

RESEARCH MONOGRAPH NO. 5

# **Repair principles for corrosion-damaged reinforced concrete structures**

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## **INTRODUCTION**

Corrosion is the inevitable process that occurs when refined metals return to their more stable combined forms as oxides, carbonates and sulphides. The corrosion process may be defined as the surface wastage that occurs when metals are exposed to reactive environments. Costs associated with corrosion damage and control can be substantial, being as much as 3.5% of the GNP of some industrial countries.

Reinforced concrete structures have not been immune to the ravages of corrosion despite the protection that concrete provides to embedded steel. Reasons for the increasing incidence of corrosion damage to reinforced concrete structures include the use of deicing salts and calcium chloride set-accelerators, increased construction in aggressive environments, fast-track construction practices, changing cement composition resulting in finer grinding and lower cement contents, lower cover depths and poor construction practice including inadequate supervision.

Reinforcement corrosion is particularly pernicious in that damage may occur rapidly and repairs are invariably expensive. Furthermore by the time visible corrosion damage is noticed, structural integrity may already be compromised. There is currently considerable debate about the merits of the various systems for the repair of reinforcement corrosion. This monograph attempts to clarify some of the important issues by drawing on international experience as well as local findings. Ultimately the effectiveness of repair systems should be measured in terms of cost, risk of failure and long-term performance. As such no single system is appropriate for all repairs but will depend on the type of structure, service conditions, level of deterioration and financial constraints of the project.

This monograph focuses on repair principles rather than dealing with issues of detail that have been competently published by others. Repair options can only be rationally compared when the corrosion process and its influence on concrete are fully understood. The document also focuses on South African conditions and experiences, derived from almost ten years of research on concrete durability and repairs at the University of Cape Town.

## **CORROSION FUNDAMENTALS**

Steel reinforcing bars will corrode freely when exposed to moisture and oxygen under ambient conditions. When steel is embedded in concrete

however the high alkalinity (pH of 12.5 or higher) stifles corrosion by the formation of a passive ferric oxide film on the steel surface. The ferric oxide layer forms a dense, impenetrable film that suppresses further corrosion by limiting the movement of cations and anions near the steel surface. This passive ferric oxide film on embedded reinforcement may be disrupted by a reduction in the alkalinity of the concrete (principally by carbonation) or by the presence of aggressive ions such as chlorides and sulphates. Depassivation of the steel occurs as follows:

- in carbonated concrete, insufficient hydroxyl ions are available to repair pits in the passive film
- in salt contaminated concrete, chloride ions break down the passive layer at localized pits and encourage metallic dissolution

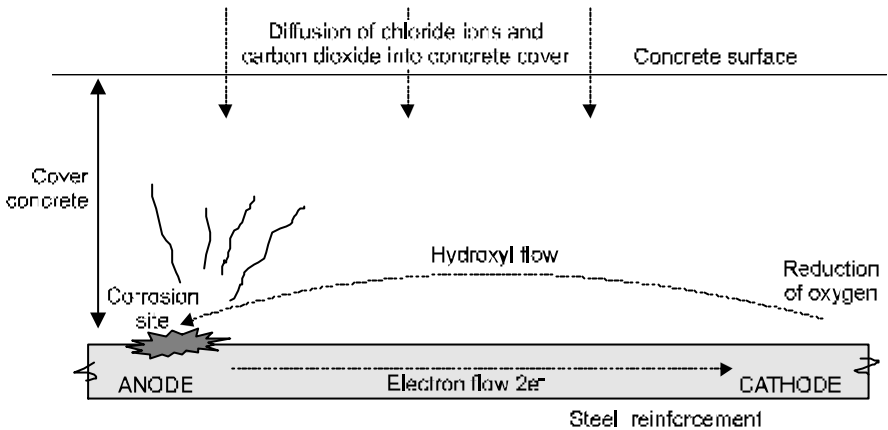
Once depassivating conditions exist in concrete either by a reduction in alkalinity (pH <10.5) or by the presence of sufficient chloride ions (termed the corrosion threshold value), corrosion may occur. For corrosion to occur at a significant rate the following conditions are required:

- a reactive metal that will oxidise anodically to form soluble ions
- a reducible metal that provides the cathodic reactant (typically hydroxyl ions)
- an electrolyte that allows ionic movement between the material and environment

It is important to note that the establishment of depassivating conditions at the steel (i.e. carbonation or chlorides) is not necessarily indicative of a high probability of corrosion damage since other factors (e.g. oxygen availability, moisture content) will largely determine the rate of corrosion. A schematic diagram of the corrosion process of steel in concrete is shown in Figure 1.

