*., 2004). Corrosion rate measurements are the only reliable method of measuring the actual corrosion activity in RC structures (Mackechnie and Alexander 2001). Thus, quantitative information pertaining to the corrosion rate of steel in concrete is important for the identification of actively corroding zones (Andrade *et al*., 2004), the evaluation of repair methods (Soleymani and Ismail, 2004), the control of repair work on site, service life prediction (Wojtas, 2004; Elsener, 2005; Pradhan, and Bhattacharjee, 2009) and structural assessment of corroding structures. The Gecor-6 corrosion rate meter with a guard ring and sensors (Figure 4.11), was used to measure corrosion rates.

Figure 4.11: Gecor-6 corrosion rate measurement device

Corrosion rate measurements were made after removing the current that was used to induce corrosion. These measurements were taken once a week and on an alternating schedule – that is, consecutive measurements within the portion of the slab that was admixed with either 1.8% or 0.6% chloride were made after two weeks. The points where corrosion

rates were measured are shown in Figure 4.8. In addition, the processes that pertain to the preparation of the concrete surface as well as the corrosion rate measurements were done according to the procedures described by Andrade *et al* (2004). Auxiliary data such as the atmospheric temperature and relative humidity were obtained from local daily weather forecast reports and corroborated independently using temperature and relative humidity probes at the time of corrosion rate measurements. These parameters, ambient temperature and relative humidity, were taken into consideration because fluctuations in ambient temperature, moisture levels in the concrete and oxygen availability result in variations in instantaneous corrosion rates (Andrade and Alonso, 2001; Bungey *et al*., 2006).

4.8.2.2. Half-cell potential measurements

The half-cell potential (HCP) (also known as open-circuit potential, rest potential or corrosion potential) is the electrochemical potential, measured in Volts, between a sacrificial black steel working electrode and a reference electrode (Duffó and Farina, 2009; Trejo *et al*., 2009). Any surface (typically a piece of metal) on which an electrochemical reaction takes place will generate an electrochemical potential difference when it is in contact with an electrolyte (Myrdal, 2007). Thus, the measurement of corrosion potential (or half-cell potential) is one of the simplest ways of assessing the severity of corrosion or the presence of electrochemical activities that take place on the rebar surface.

Half-cell potential measurements, in the form of potential maps, are used in condition assessment and repair work. While half-cell potential maps have been reported to help in indicating the probability of reinforcement corrosion (Broomfield *et al*., 2002; Ahmad, 2003; Grantham, 2003; González *et al*., 2004; Poupard *et al*., 2006; Song and Saraswathy, 2007), they can also be used to determine the precise location of corroding zones. In addition, the potential maps (that result from half-cell potentials) are useful while assessing the efficiency and durability of electrochemical repair procedures (Elsener, 2001 and 2005). The half-cell potential of steel provides information about the protection level of the steel from corrosion when cathodic protection is applied. On the contrary, potential maps do not provide information on the corrosion rate or the amount of steel lost. Thus, they ought to be interpreted carefully.

A Silver/Silver chloride reference electrode (Figure 4.12) was used to measure half-cell potentials. Owing to the fact that SACP systems - especially surface-applied SACP systems - interfere with HCP measurements (Gu and Beaudoin, 1998), HCP measurements were not made on top of the repair mortars that were used as a backfill for the anodes. Concrete resistivity measurements were taken alongside half-cell potential measurements. The effectiveness of sacrificial anodes is usually evaluated using the 4 hour and 24 hour depolarisation tests (Schwarz, 2003; Bertolini *et al*., 2004a). These tests involve the measurement of the decay in the potential of the reinforcing steel over a period of 4 hours or 24 hours after the electrical circuit between the anode and the steel is open. The decay in potential ought to meet the 100 mV potential decay criterion (i.e., be greater than 100 mV) before an anode can be deemed to be effective. The 4 hour and 24 hour depolarisation tests

were not done. The manner in which the anodes were installed in the slabs as well as the lack of instruments hindered the ease, effectiveness and accuracy with which these measurements could be taken.

The half-cell potential measurements were taken over a period of five months. Three half-cell potential measurements were taken each week on Tuesday, Friday and Sunday. The continuity of the steel was first checked using a direct current (DC) resistant meter. Thereafter, HCP measurements were measured as follows (Mackechnie and Alexander, 2001; Broomfield, 2007):

i. The Silver/Silver chloride reference electrode, the voltmeter and its associated cables and connections were checked

ii. Connections were made to steel using a cable that was cast in the concrete

iii. A 100 x 100 mm grid that corresponds to the layout of the embedded steel reinforcement was marked out on the slabs using a crayon

iv. The Silver/Silver chloride reference electrode was connected to the reinforcement cage and consequently used to measure half-cell potentials on points along the marked grid. Six half-cell potential measurements (with three readings each being taken from the portion of the slab that was admixed with 0.6% and 1.8% chloride) were made at the locations shown in Figure 4.8. The mean of the three readings that were taken within a given slab portion was recorded as the half-cell potential of the steel within the given portion.

Figure 4.12: Half-cell potential measurements using a Silver/Silver chloride reference electrode

Before the half-cell potential measurements were made, the surface of the concrete was wetted using tap water to ensure a good electrical contact between the reference electrode and the concrete. The reading and interpretation of half-cell potential has been reported to be affected by several factors such as the condition of the concrete surface, type and condition of reference electrode as well as ambient temperature and relative humidity (Soleymani and Ismail, 2004; Ann and Song, 2007; Klechka, 2007; Song and Saraswathy, 2007; Nygaard and Geiker, 2010). Therefore, parameters such as environmental conditions (e.g., temperature), details of the reference electrode and the condition of concrete were simultaneously recorded during half-cell potential measurements.

4.8.2.3. Concrete resistivity measurements

The resistivity of concrete is a geometry-independent material property that describes the ratio between applied voltage and current in a unit cell. Electrical resistivity measurements have been used to indirectly evaluate concrete diffusivity, the degree of concrete saturation and its aggressiveness, and the size and tortuosity of the pores in a concrete system (Polder *et al*., 2000; Morris *et al*., 2002; Grantham, 2003; Broomfield, 2007; Song and Saraswathy, 2007). Thus, resistivity measurements can be used to assess the ease with which corrosion currents flow through concrete. Moreover, resistivity can be related to the corrosion rate and the susceptibility to penetration by chloride. Low resistivity spots within a structure, for example, indicate the possible locations where chloride penetration will be quickest. In addition, it has been observed that corrosion rate is inversely proportional to concrete resistivity (Polder, 2001; Morris *et al*., 2002; Polder and Peelen, 2002). Polder *et al* (2000) reported that the resistivity of concrete is affected by factors such as temperature, moisture content, composition of the concrete, curing time and carbonation. This inverse relationship between resistivity and corrosion rate, however, may vary with the concrete composition.

The resistivity of concrete, as well as its distribution within a structure, affects the way electrochemical repair methods operate. Specifically, concrete resistivity influences the effectiveness and durability of cathodic protection of concrete structures. This is due to the fact that it is difficult to achieve uniform protection in structures that show a wide variation in resistivity (Polder, 2001; Song and Saraswathy, 2007). The concrete resistivity measurements were made using the Resipod Proceq Resistivity Meter (Figure 4.13). Concrete resistivity was measured once a week at the locations shown in Figure 4.8. The measurements were undertaken as per the method that has been proposed in RILEM TC 154 by Polder *et al* (2000). The following procedure was followed:

i. The concrete surface was cleaned and the rebar mesh position marked with a crayon on the surface of the slab

ii. With the four electrodes as far as possible from the rebars, measurements were made diagonally inside the rebar mesh (see Figure 4.15). Five readings were taken from the same position. The electrodes were moved a few millimetres (5 mm) between the readings

iii. The median (the middle value) of the five readings for that position was taken as the resistivity of the concrete

Figure 4.13: Resipod Proceq Resistivity Meter

Figure 4.14 shows the manner in which the measurements for the resistivity of the RC slabs were taken.

Figure 4.14: A Resipod Proceq Resistivity Meter in use

Care was taken to avoid measuring the resistivity of an aggregate particle by ensuring that the spacing between the four probes of the resistivity meter was at a distance greater than the maximum size of the aggregates that were used in making the concrete (Concrete Society, 1989; Polder *et al*., 2000). Both the concrete and the probes of the resistivity meter were wetted slightly to ensure an enhanced electrical contact between concrete and the resistivity meter. Measurements were made away from the edges of the slabs.

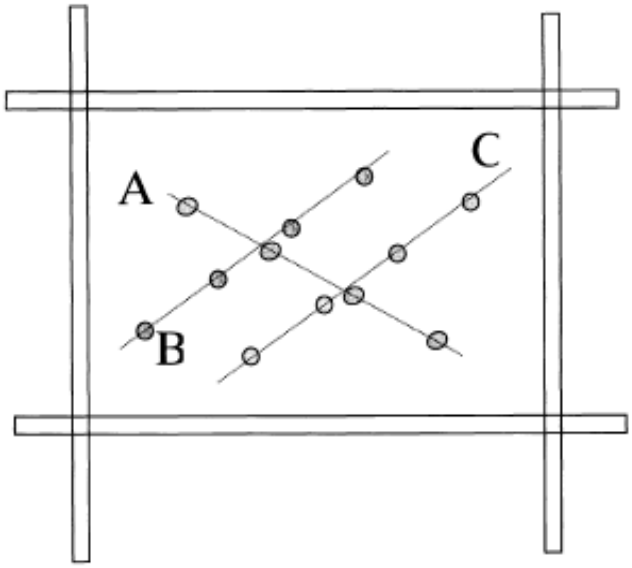


Figure 4.15: Resistivity measurements (four electrodes) points in the same area to minimise influence of rebars (Polder, 2001)

4.9. Summary

This chapter describes the experimental set up that was used in this study. Furthermore, specific details that pertain to the size of specimens, materials that were used, as well as the parameters that were monitored have been presented.

Corrosion monitoring is a vital process and can be used to assess the performance and effectiveness of rehabilitation systems such as coatings, inhibitors and cathodic protection systems – including SACP systems. Corrosion monitoring makes use of electrochemical parameters like polarisation resistance, corrosion potential, corrosion current density, half-cell potentials and concrete resistivity.

Corrosion rate measurements are the only reliable method of measuring actual corrosion activity in RC structures. Half-cell potential measurements help in indicating the probability of reinforcement corrosion. They can also be used to determine the precise location of corroding zones, assess the efficiency and durability of electrochemical repair techniques. Half-cell potentials, however, cannot be used to provide on the corrosion rate or the amount of steel that is lost during corrosion. The resistivity of concrete is a geometry-independent property that describes the ratio between the applied voltage and current in a unit cell.

Concrete resistivity measurements can be used to indirectly evaluate concrete diffusivity, the degree of concrete saturation as well as the ease with which corrosion currents flow through concrete. The corrosion rate of steel in concrete is inversely proportional to the resistivity of concrete. Finally, it is important to note that corrosion rates, half-cell potential and concrete resistivity measurements are affected by changes in atmospheric temperature as well as the temperature concrete within concrete, and relative humidity.

CHAPTER 5: RESULTS AND DISCUSSIONS

The performance of a sacrificial anode (or galvanic) CP system can be evaluated by monitoring the current provided by the sacrificial anode, the polarisation of the steel (USDoT, 2001) as well as changes in the corrosion rate of reinforcement. While the evaluation of the current provided by sacrificial anodes involves the measurement of macrocell currents (Schwarz, 2003), the evaluation of the polarisation of steel and changes in corrosion rate involves taking half-cell potential and corrosion rate measurements respectively. The effectiveness of the discrete SACP system that was installed in the slabs in this study was evaluated with respect to corrosion rate and half-cell potentials. Information pertaining to the measurement of these parameters has been presented in Chapter 4.

Data were collected from eight RC slabs (labelled as A, B, C, D, E, F, G and H). Four of these slabs (i.e., slabs A, B, C and D) were made using 100% PC CEM 1, 42.5N (hereafter referred to as PC) while the other four (i.e., E, F, G, H) were made using a 70/30 blend of PC and fly ash. Four of these slabs (i.e., slabs A, C, E and G) were used as test slabs while the other four slabs (i.e., slabs B, D, F and H) were used as controls. The sacrificial anodes were installed in the test slabs. A direct anodic current of 14.14µA was applied in slabs C, D, G and H. Similarly, a direct anodic current of 41.48µA was applied in slabs A, B, E and F. This information has been summarised in Table 5.1.

Table 5.1: Summary of experimental set-up

Slab Parameter	**Test slabs**				**Control slabs**			
	A	C	E	G	B	D	F	H
Binder type	100% PC		70/30 PC/FA		100% PC		70/30 PC/FA	
Applied direct anodic current (µA)	41.48	14.14	41.48	14.14	41.48	14.14	41.48	14.14
Sacrificial anodes present	Yes				No			

The slabs were monitored over a period of five months. The specific details pertaining to the methodology and frequency of measurements were presented in Chapter 4 (Sections 4.2 and 4.8). Owing to the delay in delivery, the discrete sacrificial anodes were installed in the slabs after 151 days (22 weeks) from the last date of casting. This day corresponds to day 41 in all graphs presented in this chapter. The following conventions have been used in the graphs throughout this chapter:

i. Corrosion rate and half-cell potential measurements began 110 days (16 weeks) from the last date of casting. This day corresponds to day 0 in all graphs.

ii. Dotted lines refer to parameters of interest within the portion of the slab that was admixed with 1.8% chloride by mass of binder

iii. Solid lines refer to the parameters of interest within the portion of the slab that was admixed with 0.6% chloride by mass of binder

Due to the varying nature of the test parameters as well as the limited number of data points, all analysed data within this chapter are represented in the form of 2-point moving average trend lines. The installation of sacrificial anodes in the test slabs resulted in disruptions in the electrochemical equilibrium of steel in concrete. These disruptions were accompanied by changes in corrosion rate and half-cell potential of steel. The changes in corrosion rate and half-cell potentials immediately after the installation of anodes could not be captured accurately due to lack of proper instrumentation. Consequently, it was not convenient to represent these changes using a straight line. It was deemed fit, therefore, to connect the trend lines of corrosion rate and half-cell potentials of steel, before and after the installation of sacrificial anodes in the test slabs, using an arrow. These arrows have been used in all the graphs that have been presented within this chapter. The arrows represent the immediate changes (in corrosion rate and half-cell potential) that took place in the test slabs after the installation of sacrificial anodes.

5.1. Corrosion rate measurements

Corrosion rate measurements were taken as described in Chapter 4 (Section 4.8.2.1). Direct anodic currents of 14.14µA and 41.48µA were applied to the slabs over a period of one week as discussed and presented in Sections 4, 4.8.1 and Appendix B.1. Throughout this section, the average corrosion rate after removal of the direct anodic current (i.e., before the installation of sacrificial anodes) has been calculated as the mean of all the corrosion rate values before the sacrificial anodes were installed. The average corrosion rate after the installation of the sacrificial anodes has been calculated as the mean of the last two achieved corrosion rate values. The use of the last two achieved corrosion rate values in calculating this average was based on the need to avoid the possibility of underestimating the effect of the discrete sacrificial anodes on corrosion rate that could have resulted from disturbances in the electrochemical equilibrium of the steel in the slab after the installation of the discrete sacrificial anodes.