Four states of corrosion may be defined for reinforced concrete depending on environmental conditions<sup>1</sup>:

- Passive state where minute levels of corrosion are needed to sustain the ferric oxide film (typical of embedded reinforcement in sound, alkaline and uncontaminated concrete).
- Pitting corrosion causing local breakdown of the passive film, usually due to the presence of chloride ions. Adjacent steel acts as the cathode, being considerably larger in area than the anode (typical of steel embedded in chloride contaminated concrete).



**Figure 1: Schematic of corrosion of steel in concrete**

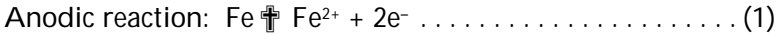
- General corrosion due to an overall loss of passivity that results in multiple pits along the steel surface (typical of steel in carbonated concrete or concrete containing high chloride concentrations).
- Active, low potential corrosion that occurs slowly when insufficient oxygen is available to sustain the passive film despite the high alkalinity of the concrete (typical of reinforcement embedded in concrete underwater).

Clearly only pitting and general corrosion represent a threat to the reinforcement and their severity will depend on a number of internal and external factors which need to be assessed when doing a corrosion survey. Internal factors include concrete microstructure, cover depth and moisture condition. External influences such as stray currents and microbial activity may introduce a new dimension into the corrosion system, but are not considered here.

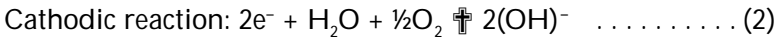
The nature of steel corrosion in concrete depends on local conditions at the surface of the bar. High resistivity concrete with relatively deep covers tends to favour micro-cell corrosion where anode and cathode are close together and cause localized pitting. Conductive concrete contaminated with salt is often able to sustain more widely spaced anode and cathode sites, termed macro-cell corrosion.

**CORROSION DAMAGE**

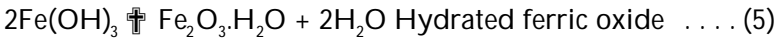
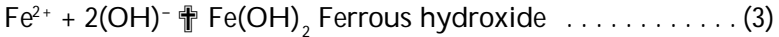
Once the passive layer on the reinforcing steel has been disrupted and corrosion is activated, the chemical reactions are similar whether the corrosion was initiated by chloride attack or by carbonation. Steel dissolves into solution and gives up electrons at the anode.



The excess electrons are used up at the cathodic site where water and oxygen are reduced to hydroxyl ions.



These two reactions are necessary for electrochemical corrosion to proceed. Little distress would be caused to the surrounding concrete however if steel merely dissolved into the pore water without further oxidation. Several more oxidation stages occur which form expansive corrosion products or rust capable of causing cracking and spalling of the surrounding concrete. The oxidation stages may be described as follows:



The expansion associated with rust is mostly due to hydrated oxides that may swell up to ten times the original volume of the steel. The type of corrosion product formed at the steel depends on environmental conditions:

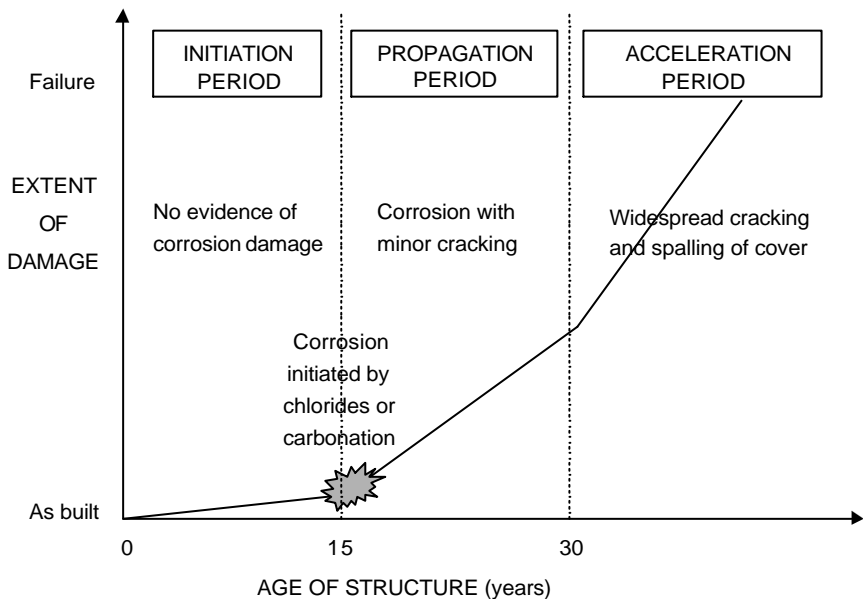
- red or brown rust forms under high oxygen concentrations, forming flakey rust which is relatively soft and easy to dislodge from the rebar
- black rust forms under low oxygen concentrations, forming a relatively dense and hard layer that may be difficult to remove from the parent steel

Two major consequences of reinforcement corrosion are commonly observed, cracking and spalling of the cover concrete as a result of expansion of the corrosion product, and a reduction of cross-sectional area of the rebar by pitting (usually only a problem in prestressed concrete structures). Manifestations of corrosion depend on a number of influences that include:



- geometry of the element (large diameter bars at low covers allow easy spalling)
- cover depths (deep cover may prevent full oxidation of corrosion product)
- moisture condition (conductive electrolytes encourage well-defined macro-cells)
- age of structure (rust stains progress to cracking and spalling)
- rebar spacing (closely spaced bars in walls and slabs encourage delaminations)
- crack distribution (cracks may provide low resistance paths to the reinforcement)
- service stresses (corrosion may be accelerated in highly stressed zones)

The loss of serviceability of corroded reinforced concrete structures may be described by a three phase damage model shown in Figure 2 <sup>2</sup>.



**Figure 2: Three-phase corrosion damage model**

The different phases are defined as follows:

- An initiation period, before corrosion is activated by either carbona-

tion or chloride attack, during which negligible concrete deterioration occurs.

- A propagation period in which active corrosion commences and cracking of the cover concrete occurs due to the formation of expansive corrosion products at the steel surface.
- An acceleration period of damage where corrosion increases due to easy access of oxygen and water through cracks in the cover concrete, resulting in spalling of concrete.

Unfortunately most reinforced concrete structures that exhibit cracking and spalling have gone beyond the point where simple, cost-effective measures can be taken to restore durability. Condition surveys are therefore an important strategy to identify and quantify the state of corrosion of a structure timeously.

## **CONDITION SURVEYS**

A detailed corrosion or condition survey is vital in order to identify the exact cause and extent of deterioration, before repair options are considered. Various diagnostic sheets are given in the Appendix for guidance during condition surveys.

### **a) Visual assessment**

Corrosion damage may be identified and defined using a systematic visual survey. Classification of visual evidence of deterioration must be done objectively, following clear guidelines that define damage in terms of appearance, location and cause. Defects may be defined in terms of cracks (caused by corrosion, temperature, shrinkage or fatigue), joint deficiencies (joint spalls, upward movement, lateral movement, seal damage) surface damage (abrasion, rust stains, delaminations, popouts, spalls), changes in member shape (curling, deflection, settlement, deformation) and textural features (blow holes, honeycombing, sand pockets, segregation).

Visual assessment of deterioration can provide useful information when done in a rational, systematic manner but the data may come too late for cost-effective repairs. Rebar corrosion damage is often only fully manifest at the surface after significant deterioration has occurred. Early evidence of distress can sometimes be detected by an experienced engineer before major distress takes place.

### **b) Delamination survey**

A hammer survey or chain drag is a simple method of locating areas of delamination in concrete. Hollow sounding areas can be marked up on the concrete or recorded directly in a survey form. Delamination surveys often under-estimate the full extent of internal cracking and should not be considered as definitive. Radar and ultrasonic instruments may provide a more sophisticated approach to locating areas of delamination, particularly at greater depths.