Whereas the direct anodic current that was used to induce a specific target corrosion rate within a set of slabs connected in series was the same; the average corrosion rate values achieved within the specific slabs, 7 weeks after this current was removed, were different. The average corrosion rates achieved in the slabs are presented in Table 5.2. The average corrosion rates achieved in all slabs, 7 weeks after the time of withdrawal of the direct anodic current that was used to induce and accelerate corrosion, were different from the target corrosion rates. The slabs subjected to a low direct anodic current achieved corrosion rates that were higher than those subjected to a high corrosion current. This anomaly could not be explained from a corrosion science perspective. Nevertheless, the observed anomaly could be

attributed to either a fault in one of the DC power supplies that was used to supply the direct anodic current or a temporary disconnection of the current at the source by an unsuspecting third party.

Table 5.2: Comparison of target versus achieved corrosion rates, 7 weeks after removal of inducing anodic current

Parameter		**Slab ID**							
		C	D	G	H	A	B	E	F
Binder type		100% PC		70/30 PC/FA		100% PC		70/30 PC/FA	
Applied direct anodic current (µA)		14.14				41.48			
Average corrosion rate achieved after the withdrawal of anodic current (µA/cm^2)									
Portion of slab	0.6% admixed chloride	1.67	1.30	0.55	0.38	1.26	0.91	0.38	0.37
	1.8% admixed chloride	2.97	2.77	1.50	0.61	1.74	2.55	0.52	0.74

The corrosion rates achieved in slabs cast using 100% PC were higher than those achieved in slabs cast using 70/30 PC/FA. The low average corrosion rates achieved in slabs cast using 70/30 PC/FA can be attributed to their refined and less continuous pore structure that leads to a decrease in permeability and chloride solidification (Asrar *et al*., 1997; Rha *et al*., 2001; Trejo *et al*., 2009). Also, the low corrosion rates in the 70/30 PC/FA slabs can be attributed to the electrochemical properties of concrete made using a blend of 70/30 PC/FA (especially the high electrical resistivity). The less continuous and refined microstructure coupled with the high resistivity of 70/30 PC/FA slabs hinders the ease with which electrons and ions (or the anodic current that was used to induce corrosion) flow within the concrete. The resistivity of slabs cast using 100% PC ranged between 3-5 kΩ cm while that of slabs cast using 70/30 PC/FA ranged between 15-26 kΩ cm. Thus, it is evident that the resistivity of the 70/30 PC/FA was 5 times higher than the resistivity of 100% PC slabs. The trend lines pertaining to the resistivity of concrete slabs are presented in Appendix B.4.

The average corrosion rates achieved within the portions of the slab admixed with 1.8% chloride were higher than the average corrosion rates achieved within the portions admixed with 0.6% chloride. Thus, it can be inferred that a high concentration of chloride ions facilitates the attainment of high corrosion rates. An increase in concentration of chloride ions at the rebar surface corresponds to an increase in the rate at which the passive protective film breaks down resulting in higher corrosion rates. An increase in concentration of chloride ions also results in an increase in concrete conductivity. The increase in concrete conductivity (or

reduction in resistivity) enhances the ease with which electrons (or current) flow within concrete.

The use of a direct anodic current would have resulted in the electromigration of negatively charged chloride ions to the positively charged steel surface. Thus, with a constant amount of current being supplied to any given slab, the reinforcement embedded within portion of the slab that was admixed with 1.8% chloride would have attracted more chloride ions than the reinforcement embedded in the portion that was admixed with 0.6% chloride. The presence of a higher concentration of chloride ions at the steel surface would result in the rapid breakdown of the protective passive file; thus contributing to the high corrosion rates that were achieved in the portions of the slabs that were admixed with 1.8% chloride.

5.1.1. Corrosion rate measurements in 100% PC slabs

The changes in corrosion rate within the slabs upon which a direct anodic current of 41.48µA was applied (i.e., slabs A and B) are shown in Figure 5.1.

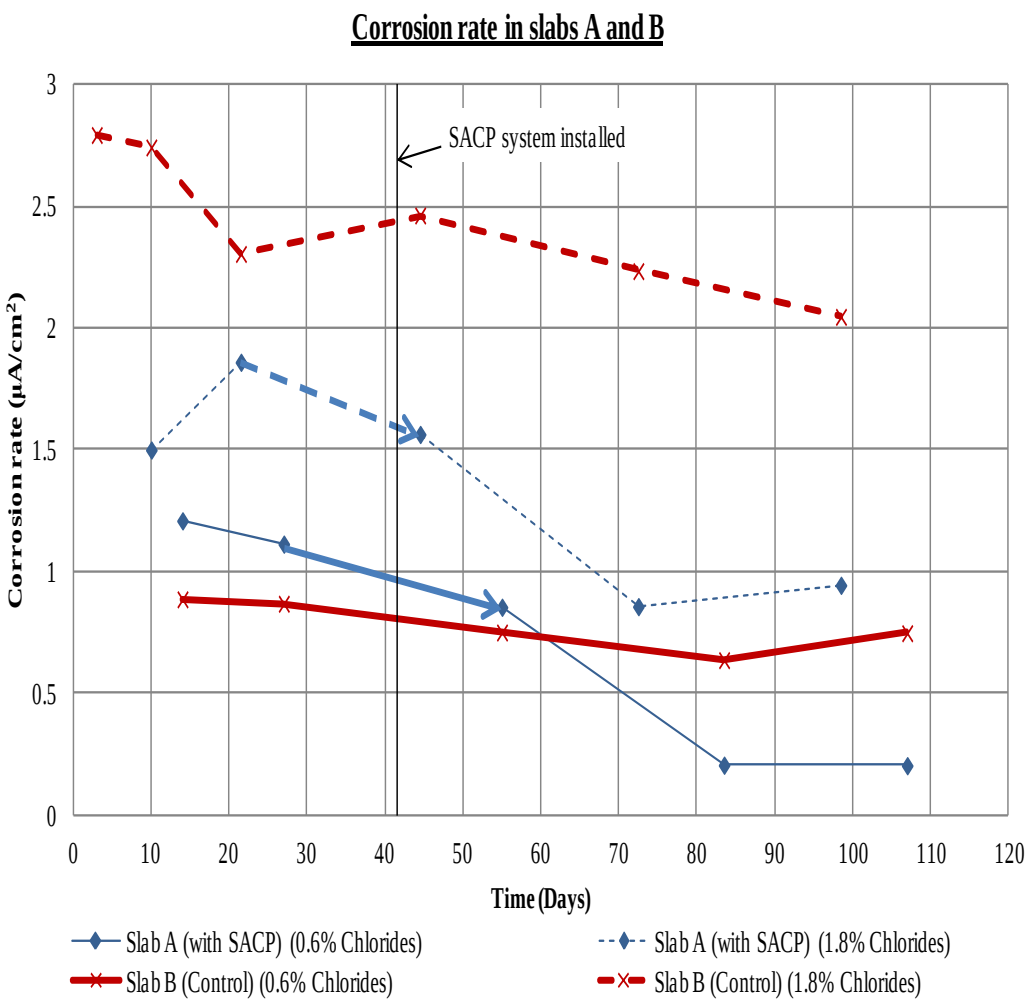


Figure 5.1: Trend lines of corrosion rate in slabs A and B (100% PC)

The changes in corrosion rate within the slabs upon which a direct anodic current of 14.14µA was applied (i.e., slabs C and D) are shown in Figure 5.2.

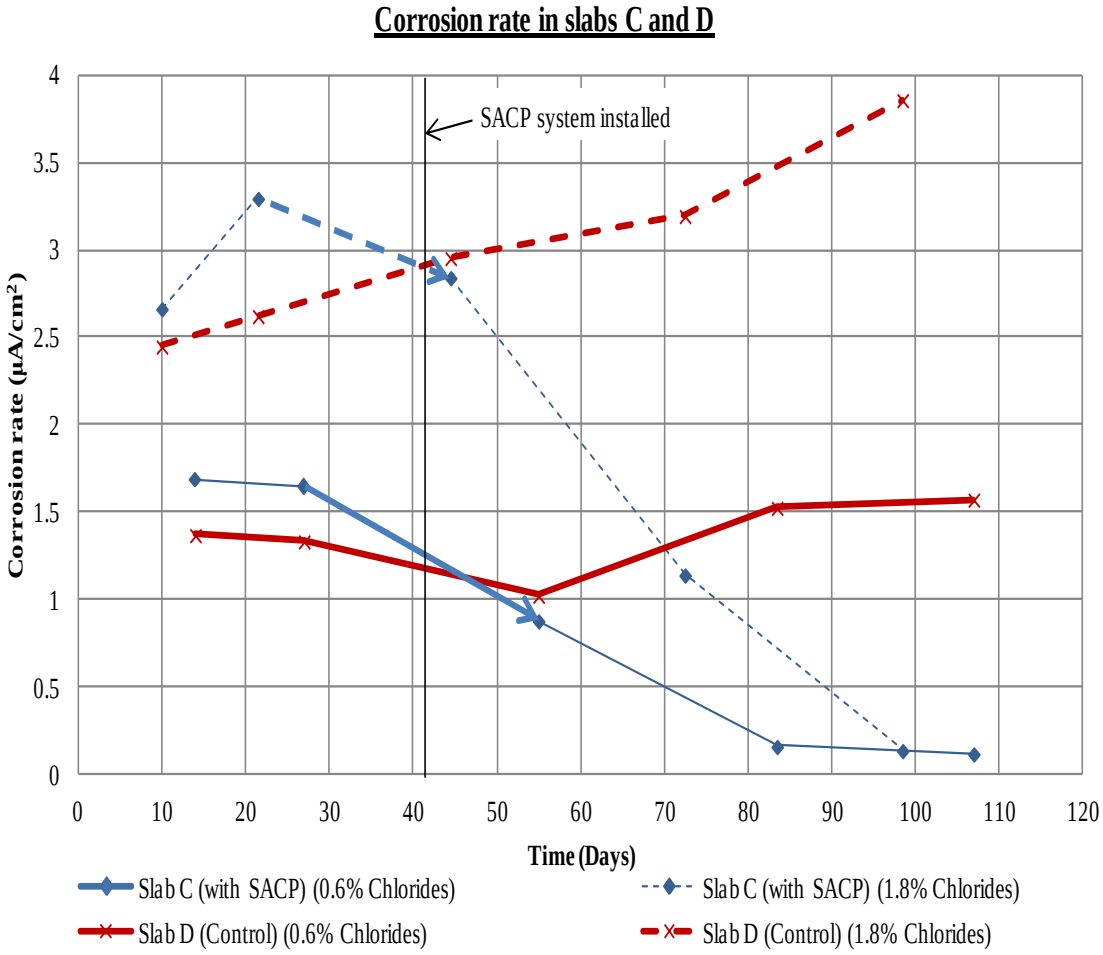


Figure 5.2: Trend lines of corrosion rate in slabs C and D (100% PC)

From Figure 5.1 and Figure 5.2, it can be observed that all slabs cast using 100% PC, with the exception of slab D, experienced a reduction in corrosion rate with time. The greatest reductions in the average corrosion rates during the test period, i.e., before and after the installation of sacrificial anodes, were experienced in the test slabs (i.e., slabs A and C). With the exception of the portion of slab A that was admixed with 1.8% chloride, the average corrosion rates were lowered from values that correspond to 'high corrosion' conditions, before the installation of sacrificial anodes, to values that correspond to 'low to moderate corrosion' conditions after the installation of sacrificial anodes. Quantitatively, the average corrosion rate after the installation of sacrificial anodes in slab A reduced by 1.06µA/cm^2 (i.e., 1.26 – 0.20µA/cm^2 or 84%) and by 0.80µA/cm^2 (i.e., 1.74 – 0.94µA/cm^2 or 46%) in the slab portions admixed with 0.6% and 1.8% chloride respectively. Similarly, the average corrosion rate after the installation of sacrificial anodes in slab C decreased by 1.55µA/cm^2 (i.e., 1.67µA/cm^2 – 0.12µA/cm^2 or 93%) and by 2.83µA/cm^2 (i.e., 2.97µA/cm^2 – 0.14µA/cm^2 or 95%) in the slab portions admixed with 0.6% and 1.8% chloride respectively. The changes in average corrosion rate in slabs B and D are presented in Appendix B.2, Table B.1.

The observed reductions in the average corrosion rate in test slabs A and C can be attributed to the presence of discrete sacrificial anodes that were installed in these slabs. While the percentage reduction in the average corrosion rates within the portions of slab C that were admixed with 0.6% and 1.8% chloride were almost equal (93% and 95% respectively), a different observation was made in slab A. The percentage reduction in the average corrosion rate within the portion of slab A that was admixed with 0.6% chloride was higher (i.e., 84%) than in the portion that was admixed with 1.8% chloride (i.e., 46%). This phenomenon can be attributed to the higher proportion of current - from the discrete sacrificial anodes – that flows to the cathodic sites (i.e., portions admixed with 0.6% chloride) than in the anodic sites (i.e., portions admixed with 1.8% chloride) in the slab macrocell. The cathodic sites achieved potentials that were more positive than the anodic sites (see Appendix B.3, Table B.2). Thus, under cathodic protection, the driving force inducing current flow to the cathodic sites would have been greater than that inducing current to the anodic sites; hence the tendency for a greater proportion of the current to flow to the cathodic sites than in the anodic sites in the slab macrocell (Glass *et al*., 2000).

5.1.2. Corrosion rate measurements in 70/30 PC/FA slabs

The changes in corrosion rate within the slabs upon which a direct anodic current of 41.48µA was applied (i.e., slabs E and F) are shown in Figure 5.3.

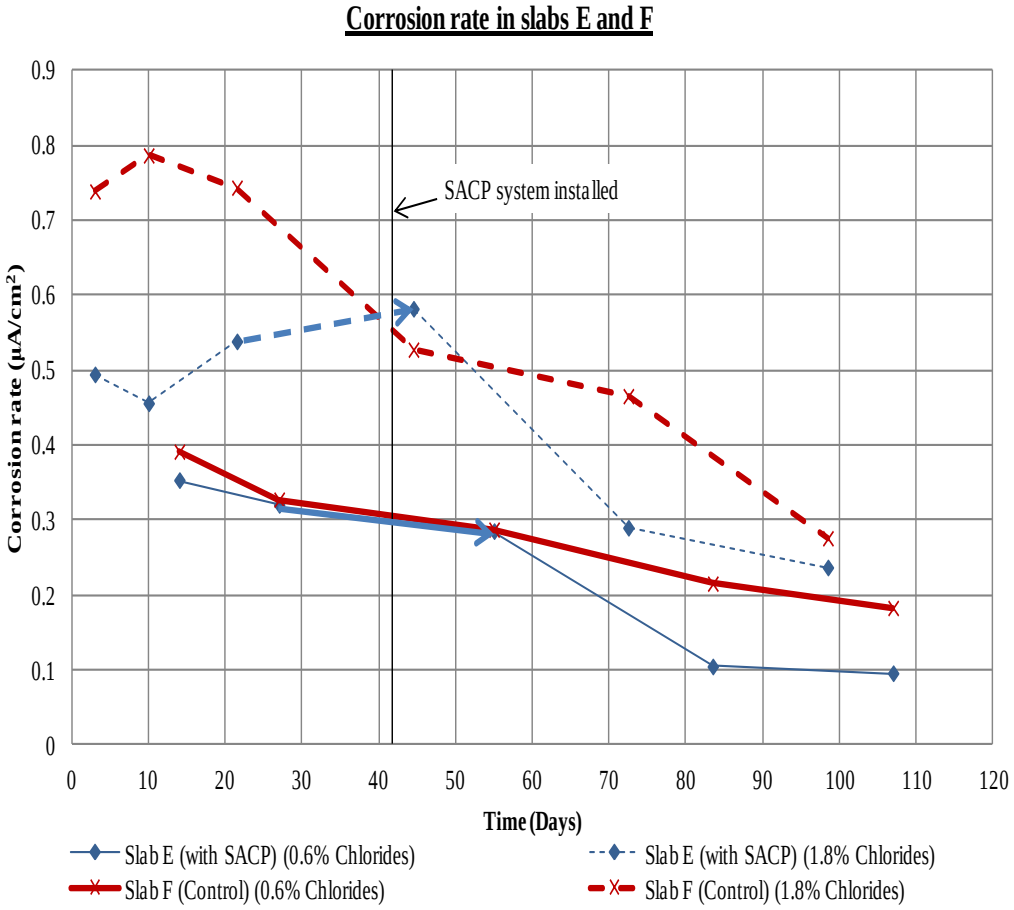


Figure 5.3: Trend lines of corrosion rate in slabs E and F (70/30 PC/FA)

The changes in corrosion rate within the slabs upon which a direct anodic current of 14.14µA was applied (i.e., slabs G and H) are shown in Figure 5.4.