### **c) Cover surveys**

Cover surveys are routinely done to locate the position and depth of reinforcement within a concrete structure. Covermeters use an alternating magnetic field to locate steel and any other magnetic material in concrete (note that austenitic stainless steels are non-magnetic). Cover measurements may be unreliable when:

- rebar is at deep covers (e.g. covers greater than 80 mm)
- measuring regions of closely spaced bars
- measuring differing bar types and sizes (unless specifically calibrated)
- other magnetic material is nearby (e.g. window frames, wire ties, bolts)

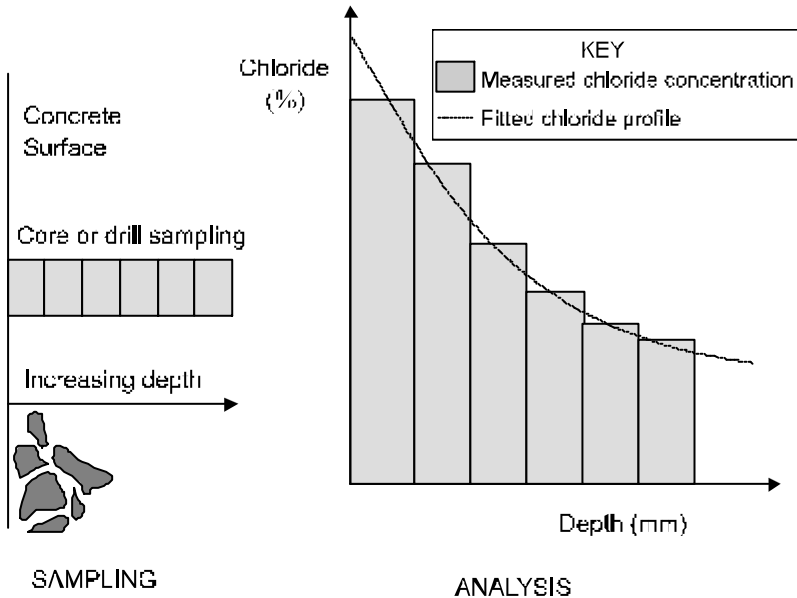
To ensure reliable cover depths from a survey, direct measurements of rebar depths should be made by exposing a limited number of bars. Calibration can then be made for site specific conditions such as rebar type, concrete and environmental influences.

### **d) Chloride testing**

The presence of sufficient chloride at the surface of reinforcement is able to depassivate steel and allow corrosion to occur. Chlorides exist in concrete as both bound and free ions but only free chlorides directly affect corrosion. Measuring free chlorides accurately is extremely difficult and water-soluble chloride tests are unreliable, being strongly affected by the method of sample preparation. Further, bound chlorides may be released into solution under carbonating conditions or by dissolution, making all chlorides in concrete potentially corrosive. Chlorides are therefore most commonly determined as acid soluble or total chlorides in accordance with BS 1881<sup>3</sup>.

Chloride sampling and determination in concrete is illustrated in Figure 3 and is usually done in the following manner:

- concrete samples are extracted as either core or drilled powder samples
- depth increments are chosen depending on the cover to steel and the likely level of chloride contamination (increments are typically between 5 and 25 mm)
- dry powder samples are digested in concentrated nitric acid to release all chlorides
- chlorides are analysed using a colorimetric or potentiometric titration
- chloride contents are generally expressed as a percentage by mass of cement
- chloride profiles may be drawn such that chloride concentrations may be interpolated or extrapolated for any depth (see Figure 3)
- future chloride levels can be estimated from Fick's second law of diffusion



**Figure 3: Chloride content determination and typical chloride profile**

The corrosion threshold depends on several factors including concrete quality, cover depth, and saturation level of the concrete. The probability of corrosion may be assessed from the following qualitative rating shown in Table 1 for acid-soluble chloride contents.

**Table 1: Qualitative risk of corrosion based on chloride levels**

<i>Chloride content by mass of cement (%)</i>	<i>Probability of corrosion</i>
< 0.4	Low
0.4 – 1.0	Moderate
> 1.0	High