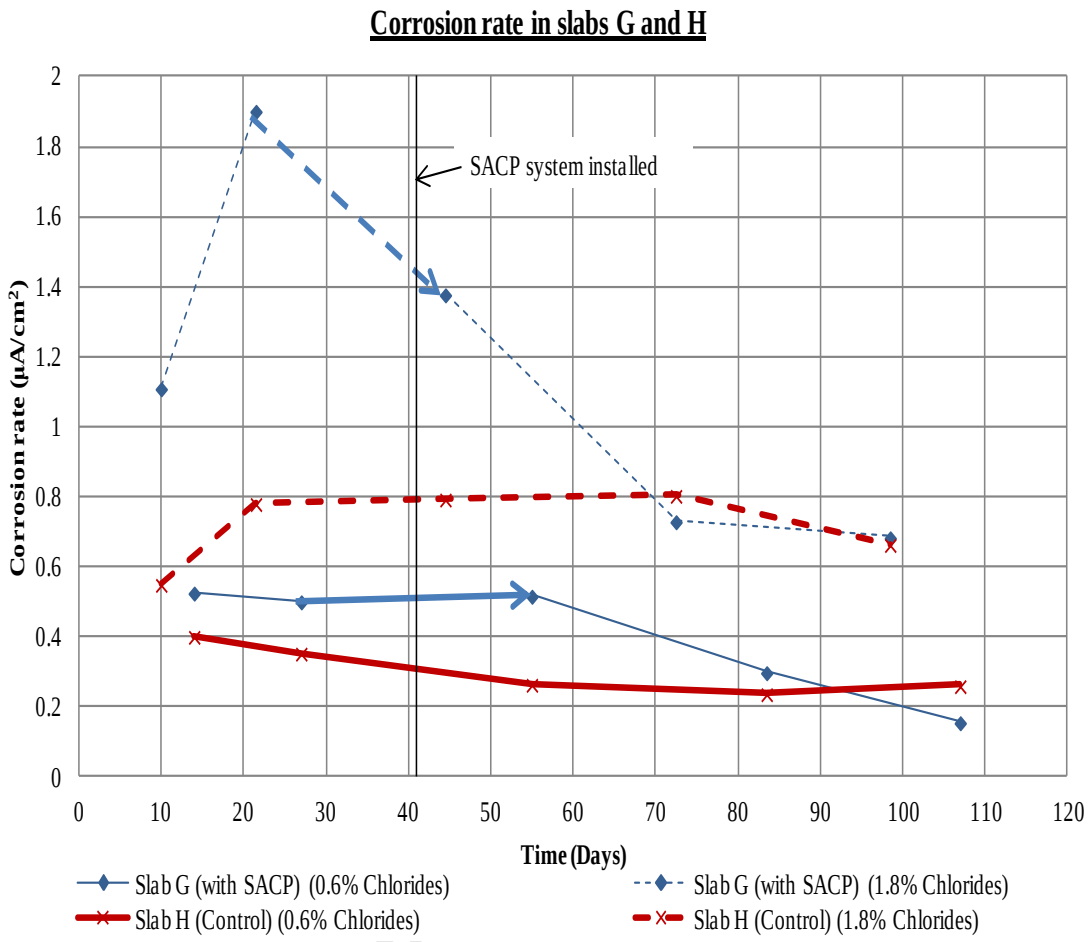


Figure 5.4: Trend lines of corrosion rate in slabs G and H (70/30 PC/FA)

From Figure 5.3 and Figure 5.4, it can be observed that there were reductions in average corrosion rate after the installation of sacrificial anodes in the test slabs (i.e., slabs E and G). A similar phenomenon was also observed in the control slabs (i.e., slabs F and H). However, slabs E and G experienced the greatest reductions in average corrosion rate after the installation of sacrificial anodes. The average corrosion rates in slab E were reduced from values that correspond to 'moderate to high' corrosion conditions, before the installation of sacrificial anodes, to values that correspond to 'low to moderate corrosion conditions after the installation of sacrificial anodes. In slab G, however, the average corrosion rates were lowered from values that correspond to 'moderate to high' corrosion conditions before the installation of sacrificial anodes to values that correspond to 'low to moderate' corrosion conditions in the portion of the slab that was admixed with 0.6% chloride. The average corrosion rates in the portion that was admixed with 1.8% chloride reduced from values that correspond to 'high corrosion' conditions, before the installation of sacrificial anodes, to values that correspond to 'moderate to high' corrosion conditions after the installation of discrete sacrificial anodes.

Quantitatively, the average corrosion rate after the installation of sacrificial anodes in slab E reduced by 0.28µA/cm^2 (i.e., 0.38 – 0.10µA/cm^2 or 74%) and by 0.28µA/cm^2 (i.e., 0.52 - 0.24µA/cm^2 or 54%) in the slab portions admixed with 0.6% and 1.8% chloride respectively. Similarly, the average corrosion rate after the installation of discrete sacrificial anodes in slab G decreased by 0.39µA/cm^2 (i.e., 0.55µA/cm^2 – 0.16µA/cm^2 or 72%) and by 0.82µA/cm^2 (i.e., 1.50µA/cm^2 – 0.68µA/cm^2 or 54%) in the slab portions admixed with 0.6% and 1.8% chloride respectively. Changes in average corrosion rates in slabs F and H are presented in Appendix B.2, Table B.1.

The observed reductions in average corrosion rate in test slabs E and G can be attributed to the sacrificial anodes that were installed in these slabs. The percentage reduction in the values of corrosion rate within the portions of slabs E and G that were admixed with 0.6% chloride were higher than in the portions that were admixed with 1.8% chloride. This observation can be attributed to the greater driving force that induces current flow to the anodic sites than in the cathodic sites of the slab macrocell. This phenomenon has been described in Section 5.1.1. The percentage reduction in the values of average corrosion rate within the portions of slabs E and G that were admixed with 0.6% chloride were almost the same (i.e., 75% and 72% respectively). Similarly, the percentage reduction in the values of average corrosion rate within the portions of slabs E and G that were admixed with 1.8% chloride were equal (i.e., 54%). While the explanation to this observation vis-à-vis those made in the slabs cast using 100% PC could not be found; it can be inferred, that the level of chloride contamination is the dominant parameter that determines the extent to which sacrificial anodes can reduce the corrosion rate of steel in concrete that has been cast using a blend of 70/30 PC/FA.

5.1.3. Combined corrosion rate measurements

A comparison of changes in average corrosion rates in the test slabs before and after the sacrificial anodes were installed is shown in Figure 5.5. From Figure 5.5, it is evident that the greatest percentage reductions in average corrosion rate were experienced in slabs that were cast using 100% PC. This observation can be attributed to the low electrical resistivity of concrete made from 100% PC, which enhances the ease with which galvanic currents generated by the discrete sacrificial anodes flow through the concrete. Throughout the slabs, with the exception of slab C, the percentage reductions in average corrosion rate were highest in the portions of the slabs that were admixed with 0.6% chloride.

The relationship between the percentage reduction in average corrosion rate and the amount of direct anodic current used to induce corrosion within the test slabs could not be established. The reductions in average corrosion rate seem to be independent of the amount of direct anodic current that was used to induce corrosion. From the figures above, it is clear that the installation of sacrificial anodes resulted in reductions in corrosion rate. These reductions in corrosion rate would be attributed to the primary effects of cathodic protection. These effects include: the reduction in the concentration of chloride ions at the rebar surface as a result of electromigration or the shifting of the half-cell potential of steel to values that hinder either the initiation and/or propagation of corrosion pits (Sergi and Page, 1999;

Bertolini *et al*., 2009). The reduction in corrosion rate of steel would also be attributed to the secondary effect of cathodic protection; namely: the production of hydroxyl ions (increase in alkalinity) at the steel-concrete interface (Pedeferri, 1996; Glass *et al*., 2000). Parameters, such as the removal of chloride ions and the production of hydroxyl ions (increase of alkalinity) at the steel surface were not tested due to lack of testing equipment.

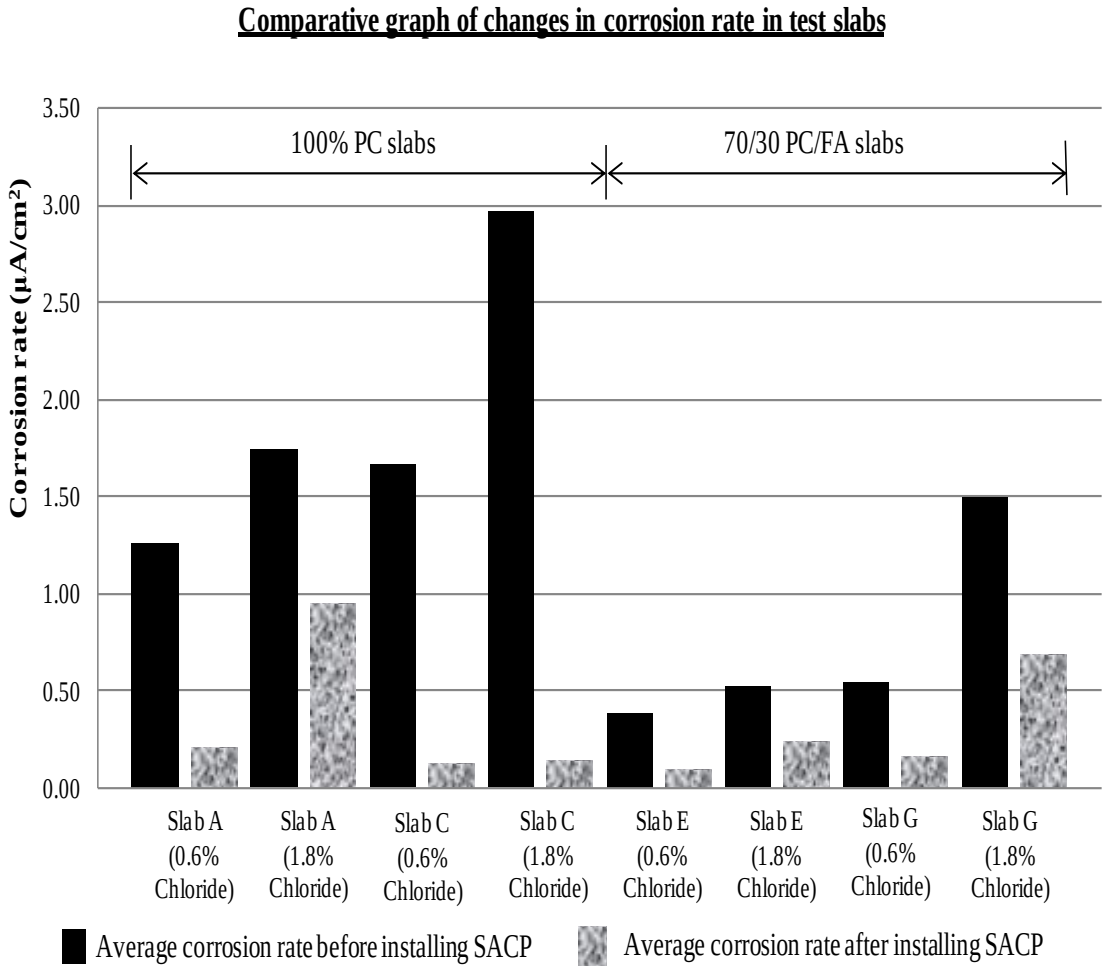


Figure 5.5: Comparison of changes in average corrosion rates in test slabs during the test period

The fact that there were reductions in average corrosion rate in slabs within which sacrificial anodes were not installed implies that the reductions in corrosion rate could also be attributed to changes in the electrochemical properties of concrete (especially the concrete resistivity). The trend lines that pertain to the concrete resistivity are presented in Appendix B.4. The trend lines of concrete resistivity show that all slabs experienced an increase in concrete resistivity (or reduction in concrete conductivity). The increase in concrete resistivity could have been caused by the consumption of the chemically bound water (pore water) when concrete cures (Parthiban *et al.*, 2008b). A summary of the numeric changes in corrosion rate in all the slabs are presented in Appendix B.2 (Table B.1).

5.2. Half-cell potential measurements

The half-cell potentials (HCP) were measured using a silver/silver chloride half-cell as described in Chapter 4 (Section 4.8.2.2). The results from these measurements are presented and discussed within this section. The conventions that were used in the previous section (Section 5.1) have also been used in this section. The average half-cell potential after the withdrawal of the direct anodic current (or before the installation of the discrete sacrificial anodes) was calculated as the mean of all half-cell potential values taken during the 7 weeks (49 days) after the removal of the direct anodic current. In order to avoid the possibility of underestimating the effect of the sacrificial anodes on half-cell potentials, the average half-cell potentials after the installation of sacrificial anodes was calculated as the mean of the half-cell potential measurements taken during the last four weeks (28 days).

5.2.1. HCP measurements in 100% PC slabs

The changes in half-cell potential within the slabs upon which a direct anodic current of 41.48µA was applied (i.e., slabs A and B) are shown in Figure 5.6. Similarly, the changes in half-cell potential within the slabs upon which a direct anodic current of 14.14µA was applied (i.e., slabs C and D) are shown in Figure 5.7.