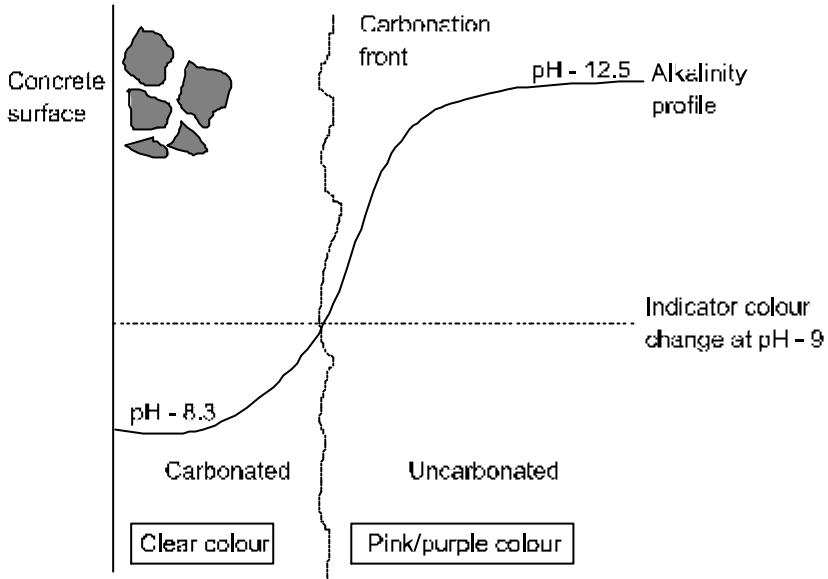
Limitations of chloride testing of concrete are as follows:

- presence of chlorides in aggregates may give misleading results
- chloride contents in cracks and defects cannot be accurately determined
- slag concretes may be difficult to analyse with colorimetric titration methods
- relatively large samples are required to allow for the presence of aggregates

### **e) Carbonation depth**

Carbonation depth is measured by spraying fresh concrete with a phenolphthalein indicator solution (1% by mass in ethanol/water solution). Phenolphthalein remains clear where concrete is carbonated but turns pink/purple where concrete is still strongly alkaline (pH > 9.0). Carbonation moves through concrete as a distinct front and reduces the natural alkalinity of concrete from a pH in excess of 12.5 to approximately 8.3, with a pH level of 10.5 being sufficiently low to depassivate steel. The progress of the carbonation front is shown in Figure 4.

Environmental conditions most favourable for carbonation (i.e. 50 – 65 % R.H.) are usually too dry to allow rapid steel corrosion that normally requires humidity levels above 80% R.H. Structures exposed to fluctuations in moisture conditions of the cover concrete, such as may occur during rainy spells, are however vulnerable to carbonation-induced corrosion.



**Figure 4: Schematic of the carbonation process**

Limitations to carbonation testing are as follows:

- phenolphthalein changes colour at pH 9.0 whereas steel depassivation occurs at a pH of approximately 10.5, hence the corrosion risk is slightly under-estimated
- some concretes are dark (e.g. slag concretes) and a distinct colour change is difficult to discern visually
- phenolphthalein may bleach at very high pH levels (e.g. after electrochemical realkalization)
- testing must be done on freshly exposed concrete surfaces before atmospheric carbonation occurs

#### **f) Rebar potentials**

Chloride-induced corrosion of steel is associated with anodic and cathodic areas along the rebar with consequent changes in electropotential of the steel. It is possible to measure these rebar potentials at different points and plot the results in the form of a 'potential map'. Measurement of rebar potentials may determine the thermodynamic risk of corrosion but cannot evaluate the kinetics of the reaction. Rebar potentials are normally determined in accordance with ASTM C876 using a

copper/copper sulphate reference electrode connected to a handheld voltmeter<sup>4</sup>. The qualitative risk of corrosion based on rebar potentials is shown in Table 2. Note that the technique is not recommended for carbonation-induced corrosion where clearly defined anodic regions are absent.

**Table 2: Qualitative risk of chloride-induced corrosion<sup>4</sup>**

<i>Rebar potential (-mV Cu/CuSO<sub>4</sub>)</i>	<i>Qualitative risk of corrosion</i>
< 200	Low
200-350	Uncertain
>350	High

The procedure for undertaking a rebar potential survey is as follows:

- mark up a grid pattern in the area of measurement (not more than 500 mm centres)
- make an electrical connection to clean steel by coring or breaking out concrete
- check the steel is electrically continuous over the survey area using a multimeter
- wet the concrete surface with tap water if the concrete appears to be dry
- take and record readings either manually or using a data logger
- check data on site to correlate with visual signs of corrosion

Rebar potential measurements are relatively quick to perform but have the following limitations:

- interpretation of results must be done with caution (preferably by a specialist)
- rebar potentials from carbonated concrete are difficult to interpret (the reading is a mixed potential of anodic and cathodic sites)
- delaminations may disrupt the potential field producing false readings
- environmental effects will influence potentials (e.g. temperature and humidity)
- rebar potentials cannot be directly correlated with corrosion rates
- stray currents may affect measured potentials

Absolute values are often of lesser importance than differences in rebar potential measured on a structure. A shift of several hundred millivolts over a short distance of 300-500 mm often indicates a high risk of corrosion.