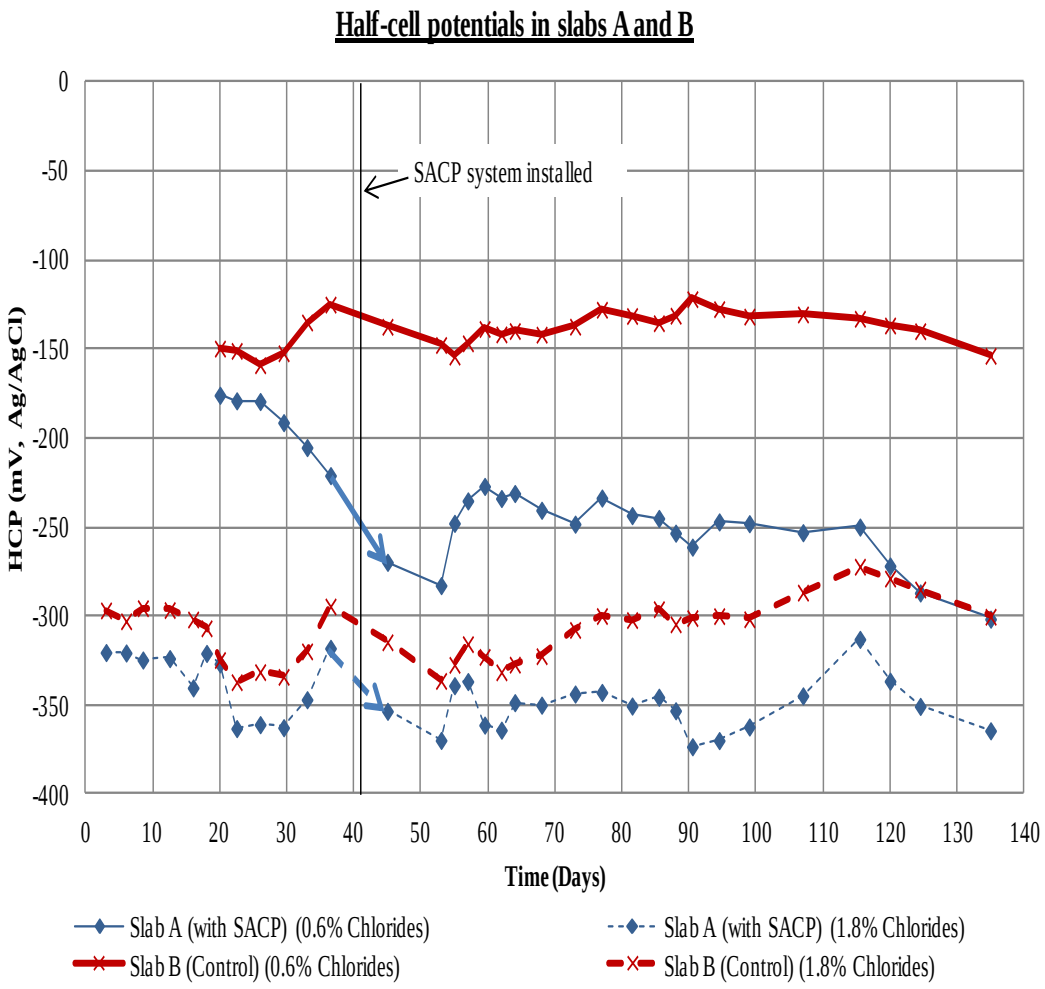


Figure 5.6: Trend lines of HCP in slabs A and B (100% PC)

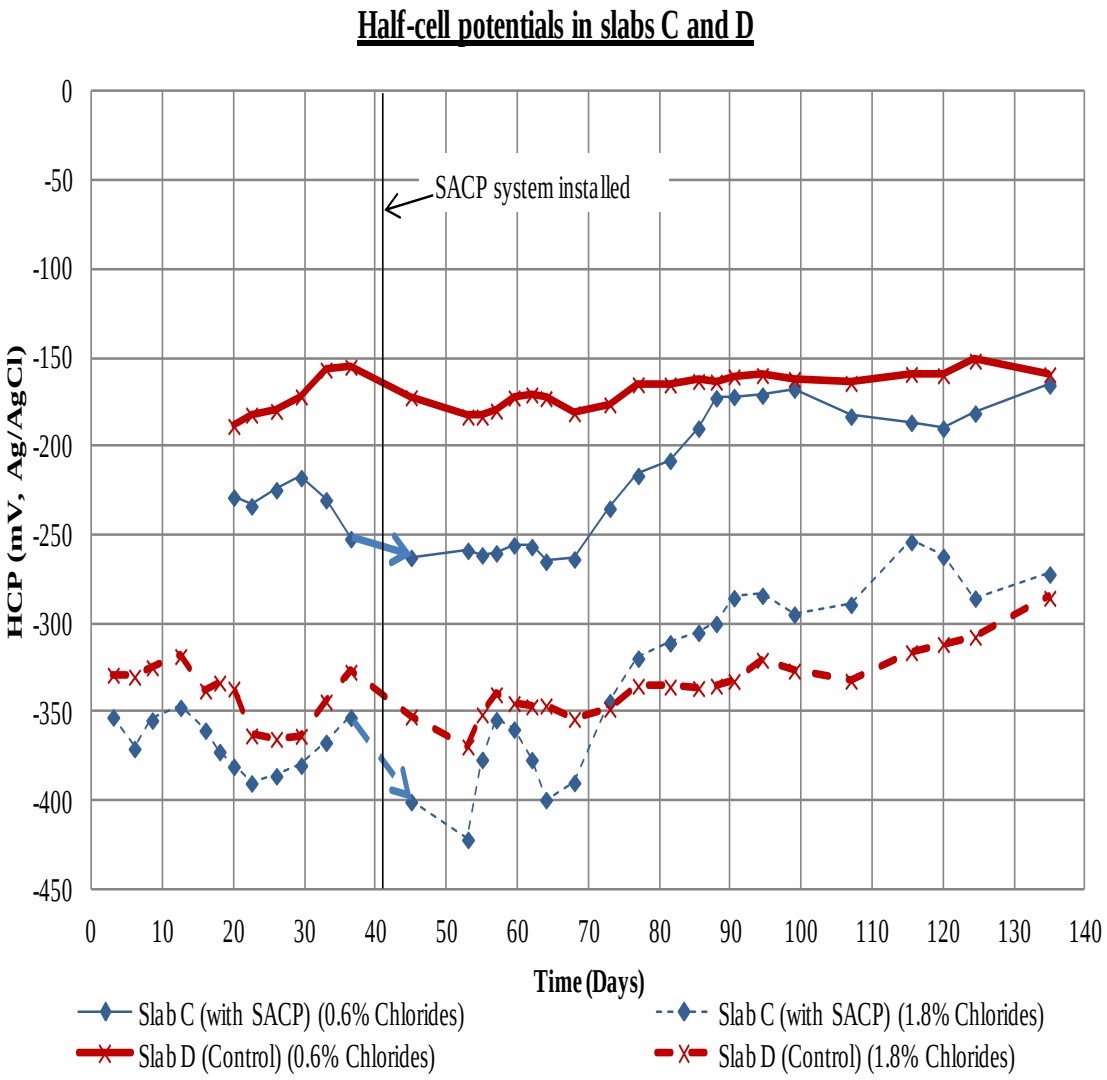


Figure 5.7: Trend lines of HCP in slabs C and D (100% PC)

From Figure 5.6 and Figure 5.7, it can be observed that the average half-cell potential of steel within the portion of the slab that was admixed with 1.8% chloride is more negative than in the portion admixed with 0.6% chloride. Whereas the average half-cell potentials of steel in the test slabs (i.e., slab A and C) became more negative with time than in the control slab (i.e., slab B and D); the average half-cell potential of steel in slab C became less negative with time as shown in Figure 5.7.

The shift in the half-cell potential in slab A towards values that are more negative vis-à-vis the corresponding reduction in corrosion rate within the same slab is consistent with the theory of the Pourbaix diagram of steel (see Figure 2.1) and the principles of cathodic protection (see Section 3.2). Thus, it can be inferred that the installed discrete sacrificial anodes resulted in a shift in half-cell potentials towards values that are more negative, hence reducing the corrosion rate of steel. The shift in the half-cell potential of steel towards values that are less negative vis-à-vis the corresponding reduction in corrosion rate in slab C was not consistent with the theory of the Pourbaix diagram of steel as well as the principles of

cathodic protection. A review of literature, however, attributed this observation to the polarisation of the anodic reaction kinetics (Glass *et al*., 2000). The polarisation of the anodic reaction kinetics has been reported to shift the potential of steel towards more positive values while resulting in a reduction in corrosion rate. This phenomenon (i.e., the polarisation of the anodic reaction kinetics), however, could not be affirmed due to lack of adequate instrumentation.

The quantitative changes in average half-cell potentials, before and after the installation of sacrificial anodes, in the test slabs cast using 100% PC are as follows:

i. The average half-cell potentials of steel in slab A became more negative with time. They changed by -94mV (i.e., from -193mV before the installation of sacrificial anodes to -287mV after the installation of sacrificial anodes) and by -15mV (i.e., from -335mV before the installation of the sacrificial anodes to -350mV after the installation of sacrificial anodes) in the portion of the slab that was admixed with 0.6% and 1.8% chloride respectively.

ii. The average half-cell potentials of steel in test slab C became less negative with time. They changed by 56mV (i.e., from -233mV before the installation of sacrificial anodes to -177mV after the installation of sacrificial anodes) and by 98mV (i.e., from -365mV before the installation of sacrificial anodes to -267mV after the installation of sacrificial anodes) in the portion of the slab that was admixed with 0.6% and 1.8% chloride respectively.

A summary of the quantitative changes in half-cell potential of slabs B and D are presented in Appendix B.3 (Table B.2). The magnitude of the changes in half-cell potential in slab A was greater in the portion that was admixed with 0.6% chloride than in the portion admixed with 1.8% chloride. This trend (i.e., the higher reduction in corrosion rate within the portion of the slab that was admixed with 0.6% chloride) is similar to that observed with respect to the reduction in corrosion rate and can be attributed to the greater driving force inducing current flow to the anodic sites than in the cathodic sites as discussed in Section 5.1.1. The greater flow of current to the cathodic sites implies that there were a lot of electrons moving towards the cathodic sites. The movement of electrons; therefore, resulted into higher negative half-cell potentials.

5.2.2. HCP measurements in 70/30 PC/FA slabs

The changes in HCP within the slabs upon which a direct anodic current of 41.48µA was applied (i.e., slabs E and F) are shown in Figure 5.8.

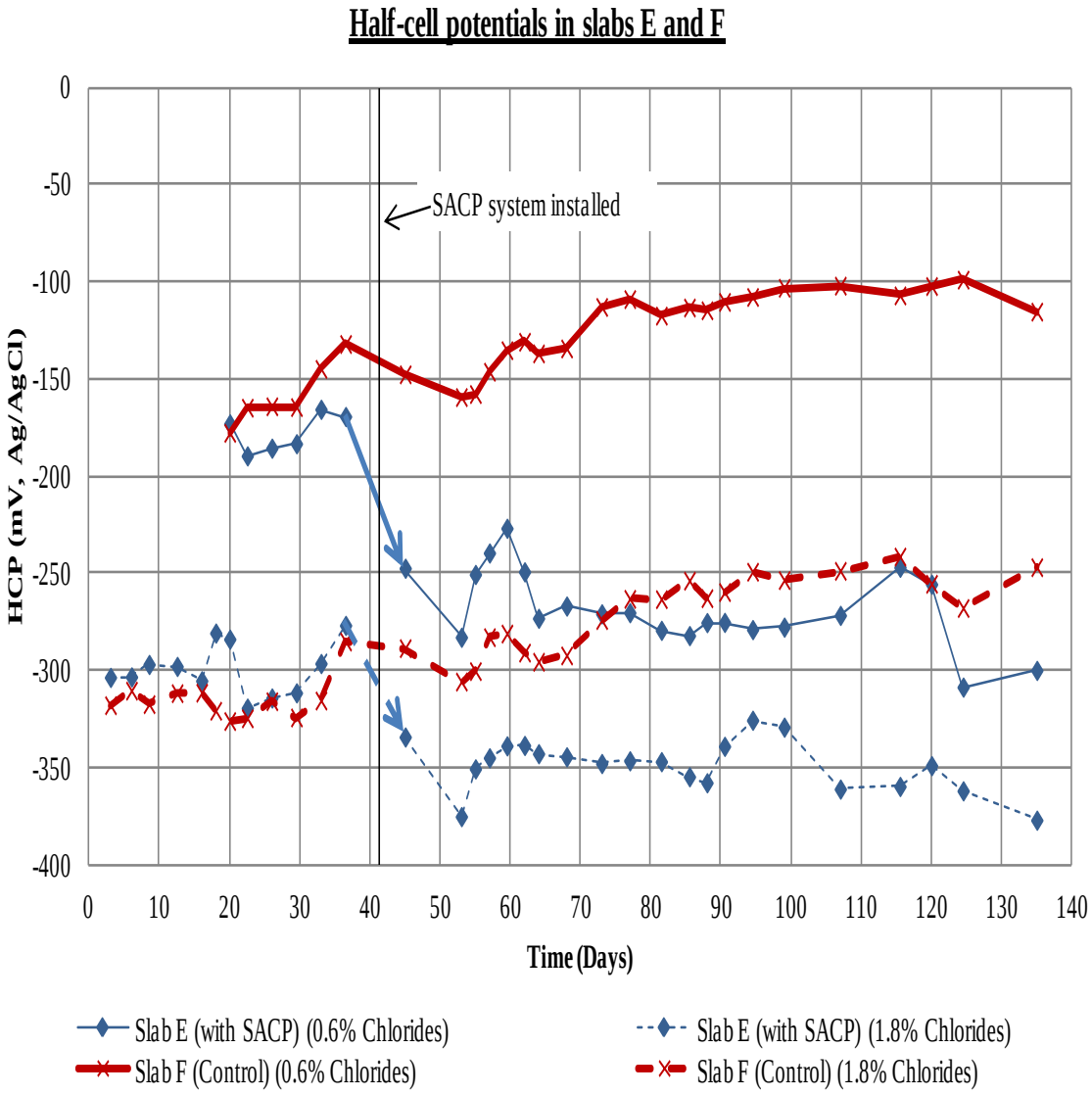


Figure 5.8: Trend lines of HCP in slabs E and F (70/30 PC/FA)

The changes in HCP within the slabs upon which a direct anodic current of 14.14µA was applied (i.e., slabs G and H) are shown in Figure 5.9. From Figure 5.8 and Figure 5.9, it can be seen that the average half-cell potentials of steel within the slab portions admixed with 1.8% chloride were more negative than within the portion admixed with 0.6% chloride. The reasons behind these observations have been presented in Section 5.2.1. Similarly, the average half-cell potentials of steel in the test slabs (i.e., slab E and G) were more negative than the corresponding values in the control slabs (i.e., slab F and H).