### g) Resistivity

Concrete resistivity controls the rate at which steel corrodes in concrete once favourable conditions for corrosion exist. Resistivity is dependent on the moisture condition of the concrete, on the permeability and inter-connectivity of the pore structure, and on the concentration of ionic species in the pore water of concrete such that:

- poor quality, saturated concrete has low resistivity (e.g. less than 10 kOhm.cm)
- high quality, dry concrete has high resistivity (e.g. greater than 25 kOhm.cm)

Measurement of resistivity is done with a simple in situ Wenner probe connected to a portable resistivity meter. The outer two probes send an alternating current through the concrete while the inner two probes measure the potential difference in the concrete. Once the concrete resistivity is known a rough assessment of likely corrosion rates can be made as shown in Table 3. This assessment assumes conditions are favourable for corrosion.

**Table 3: Likely corrosion rate based on concrete resistivity**

<i>Resistivity (kOhmcm)</i>	<i>Likely corrosion rate given corrosive conditions</i>
<12	High
12-20	Moderate
>20	Low

Resistivity measurements are simple to perform on site but have several limitations:

- measurements are affected by carbonation and wetting fronts
- surface conductive layers and rebar directly below the probe should be avoided
- readings may be unstable in concretes with high contact resistance at the surface



## h) Corrosion rate measurements

Corrosion rate measurements are the only reliable method of measuring actual corrosion activity in reinforced concrete. A number of sophisticated corrosion monitoring systems are available, based primarily on linear polarization resistance (LPR) principles. These techniques require considerable expertise to operate reliably. Corrosion rate measurements on field structures are most commonly done using galvanostatic LPR techniques with a guard-ring type sensor to confine the area of steel under test. Experience indicates that corrosion rates fluctuate significantly in response to environmental and material influences and single readings are generally unreliable. Table 4 shows a qualitative guide for the assessment of corrosion rates of site structures <sup>5</sup>.

**Table 4: Qualitative assessment of site corrosion rates**

<i>Corrosion rate (<math>\mu\text{A}/\text{cm}^2</math>)</i>	<i>Qualitative assessment of corrosion rate</i>
> 10	High
1.0 – 10	Moderate
0.2 – 1.0	Low
< 0.2	Passive

## REPAIR STRATEGIES

Numerous repair options are available and new technologies continue to make an impact in the field of concrete repairs. The suitability and cost-effectiveness of repairs depends on the level of deterioration and specific conditions of the structure.

### a) Patch repairs

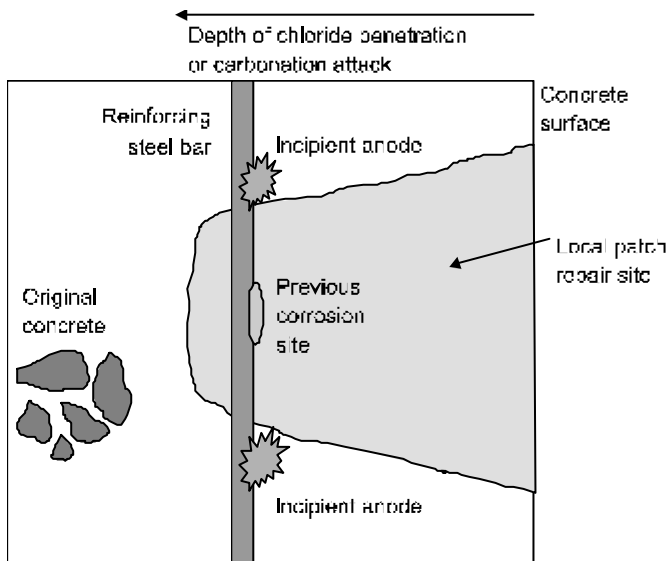
Before patch repairs are considered it is important that the distinction between chloride- and carbonation-induced corrosion is appreciated. As a general rule chloride-induced corrosion is far more pernicious and difficult to treat than carbonation-induced corrosion. This often dictates a completely different approach to repairing damage due to the two types of corrosion.