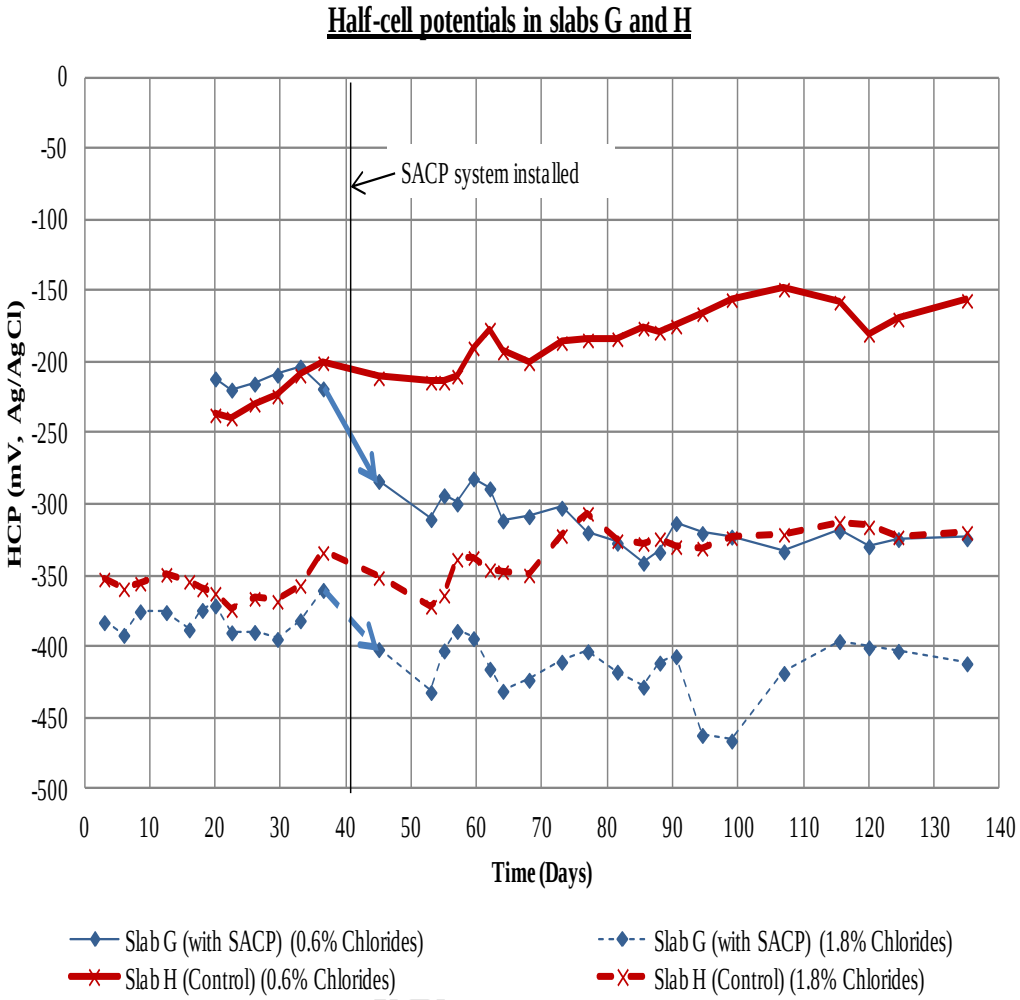


Figure 5.9: Trend lines of HCP in slabs G and H (70/30 PC/FA)

The quantitative changes in average half-cell potentials, before and after the installation of sacrificial anodes, in the test slabs cast using a 70/30 PC/FA blend are as follows:

i. The average half-cell potential of steel in slab E became more negative. Half-cell potentials changed by -101mV (i.e., from -177mV before the installation of sacrificial anodes to -278mV after the installation of sacrificial anodes) and by -65mV (i.e., from -298mV before the installation of sacrificial anodes to -363mV after the installation of sacrificial anodes) in the portion of the slab that was admixed with 0.6% and 1.8% chloride respectively.

ii. The average half-cell potential of steel in slab G became more negative. The half-cell potential of steel changed by-113mV (i.e., from -213mV before the installation of sacrificial anodes to -326mV after the installation of sacrificial anodes) and by -27mV (i.e., from -379mV before the installation of sacrificial anodes to -406mV after the installation of sacrificial anodes) in the portion of the slab that was admixed with 0.6% and 1.8% chloride respectively.

A summary of the quantitative changes in average half-cell potential of slabs F and H are presented in Appendix B.3 (Table B.2). The shift in the average half-cell potential in the test slabs (i.e., slabs E and G) towards values that are more negative can be attributed to the presence of the installed discrete sacrificial anodes as well as the effects discussed in Section 5.2.1. Also, the changes in average half-cell potential in test slabs E and F were greater in the portion of the slab that was admixed with 0.6% chloride than in the portion that was admixed with 1.8% chloride. A similar trend was observed with respect to the reductions in corrosion rates in slabs cast using 70/30 PC/FA. These phenomena can be attributed to the greater driving force inducing current flow to the anodic sites than in the cathodic sites as discussed in Section 5.1.1. Similarly, as in the case of slabs cast using 100% PC, it can be inferred that the installed discrete sacrificial anodes were effective in reducing the average corrosion rate in the test slabs (i.e., slabs E and G). In addition, the sacrificial anodes were more effective in lowering the average half-cell potential of steel to values that are more negative in the portion of the slab that was admixed with 0.6% chlorides.

5.2.3. Combined HCP measurements

From the figures presented within this section, it is evident that the installation of the discrete sacrificial anodes resulted in changes in the average half-cell potentials. The magnitude of the actual and percentage changes in average half-cell potentials of steel before and after the time of installation of sacrificial anodes within the test slabs cast using 70/30 PC/FA were higher than the corresponding changes in the test slabs cast using 100% PC. The relationship between the changes in average half-cell potentials and the amount of direct anodic current that was used to induce corrosion in the slabs could not be established. Similarly, the average half-cell potentials that were achieved after the installation of the sacrificial anodes were low (-250mV to -410mV). This range of half-cell potentials is consistent with the values that have been reported in literature by de Rincón *et al* (2002).

5.4. Summary

The results and discussions from this study have been presented in the form of tables, and 2-point moving average trend lines. Despite the fact that a specific amount of direct anodic current was used to induce corrosion within a set of slabs connected in series; the average corrosion rates and half-cell potentials achieved within the specific slabs, 7 weeks after the current that was used to induce corrosion was removed, were different. The average corrosion rates achieved after the removal of the direct anodic current were higher in slabs cast using 100% PC than in slabs cast using a blend of 70/30 PC/FA.

The installation of sacrificial anodes resulted in changes in average corrosion rate and half-cell potential in the test slabs. There was a reduction in the average corrosion rate of steel in most test slabs. Quantitatively, the reductions in average corrosion rate ranged between 45-95% in slabs cast using 100%PC and between 54-75% in slabs cast using 70/30 PC/FA. The reduction in average corrosion rate of steel was accompanied by shifts in average half-cell potentials towards values that are more negative. In some slabs, however, the

reduction in average corrosion rate was accompanied by a shift in half-cell potential towards values that are less negative. The reduction in the average corrosion rate of steel in concrete depends on binder type, the amount of admixed chloride, and the resistivity of concrete. The greatest percentage reductions in corrosion rate were observed in slabs cast using 100% PC. Similarly, the portions of the test slabs that were admixed with 0.6% chloride experienced the greatest reduction in average corrosion rate. The average half-cell potentials of steel in the slab portions admixed with 1.8% chloride were more negative than within the portions admixed with 0.6% chloride.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

This study aimed at investigating the use of discrete sacrificial anode cathodic protection systems in service life extension of chloride contaminated reinforced concrete structures in South Africa. This objective was achieved through an investigation into the effectiveness of discrete sacrificial anode cathodic protection systems with respect to the level of chloride contamination, induced corrosion rate and binder type. The following conclusions can be drawn from the data that was collected and analysed.

6.1.1. The influence of SACP systems on corrosion rates

Discrete SACP systems can result in an extension in the service life of chloride contaminated RC structures that are exhibiting rebar corrosion. The discrete sacrificial anodes reduced the corrosion rate of steel in chloride contaminated reinforced concrete slabs. The extent to which these anodes reduce the corrosion rate of steel depends on the binder type and the level of chloride contamination. The installed discrete SACP system was more effective in reducing the corrosion rate of steel in the test slabs cast using 100% PC than in the test slabs cast using 70/30 PC/FA. The percentage reductions in corrosion rate within the test slabs cast using 100% PC ranged between 45%-95%. Similarly, the percentage reductions in corrosion rate within the test slabs cast using 70/30 PC/FA ranged between 54%-75%.

Discrete sacrificial anodes appear to be more effective in reducing corrosion rate in reinforced concrete structures that are contaminated with less chloride. The percentage reduction in average corrosion rate was higher within the portion of the slab that was admixed with 0.6% chloride than in the portion that was admixed with 1.8% chloride. The extent to which discrete SACP systems reduce the corrosion rate of steel in chloride contaminated RC structures seems to be independent on the prevailing corrosion rate (i.e., the corrosion rate of steel before the installation of the discrete SACP system). Whereas two distinct direct anodic currents were used to induce corrosion in the slabs, a relationship between the applied direct anodic current (used to induce corrosion) and the corresponding percentage reduction in corrosion rate could not be established from the analysed results.

6.1.2. The influence of SACP systems on half-cell potentials

Discrete SACP systems cause changes in average half-cell potential. However the shift in the half-cell potential of steel towards a value that is either more negative or more positive depends on the influence of the installed sacrificial anode on the cathodic or anodic reaction kinetics. The installation of discrete sacrificial anode CP systems shifted the average half-cell potentials of steel in most slabs towards more negative values. The percentage changes in average half-cell potential, however, were not as profound as the corresponding changes in average corrosion rate; thus supporting the facts in literature that sacrificial anodes do not usually result in profound changes in the half-cell potential of reinforcing steel. The inability of sacrificial anodes to result in profound changes in half-cell potential has always been

posited as their major setback. In addition, the fact that high reductions in average corrosion rate were achieved despite the small changes in average half-cell potential rate implies that half-cell potential measurements, though important, should not be used as the only criteria for evaluating the effectiveness of sacrificial anode systems. The evaluation of sacrificial anodes ought to incorporate half-cell potential as well as corrosion rate measurements.

The changes in the half-cell potential towards more negative values is greater in concrete cast using a blend of 70/30 PC/FA than in concrete cast using 100% PC. The relationship between changes in half-cell potential and the amount of current that was used to induce corrosion could not be established from the analysed data. Nevertheless, the changes that discrete sacrificial anodes cause in half-cell potential are dependent on the level of chloride contamination. The greatest shifts in average half-cell potential in the test slabs, towards more negative values, was experienced within the portions of the slabs that were admixed with 0.6% chloride.

6.1.3. Miscellaneous

The amount of corrosion that can be induced in reinforced concrete specimens using a direct anodic current is a highly stochastic process that varies with the degree of contamination by anionic species (e.g., chloride ions) and the type of binder in the concrete. In addition, the use of a constant amount of current to induce corrosion across a number of specimens does not guarantee the attainment of the same amount of corrosion rate across the specimens. Specimens cast using 100% PC achieve corrosion rates that are relatively higher than the ones achieved in specimens cast using a blend of 70/30 PC/FA. Similarly, the portions of the slabs that were contaminated with 1.8% chloride by mass of binder achieved corrosion rates that were higher than the corrosion rates achieved in the portions of the slabs admixed with 0.6% chloride by mass of binder.

6.2. Recommendations

The quality of the data and results that were collected and analysed in this study could have been improved through the use of accurate and reliable corrosion measurement and monitoring devices such as macrocell, temperature and half-cell potential sensors. Such sensors are capable of monitoring corrosion rate and other parameters continuously and more accurately, thus improving the quality of output from this study. Also, it is evident that there is need for further research into the effectiveness of SACP systems. The following are the principal further research works that need to be carried out in this broad research area:

i. Long-term monitoring of the installed discrete SACP systems
ii. The incorporation of SACP system effects in service life modelling, i.e., quantifying the service life extension in a given RC structure as a result of installing SACP
iii. Testing and monitoring the 4 hour and 24 hour depolarisation of the discrete anodes
iv. The influence of parameters such as water-to-binder ratio, binder types and binder proportions (e.g., slag) on the performance and effectiveness of SACP systems

v. The influence of discrete sacrificial anodes on the distribution of chloride ions within concrete

vi. A comparison of the effectiveness of various SACP systems

vii. The determination of the upper and lower-bound corrosion rates and chloride contaminations within which SACP systems can operate effectively and give satisfactory results

viii. There is need to establish a formula that can be used to accurately calculate corrosion rates that can be induced in reinforced concrete. The formula should incorporate the type of binder, water-to-binder ratio and other parameters

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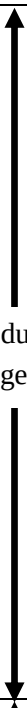
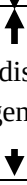
APPENDIX

A. The electrochemical series and standard electrode potential of metals

The electrochemical series and standard electrode potential of common metals is shown in Table A.1.

Table A.1: Electrochemical series and standard electrode potential of common metals

Electrode	Oxidation reaction	Standard Electrode potential (Volts)	Nature
$Li\|\ Li^+$	$Li \rightarrow Li^+ + e^-$	+3.040	 Reducing agents
$K\|\ K^+$	$K \rightarrow K^+ + e^-$	+2.924	
$Ca\|\ Ca^{2+}$	$Ca \rightarrow Ca^{2+} + 2e^-$	+2.870	
$Na\|\ Na^+$	$Na \rightarrow Na^+ + e^-$	+2.710	
$Al\|\ Al^{3+}$	$Al \rightarrow Al^{3+} + 3e^-$	+1.660	
$Zn\|\ Zn^{2+}$	$Zn \rightarrow Zn^{2+} + 2e^-$	+0.762	
$Fe\|\ Fe^{2+}$	$Fe \rightarrow Fe^{2+} + 2e^-$	+0.441	
$Cd\|\ Cd^{2+}$	$Cd \rightarrow Cd^{2+} + 2e^-$	+0.403	
$Ni\|\ Ni^{2+}$	$Ni \rightarrow Ni^{2+} + 2e^-$	+0.236	
$Sn\|\ Sn^{2+}$	$Sn \rightarrow Sn^{2+} + 2e^-$	+0.140	
$Pb\|\ Pb^{2+}$	$Pb \rightarrow Pb^{2+} + 2e^-$	+0.126	
$Pt\|\ H_2H^+$	$H_2 \rightarrow 2H^+ + 2e^-$	0.000	
$Cu\|\ Cu^{2+}$	$Cu \rightarrow Cu^{2+} + 2e^-$	-0.337	 Oxidising agents
$Ag\|\ Ag^+$	$Ag \rightarrow Ag^+ + e^-$	-0.799	
$Hg\|\ Hg^+$	$Hg\ (l) \rightarrow Hg^{2+} + 2e^-$	-0.920	

B. Corrosion monitoring

The calculations and a summary of the results that were obtained from corrosion monitoring are presented within this section.

B.1. Induced corrosion rate calculations

To achieve the corrosion rates that this study had postulated, the amount and duration of application of current were calculated as follows:

The corrosion rate was converted into a physical quantity (i.e., the amount of material that is lost) using the formula below (Bertolini *et al*., 2004a):

$$\mathbf{1\mu A/cm^2 = 11.6\mu m/year} \quad \text{(B.1)}$$

Using the relationship above, the corrosion rate that had been targeted can be converted into the weight of material lost as follows:

$0.75\mu A/cm^2 = (0.75*11.6) =$ **8.7µm/year** (B.2)

$2.2\mu A/cm^2 = (2.2*11.6) =$ **25.52µm/year** (B.3)

Also, to determine the amount of material that has been lost, the following formula was used

$$\mathbf{mm/year = 87.6*(W/DAT)} \quad \text{(B.4)}$$

Where:

W = weight loss in milligrams

D = metal density in g/cm^3

A = area of the sample in cm^2

T = time of exposure of the metal sample in hours

The weight of the material (W_t) that has been lost (in grams) can be calculated using Faraday's law of electrolysis as follows:

Given,

Density of iron = 7.87 g/cm^3

Faraday's constant = 96487 Amp.sec

Atomic weight of iron = 55.847

No. of valence electrons in iron = 2

Area of embedded steel = 1884.96 mm^2

$$\mathbf{W_t = (t*i*55.487)/2*96487} \quad \text{(B.5)}$$

The parameters, t and i, represent the time and amount of current that will be applied to achieve the desired corrosion rate. These values can be obtained by varying one of them. The time of application of the current was fixed at 7 days. Thus, the amount of material that was lost/weight loss in one week as well as the amount of current that was required to achieve the desired corrosion rate is calculated as follows:

For a corrosion rate of 0.75µA/cm^2

Weight loss, W_t = (0.087*7.87*1884.96*[7*24])/87.6 = **2.475mg**

Current, i = (2*96487*2.475*0.001)/(7*24*3600*55.847) = **14.14µA (for 1 week)**

For a corrosion rate of 2.2µA/cm^2

Weight loss, W_t = (0.02552*7.87*1884.96*[7*24])/87.6 = **7.260mg**

Current, i = (2*96487*7.260*0.001)/(7*24*3600*55.847) = **41.48µA (for 1 week)**

B.2. Corrosion rate measurements

A summary of the average corrosion rates in the slabs is shown in Table B.1.

Table B.1: A summary of corrosion rate measurements

Parameter		**Slab ID**							
Slab type and designation		Test slabs				Control slabs			
		A	C	E	G	B	D	F	H
Binder type		100% PC		70/30 PC/FA		100% PC		70/30 PC/FA	
Average corrosion rate after withdrawal of current (µA/cm^2)									
Portion of slab	0.6% admixed chloride	1.26	1.67	0.38	0.55	0.91	1.30	0.37	0.38
	1.8% admixed chloride	1.74	2.97	0.52	1.50	2.55	2.77	0.74	0.61
Average corrosion rate after installation of sacrificial anodes (or from day 41) (µA/cm^2)									
Portion of slab	0.6% admixed chloride	0.20	0.12	0.10	0.16	0.68	1.35	0.21	0.24
	1.8% admixed chloride	0.94	0.14	0.24	0.68	2.13	3.41	0.36	0.73
Change in corrosion rate (%) after installation of anodes									
Portion of slab	0.6% admixed chloride	-84	-93	-75	-72	-25	4.5	-44	-38
	1.8% admixed chloride	-46	-95	-54	-54	-17	23	-51	21

NB: A negative change in corrosion rate implies a decrease in corrosion rate.

B.3. Half-cell potential measurements

A summary of the changes in half-cell potentials during the duration of test summarised in Table B.2.

Table B.2: Combined HCP measurements in the slabs

Parameter		**Slab ID**							
Slab type and designation		**Test slabs**				**Control slabs**			
		A	C	E	G	B	D	F	H
Binder type		100% PC		70/30 PC/FA		100% PC		70/30 PC/FA	
Average potential before the installation of sacrificial anodes (mV)									
Portion of slab	0.6% admixed chloride	-193	-233	-177	-213	-145	-173	-159	-222
	1.8% admixed chloride	-335	-365	-298	-379	-310	-338	-314	-356
Average potential after installation of sacrificial anodes (or after day 41) (mV)									
Portion of slab	0.6% admixed chloride	-287	-177	-278	-326	-138	-168	-122	-181
	1.8%	-350	-267	-363	-406	-307	-333	-269	-332
Change in average potential (mV)									
Portion of slab	0.6% admixed chloride	94	-56	101	114	-7	-5	-37	-41
	1.8% admixed chloride	15	-99	65	26	-3	-5	-45	-24

NB: A negative change in potential implies the shift in potential to values that are less negative.

Change in average. Potential = Average potential before the installation of SACP (or day 41 in the case of control slabs) – average potential after the installation of SACP (or day 41 in the case of control slabs).

B.4. Concrete resistivity measurements

The changes in concrete resistivity within the 100% PC slabs upon which a direct anodic current of 41.48µA was applied (i.e., slabs A and B) are shown in Figure B.1.

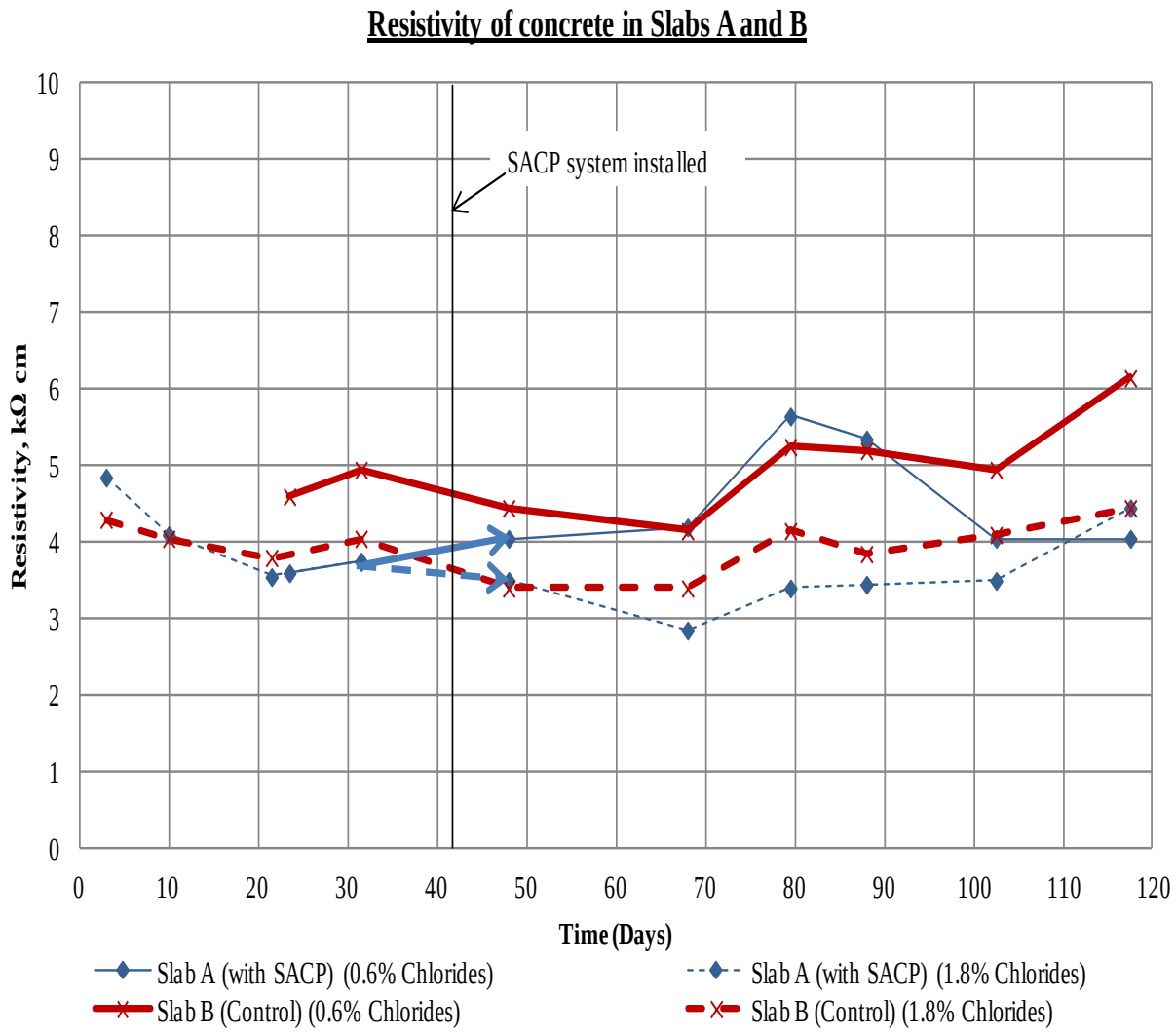


Figure B.1: Trend lines of concrete resistivity in slabs A and B (100% PC)

The changes in concrete resistivity within the 100% PC slabs upon which a direct anodic current of 14.14µA was applied (i.e., slabs C and D) are shown in Figure B.2.

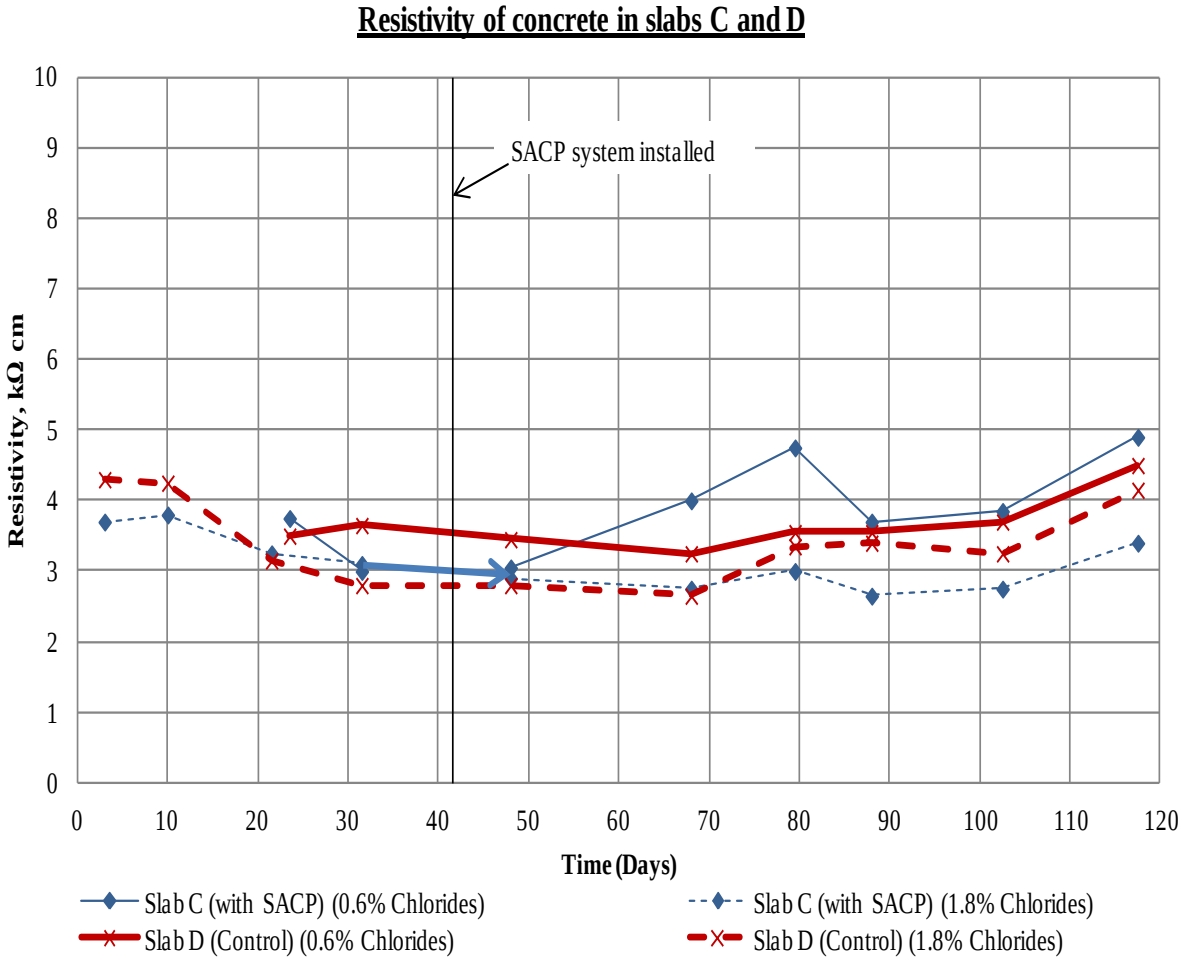


Figure B.2: Trend lines of concrete resistivity in slabs C and D (100% PC)

The changes in concrete resistivity within the 70/30 PC/FA slabs upon which a direct anodic current of 41.48µA was applied (i.e., slabs E and F) are shown in Figure B.3.

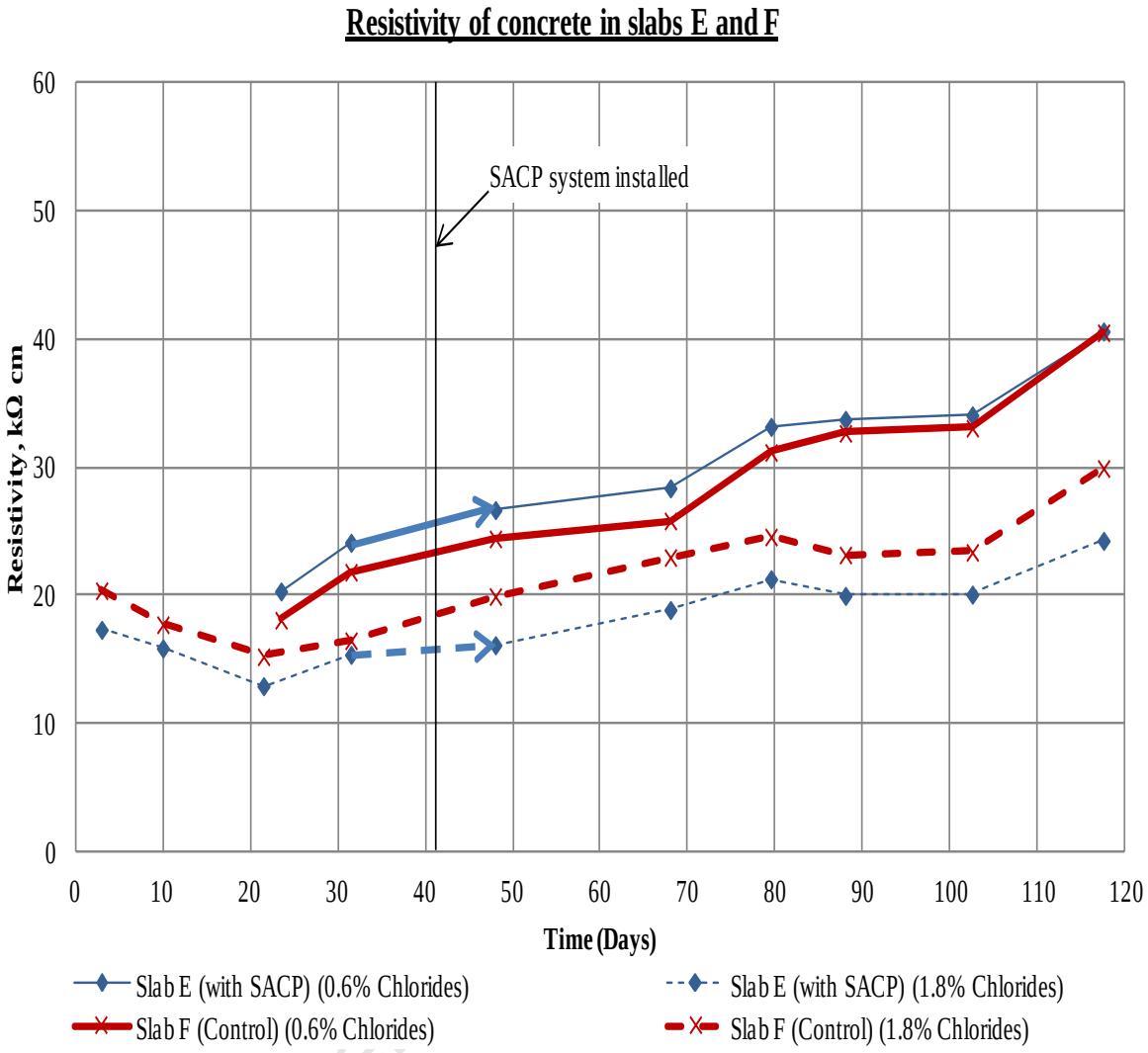


Figure B.3: Trend lines of concrete resistivity in slabs E and F (70/30 PC/FA)

The changes in concrete resistivity within the 70/30 PC/FA slabs upon which a direct anodic current of 14.14µA was applied (i.e., slabs G and H) are shown in Figure B.4.

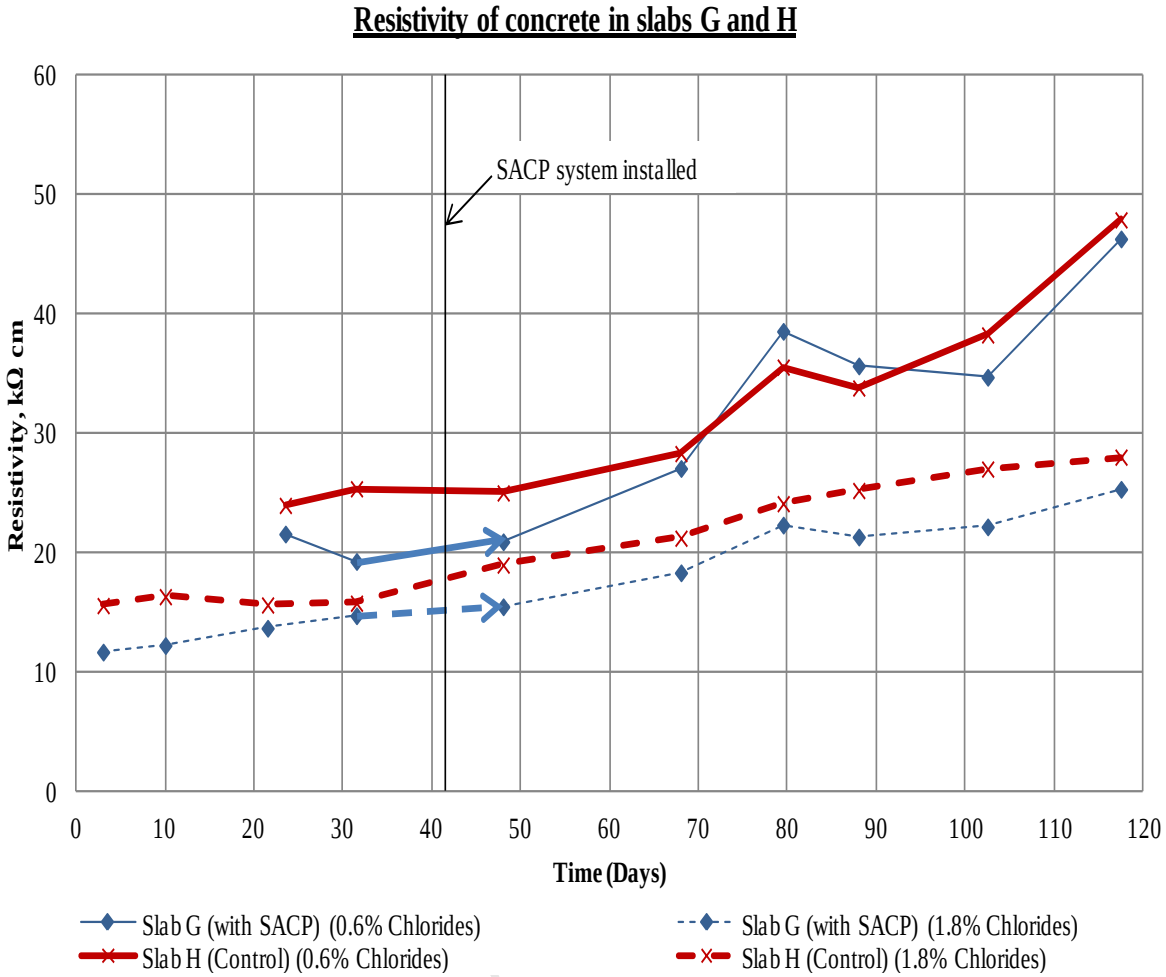


Figure B.4: Trend lines of concrete resistivity in slabs G and H (70/30 PC/FA)

C. Corrosion rate and concrete resistivity interpretation guidelines

The criteria for the interpretation of corrosion rate are presented in Table C.1 and Table C.2.

Table C.1: Criteria for interpretation of corrosion rate from linear polarisation measurements from sensor controlled guard ring device (Gu *et al*., 2001, Grantham, 2003 and Broomfield, 2007)

I_{corr} (µA/cm^2)	**Condition of steel**
< 0.1	Passive condition
0.1-0.5	Low to moderate corrosion
0.5-1	Moderate to high corrosion
> 1	High corrosion rate

Table C.2: Interpretation of concrete resistivity measurements from a four-probe system (Broomfield, 2007; Grantham, 2003 and Bungey *et al*., 2006)

Resistivity range (KΩ cm)	**Rate of corrosion**
> 20	Low corrosion rate
10-20	Low to moderate corrosion rate
5-10	High corrosion rate
< 5	Very high corrosion rate

D. Temperature and relative humidity measurements

The variations in temperature and relative humidity are presented in Table D.1.

Table D.1: Variations in temperature and relative humidity during the test duration

Time (Days)	Relative humidity (%)	Temperature (^{o}C)
1	59.0	15.0
5	72.0	13.5
7	56.7	16.8
10	33.2	23.8
15	41.3	19.5
17	47.9	16.8
19	62.8	11.5
21	52.7	18.5
24	70.6	23.9
28	51.5	17.5
31	32.9	25.1
35	61.5	21.2
38	52.8	19.3
52	41.7	22.7
54	53.3	22.4
56	63.1	18.8
58	51.8	18.2
61	42.7	28.1
63	61.0	22.2
65	65.4	20.8
71	39.8	23.5
75	57.1	25.6
79	67.1	20.3
84	70.3	24.3
87	63.4	24.3
89	35.0	27.1
92	36.2	28.6
97	46.7	26.3
101	52.7	23.6
113	39.6	27.5
118	51.3	24.7
122	70.6	19.7
127	39.6	23.3
143	81.8	17.1

E. Concrete properties

The properties of the concrete that was used to make the test slabs are given below.

E.1. Slabs cast using 100% PC

The concrete that was used to cast slabs A, B, C and D was tested for compressive strength and chloride conductivity at 28 days.

E.1.1. Compressive strength results

The tests for 28-day compressive strength were performed according to SANS 5863:2006. The results from these tests are presented in Table E.1.

Table E.1: 28 day compressive strength results for 100% PC (w/b=0.47)

Cube ID	Mass (g)	Cube Density (Kg/m^3)	Mean density (Kg/m^3)	f_{cu} (MPa)	Mean f_{cu} (MPa)	Standard deviation
Cube 1	2390	2390	2385	38.8	40.0	1.08
Cube 2	2395	2395		40.3		
Cube 3	2370	2370		40.9		

E.1.2. Chloride Conductivity Index (CCI) results

The tests for chloride conductivity were performed according to the Durability Index Testing Procedure Manual 2010 (Ver 2.0, May 2010). The results from the 28-day chloride conductivity index results are presented in Table E.2 and Table E.3.

Table E.2: 28-day CCI results for concrete admixed with 0.6% chloride

Sample	Sample 1	Sample 2	Sample 3	Sample 4
Oven dry mass (g)	251.73	253.60	257.76	250.47
Saturated Mass (g)	263.60	265.71	269.83	262.05
Thickness (mm)	29.86	30.22	30.38	29.53
Diameter (mm)	68.51	68.40	68.43	68.48
Voltage (V)	10	10	10	10
Current (mA)	104.4	110.3	108.5	108.7
Conductivity (mS/cm)	0.85	0.91	0.90	0.87

Average: 0.088; COV: 3.72; Rating: Good.

Table E.3: 28-day CCI results for concrete admixed with 1.8% chloride

Sample	Sample 1	Sample 2	Sample 3	Sample 4
Oven dry mass (g)	252.93	254.42	254.26	258.73
Saturated Mass (g)	264.81	266.92	266.02	270.30
Thickness (mm)	30.11	30.18	30.17	30.60
Diameter (mm)	68.43	68.43	68.47	68.45
Voltage (V)	10	10	10	10
Current (mA)	111.2	117.0	117.6	107.2
Conductivity (mS/cm)	0.91	0.96	0.96	0.89

Average: 0.94; COV: 3.17; Rating: Good.

E.2. Slabs cast using 70/30 PC/FA

The concrete that was used to cast slabs E, F, G and H was tested for compressive strength and chloride conductivity at 28 and 90 days.

E.2.1. Compressive strength results

The tests for 28 and 90-day compressive strength were performed according to SANS 5863:2006. The results from these tests are presented in Table E.4 and Table E.5 respectively.

Table E.4: 28 day compressive strength results for 70/30 PC/FA (w/b=0.47)

Cube ID	Mass (g)	Cube Density (Kg/m^3)	Mean density (Kg/m^3)	f_{cu} (MPa)	Mean f_{cu} (MPa)	Standard deviation
Cube 1	2425	2425	2400	39.8	40.4	1.55
Cube 2	2405	2405		39.3		
Cube 3	2370	2370		42.2		

Table E.5: 90 day compressive strength results for 70/30 PC/FA (w/b=0.47)

Cube ID	Mass (g)	Cube Density (Kg/m^3)	Mean density (Kg/m^3)	f_{cu} (MPa)	Mean f_{cu} (MPa)	Standard deviation
Cube 1	2395	2395	2408	58.2	58.1	0.1
Cube 2	2405	2405		58.0		
Cube 3	2425	2425		58.1		

E.2.2. Chloride Conductivity Index (CCI) results

The tests for chloride conductivity were performed according to the Durability Index Testing Procedure Manual 2010 (Ver 2.0, May 2010). The results from the 28 and 90-day chloride conductivity index results are presented in Table E.6 – E.9.

Table E.6: 28-day CCI results for concrete admixed with 0.6% chloride

Sample	Sample 1	Sample 2	Sample 3	Sample 4
Oven dry mass (g)	259.29	248.79	255.13	248.99
Saturated Mass (g)	271.41	259.28	266.77	260.48
Thickness (mm)	30.30	28.89	29.74	29.04
Diameter (mm)	68.53	68.42	68.45	68.40
Voltage (V)	10	10	10	10
Current (mA)	45.8	41.9	46.3	46.4
Conductivity (mS/cm)	0.38	0.33	0.37	0.37

Average: 0.36; COV: 7.38; Rating: Excellent

Table E.7: 28-day CCI results for concrete admixed with 1.8% chloride

Sample	Sample 1	Sample 2	Sample 3	Sample 4
Oven dry mass (g)	255.36	254.13	249.65	248.30
Saturated Mass (g)	267.65	266.55	262.00	260.81
Thickness (mm)	30.00	29.77	29.81	29.21
Diameter (mm)	68.45	68.56	68.49	68.42
Voltage (V)	10	10	10	10
Current (mA)	47.7	52.2	57.0	51.3
Conductivity (mS/cm)	0.39	0.42	0.46	0.41

Average: 0.42; COV: 8.55; Rating: Excellent.

Table E.8: 90-day CCI results for concrete admixed with 0.6% chloride

Sample	Sample 1	Sample 2	Sample 3	Sample 4
Oven dry mass (g)	258.97	257.67	256.62	257.32
Saturated Mass (g)	264.80	263.84	262.63	263.55
Thickness (mm)	28.42	28.39	28.28	28.03
Diameter (mm)	69.87	70.10	69.73	69.94
Voltage (V)	10	10	10	10
Current (mA)	12.2	20.0	19.9	16.9
Conductivity (mS/cm)	0.09	0.15	0.15	0.12

Average: 0.13; COV: 25.51; Rating: Excellent.

Table E.9: 90-day CCI results for concrete admixed with 1.8% chloride

Sample	Sample 1	Sample 2	Sample 3	Sample 4
Oven dry mass (g)	251.10	254.19	242.83	250.97
Saturated Mass (g)	258.44	261.08	249.44	258.26
Thickness (mm)	28.23	28.30	27.24	28.37
Diameter (mm)	69.97	69.89	70.07	70.06
Voltage (V)	10	10	10	10
Current (mA)	22.3	21.1	17.3	22.0
Conductivity (mS/cm)	0.16	0.16	0.12	0.16

Average: 0.15; COV: 14.81; Rating: Excellent.

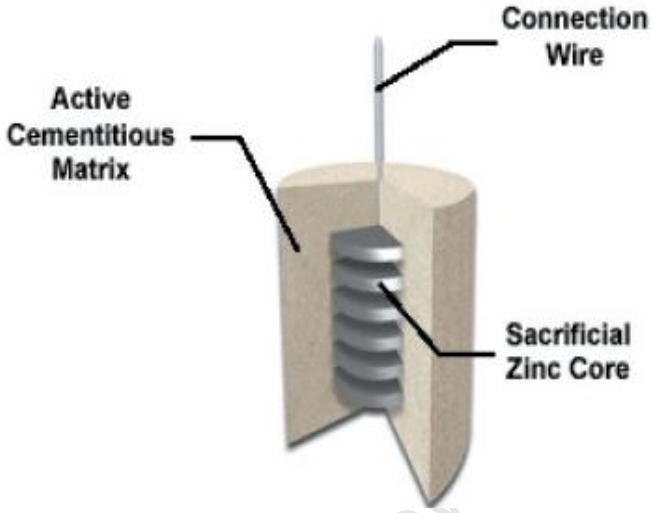
F. Product data sheet for Sika Galvashield CC65

The data pertaining to the properties of the discrete sacrificial anodes are presented in Tables F.1 – F.6. Extra information regarding these anodes can be obtained on: http://www.epms-supplies.co.uk/Admin/Products/Documents/Sika/Data/Sika%20Galvashield%20CC%20PDS.pdf

Table F.1: General information

Product Description	Sika® Galvashield CC embedded galvanic anodes are used to control ongoing corrosion and to prevent the new formation of corrosion sites on reinforced concrete structures. The specially formulated precast cementitious mortar surrounds and activates the sacrificial zinc core. The cylinders are available in a variety of sizes and are quickly and easily installed into concrete to control corrosion activity. When correctly installed and connected the zinc anode corrodes preferentially to the surrounding steel, providing galvanic corrosion prevention and control to the adjacent reinforcement.		
Uses	Reinforced concrete with high chloride content; □post-tensioning anchors; columns and beams; □car parks and bridge decks; □piers & marine structures; □pre-stressed concrete elements; □carbonated concrete		
Characteristics / Advantages	**Proven technology** - supported by independent test program. **Focused protection** - provides localized protection where actual and potential corrosion risks are high. **Economical** - low cost method protecting local areas that are chloride contaminated but sound, thereby reducing concrete breakout. **Versatile** - effective in chloride-contaminated and carbonated concrete. Can be used for both conventionally reinforced and prestressed or post-tensioned concrete. **User friendly** - installation is quick and easy. **Low maintenance** – requires no external power source or system monitoring. **Measurable** – anode performance can be easily monitored if required. **Long lasting** – 10 to 20 year service life* reduces the need for future repairs. **Full system –** can be used in conjunction with Sika FerroGard and SikaGard technology to offer a full corrosion management system. ***** As with all galvanic protection systems, service life is dependent upon a number of factors including reinforcing steel density, concrete conductivity, chloride concentration, humidity and anode spacing		
Level of protection	**Level of protection**	**Description**	**Galvashield CC**
	Corrosion Prevention 0.2 – 2 mA/m^2	Preventing new corrosion activity from initiating	•
	Corrosion Control 1 – 7 mA/m^2	Significantly reducing or stopping on-going corrosion activity	•

Table F.2: Product data

Form	
Appearance	 Fig. F.1: Cut away of Sika® Galvashield® CC
Packaging	Galvashield CC Anodes 20 anodes per box
Storage	
Storage Conditions / Shelf-Life	12 months from date of production if stored properly in original unopened, sealed and undamaged packaging in dry and cool conditions.

The design criteria for the discrete sacrificial anodes are presented in Tables F.3 and F.4.

Table F.3: Anode types and standard units

Sika® Galvashield Type	**Description**	**Unit Size diameter x length (mm)**	**Minimum Hole Size diameter x depth (mm)**
CC65	Standard unit for moderate steel density	46 x 62	50 x 95
CC100	Larger unit for higher steel density	46 x 100	50 x 130
CC135	Slim-fit for congested reinforcement	29 x 135	32 x 165

Table F.4: Maximum grid dimensions for Sika® Galvashield CC65 and CC135

Steel density ratio (steel surface area / concrete surface area)	**Maximum grid dimensions* (mm)**
< 0.2	700
0.21 - 0.4	600
0.41 - 0.54	500
0.55 - 0.67	450
0.68 - 0.80	400
0.81 - 0.94	380
0.95 - 1.07	355
1.08 - 1.2	335

*Maximum grid dimensions are based on typical conditions. Spacing should be reduced as appropriate for severe environments or to extend the expected service life of the anode.

A typical Sika® Galvashield CC series connection layout is shown in Figure F.2.

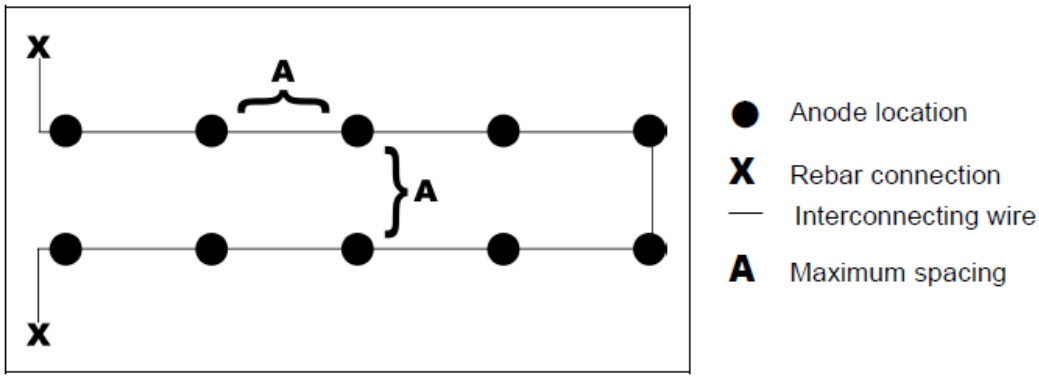


Fig. F.2: Typical series galvanic anode series connection arrangement

Table F.5: Application instructions

Application Method	The location and spacing of the Sika® Galvashield CC anodes shall be on a grid pattern as specified by the engineer. Using a rebar locator, locate all existing steel within the area designated for protection and mark areas to drill unit installation holes. Where possible, anodes should be installed a minimum of 100 mm from reinforcement grid line. Check continuity of the steel. Loss of continuity may require additional electrical connections. **Series Connection** ■ A single circuit shall contain no more than 10 Sika® Galvashield CC anodes. ■ Drill a minimum of two 12 mm rebar connection holes per string of anodes. ■ Saw cut a single continuous groove approximately 6 mm wide by 12 mm deep into the concrete to interconnect rebar connection holes and anode connection holes. **Individual Connection** ■ Drill one rebar connection hole per anode location. ■ Saw cut a groove approximately 6 mm wide by 12 mm deep into the concrete to interconnect the rebar connection hole and anode connection hole. **Connections to Reinforcement** ■ Connections should be made by drilling into reinforcement to fix stainless steel rivet from connection kit .Secure a minimum of 5mm into reinforcement. ■ Fix the interconnecting wire in place with the rivet. ■ Insulate the connection with Sikaflex® 11FC.

	Installation of CC Anodes ■ Drill holes as per the dimensions listed in Anode Types table. ■ Presoak the units for a minimum of 10 to a maximum of 20 minutes in a shallow water bath. ■ Flush drilled holes with water to clean and remove any debris or dust and to pre-wet hole. Remove any pooled water in hole prior to application of embedment mortar. ■ Sika Galvashield Embedding Mortar should be used to install the wet Sika® Galvashield CC anodes into pre-soaked (saturated-surface dry) holes. ■ Place the mixed embedding mortar into the bottom ⅔ of each hole and slowly press in the anode allowing the mortar to fill the annular space ensuring there are no air voids between the anode and the parent concrete. ■ The minimum concrete cover over the top of the anode shall be 20 mm. ■ Connect the Sika® Galvashield CC andoes directly to the rebar connection wire using a suitable electrical wire connector. ■ If installing in series, connect the units to the interconnecting wire with a suitable electrical wire connector. ■ Verify continuity between anode locations and rebar connections with a multi-meter. ■ A resistance of 1 ohm or less is acceptable. ■ Place wires into grooves and level off anode holes and saw cuts flush with the concrete surface with Sika embedment mortar. ■ Sika Galvashield Embedding mortar should be cured similar to Sika concrete repair materials and protected from traffic for 24 hours.

Table F.6: Notes on application

Notes on Application / Limitations	Sika® Galvashield CC anodes are not intended to address or repair structural damage. Where structural damage exists, consult a Structural Engineer. Sika® Galvashield CC anodes are designed to provide galvanic corrosion control. Corrosion control products significantly reduce or stop on-going corrosion. Concrete patch repairs should be completed using Sika concrete repair products in accordance with BS EN 1504-3 and Sika® Galvashield XP anodes installed around the boundary of the patch or Sika® FerroGard® 903 prior to installing Sika® Galvashield CC units in the remaining unrepaired areas. Caution should be exercised when selecting corrosion mitigation systems for posttensioned, pre-stressed or otherwise highly stressed steel. Furthermore, the system of corrosion protection, controls and monitoring should be designed and managed by specialists who can demonstrate expertise and successful project experience.

G. Product data sheet for Sika® Rep LW (Non-sag patching and repair mortar)

The data pertaining to the properties of the repair mortar that was used as a backfill to the discrete sacrificial anodes are presented in Tables G.1-G.8. Extra information regarding the repair mortar can be obtained through the following link:

http://autospec.co.za/productmedia/sika/datasheets/sikareplw.htm

Table G.1: General information

Product Description	A one component, non-sag, cement based, multi-purpose patching and repair mortar.
Uses	For the cosmetic or structural repair of deteriorated concrete and mortar. Sika® Rep LW can be used on, exterior or interior, vertical or overhead surfaces. Sika® Rep LW provides a compatible surface for the application of Sika's coatings and finishes.
Characteristics / Advantages	One component: just add water; easy to mix, apply and finish; suitable for overhead applications; excellent adhesion; shrinkage compensated; compatible with the properties of typical concretes; permeable to water vapour; high resistance to freeze/thaw cycling

Table G.2: Product data

Form	
Appearance	Powder
Colour	Grey
Packaging	25 kg bag
Storage	
Storage conditions	Store in a dry area between +5^{o}C and + 30^{o}C. Protect from direct sunlight.
Shelf-Life	Six months in original unopened packaging

Table G.3: Technical data

Chemical base	Cement and Crystalline free Silica aggregate
Density	1.6 – 2.7 kg/litre
Layer thickness	Min. 10 mm – max. 50 mm

Table G.4: Mechanical properties

Compressive strength	12.0 N/mm^2 @ 24 hours 31.0 N/mm^2 @ 7 days 43.0 N/mm^2 @ 28 days
Bond strength	Concrete substrate failure (Dyna Pull off Adhesion Tester)

Table G.5: System Information

System structure	**Primer / Bonding Coat**: 1 Sika Mono Top® or SikaTop Armatec® 110 EpoCem® **Repair Mortar**: 10 – 50 mm layer thickness
Application Details	
Consumption/Dosage	Volume Yield: 15.3 litres per 25 kg bag
Substrate Quality	The concrete substrate must be sound and sufficient compressive strength (min 20 N/mm^2 (MPa) with a minimum pull of strength of 1.5 N/mm^2 (MPa)). The surface must be dry and free of all contaminants such as oils, grease, coatings and surface treatments, etc. The substrate must be prepared mechanically to remove cement laitance and achieve a profile open textured surface Weak concrete should be removed and surface defects such as honeycombed areas; blowholes and voids must be fully exposed. Repairs to substrate, filling of blowholes/voids and surface levelling should be carried out using the appropriate product from the Sikafloor®, SikaDur® and SikaGard® range of materials
Substrate Preparation / Priming	All surfaces must be clean, sound and properly cured. Remove all loose materials mechanically with a wire brush, or by water or sand blasting. Embedded reinforcing steel should be free from scale, rust, oil and grease and treated with a suitable protective coating such as SikaTop Armatec® 110 EpoCem® The extremities of all repairs should be recessed at least 10 mm to avoid featheredges.

Table G.6: Application Conditions/Limitations

Substrate Temperature	Min. 5^{o}C – max. 30^{o}C
Ambient Temperature	Min 5^{o}C – max. 35^{o}C

Table G.7: Application Instructions

Mixing (Ratio/Dosage)	For Normal applications add 3.8 – 4.0 litres of water per 25 kg bag of Sika Rep LW depending upon the desired consistency. Mixing can be achieved either manually or mechanically. Mechanical mixing is preferred for quantities greater than 1 bag. Mixing should continue until a uniform, lump-free consistency is obtained.
Mixing time	3 minutes
Mixing tools	mechanical slow speed drill
Application method / Tools	Dampen the surface thoroughly with clean water and then prime it with Sika MonoTop® 610 or SikaTop Armatec® 110 EpoCem® depending on the applications. Apply Sika® Rep LW within open time of primer to a maximum thickness of 50 mm per layer and compact thoroughly by trowel.
Cleaning Tools	Remove uncured Sika® Rep LW from tools and equipment with water. Cured material can only be removed mechanically.
Pot Life (max. open time)	50 – 60 minutes after mixing depending on ambient and surface conditions.
Notes on Application / Limits	Freshly applied Sika® Rep LW should be protected from damp, condensation and water for at least 12 hours. Avoid puddles on surface The normal precautions pertaining to dew point should be observed **Protect from rain until initial set has been achieved** Minimum thickness: 10 mm Maximum thickness: 50 mm

Table G.8: Curing details

Curing treatment	Curing is important when ambient conditions may lead to rapid surface drying, the use of light water fogging for 48 hours or Sika® Antisol® curing compounds are recommended. (Do not commence fogging until initial set has been reached; typically 1-2 hours depending upon ambient conditions.)
Local restrictions	Please note that as a result of specific local regulations the performances of this product may vary from country to country. Please consult the local Product Data sheet for the exact description of the application fields.

GLOSSARY

Anodic efficiency: the ratio of the actual coulombs passed to the theoretical coulombs obtainable from the actual weight loss.

Anodic polarization: the polarisation by which the electrochemical state of the metallic component is moved in an anodic (corroding) direction.

Cathodic polarization: the polarisation produced by which the electrochemical state of the metallic component is moved in a cathodic (non-corroding) direction.

Corrosion rate: the volumetric loss of metal by unit of area and unit of time. It is also called corrosion velocity (V_{corr})

Current: a measure of the rate of flow of electricity in a conductor. It is usually expressed in amperes (A) or milliamps (mA).

Current density: the current passing across unit area of surface. It is usually expressed in amperes per square metre (mA/m^2)

Current efficiency: the ratio of the liberated to theoretical mass. In an electrical process, it is the mass of the substance liberated by a current divided by the theoretical mass, as predicted by Faraday's law

Driving potential: the voltage difference between the anode and cathode of a couple

Effective current capacity: the total coulombic charge (current*time) produced by a unit mass of an anode as a result of electrochemical dissolution. The theoretical current capacity can be calculated according to Faraday's Law.

Efficiency (%): the product of the ratio between the effective current capacity and the theoretical current capacity by one hundred.

Electrode potential: the electromotive force (emf) of a cell between two electrodes.

Electropositive: the tendency of a metal to lose electrons and become positively charged

Electrical resistivity: a measure of how strongly a material resists the flow of electric current. It is also known as electrical resistance

Electrochemical cell: the circuit between an anode and a cathode in an electrically conducting media.

Electrode potential: the potential of an electrode within an electrolyte measured with respect to a reference electrode. Electrode potential measurements indicate whether the electrochemical reactions taking place at the electrode surface are predominantly anodic or cathodic and thus can give an indication of the condition of the electrode material. It is also referred to as half-cell potential.

Electrochemical series: the gradation of metals or alloys related to their degree of activity in a given environment. It is also known as galvanic series.

Electronegative: the tendency of a non-metal to attract electrons and become negatively charged.

Equilibrium potential: A point in which forward and reverse reaction rates are equal in an electrolytic solution, thereby establishing the potential of an electrode.

Galvanic corrosion: an electrochemical process in which one metal corrodes preferentially to another when both metals are in electrical contact and immersed in an electrolyte.

Instant off potential: the potential after disconnection of current (supply).

Instantaneous corrosion rate: the corrosion rate at a particular instant/given point in time.

Linear polarization resistance: a technique used to determine the instantaneous corrosion rate of test electrodes. The technique requires that the polarisation shift created by the passage of current of a small DC electric current between test electrodes is determined.

Open Circuit potential: the voltage at the terminals of a source when no appreciable current is flowing. It is also known as no-load voltage.

Polarisation: the change in the potential of an electrode. It results from changes which occur due to the passage of the current. The most influential of these changes are the build up of thin films of relatively higher resistance at electrochemical interfaces and a change in the concentration of reacting species at the surfaces of the electrode.

Potential decay: the extent of depolarisation from the polarised (IR free) condition following complete disruption of DC current.

Potential shift: the extent of polarisation from the natural (as found) potential to the operating potential.

Protection current density: the current density required to be applied by cathodic protection to the reinforcing steel, so that the corrosion of the reinforcing steel is controlled.

Reference electrode: a piece of metal in a fixed concentration solution of its own ions (such as copper in saturated copper sulphate, silver in silver chloride, etc). Also, it can be defines as an electrode which has a stable and well known electrode potential. It is also referred to as the standard electrode potential.

Throwing power: the maximum distance within which the protective effects of a cathodic protection system are experienced.

Volume resistivity*:* the ratio of the potential gradient parallel to the current in a material, to the current density. It is numerically equal-to the direct current resistance between opposite faces of a one centimetre cube of the material. It is expressed in ohm-centimetres. It is also known as specific insulation resistance.