Carbonation-induced corrosion causes general corrosion with multiple pitting along the reinforcement. Carbonated concrete tends to have fairly high resistivity that discourages macro-cell formation and allows moderate corrosion rates. Steel exposed to corrosive conditions will therefore show signs of corrosion that can be easily identified (e.g. sur-

face stains, cracking or spalling of concrete). Repairs are generally successful provided all of the corroded reinforcement is treated.

Chloride-induced corrosion is characterized by pitting corrosion with distinct anode and cathode sites. The presence of high salt concentrations in the cover concrete means that macro-cell corrosion is possible with relatively large cathodic areas driving localized intense anodes. High corrosion rates can be sustained under such conditions resulting in severe pitting of the reinforcement and damage of the surrounding concrete. Much of the reinforcement may be exposed to corrosive conditions without showing any signs of corrosion, this is particularly noticeable when corroded structures are demolished.

Localized patch repairs of areas of corrosion damage are popular due to their low cost and temporary aesthetic relief. This form of repair has limited success against chloride-induced corrosion as the surrounding concrete may be chloride-contaminated and the reinforcement is therefore still susceptible to corrosion. The patched area of new repair material often causes the formation of incipient anodes adjacent to the repairs as shown in Figure 5. These new corrosion sites not only affect the structure but often also undermine the repair leading to accelerated patch failures in as little as two years. Consequently, it is necessary to remove all chloride-contaminated concrete from the vicinity of the reinforcement.



**Figure 5: Formation of incipient anodes after patch repairs**

Complete removal of chloride-contaminated concrete, where it is possible should successfully halt corrosion by restoring passivating conditions to the reinforcement. Mechanical removal of cover concrete is usually done with pneumatic hammer, hydrojetting or milling machines. This form of repair is most successful when treating areas of localized low cover, before significant chloride penetration has occurred. If repairs are only considered once corrosion damage is fairly widespread it will be expensive to mechanically remove chloride-contaminated concrete from depths well beyond the reinforcement.

Patch repairs consist of the following activities that are briefly described below:-

- removal of cracked and delaminated concrete to fully expose the corroded reinforcement
- cleaning of corroded reinforcement and the application of a protective coating to the steel surface (e.g. anti-corrosion epoxy coating or zinc-rich primer coat)
- application of repair mortar or micro-concrete to replace the damaged concrete
- possible coating or sealant applied to the entire concrete surface to reduce moisture levels in the concrete

## **b) Coating systems**

A variety of coating and penetrant systems are available that are claimed to be beneficial in repairs of concrete structures. Barrier systems attempt to seal the surface thereby stifling corrosion by restricting oxygen flow to the cathode. In large concrete structures, corrosion control is theoretically unlikely due to the presence of oxygen already in the system. In practice barrier systems are generally ineffective due to the presence of defects in the new coating during application and further damage during service. Such an approach is more likely to promote the formation of differential aeration cells further exacerbating the potential for corrosion.

The application of a hydrophobic coating (sometimes referred to as penetrant pore-liners) may be used to reduce the moisture content of concrete and thereby electrolytically stifle the corrosion reaction. The drying action works on the principle that surface capillaries become lined with a hydrophobic coating that repels water molecules during wetting but allows water vapour movement out of the concrete, to facil-

itate drying. Hydrophobic coatings using silanes and siloxanes are generally most effective on uncontaminated concrete, free from cracks and surface defects. The feasibility of such an approach is questionable for marine structures where high ambient humidity, capillary suction effects and presence of high salt concentrations all interfere with drying.

The long-term effectiveness of hydrophobic systems applied to new construction is not known but local studies suggest reasonable performance over 10-15 years service. The Storms River bridge was coated with a silane system in 1985 and concrete cores were extracted from several parts of the structure in 1996 for analysis<sup>6</sup>. The effect of the hydrophobic coating on absorption was determined by sorptivity testing at increasing depth increments into the concrete. Sorptivity results are shown in Figure 6 for arch and column concrete. The sharp increase in sorptivity at depths between 0.5 and 3 mm may be ascribed to the presence of the silane in the concrete near-surface zone.

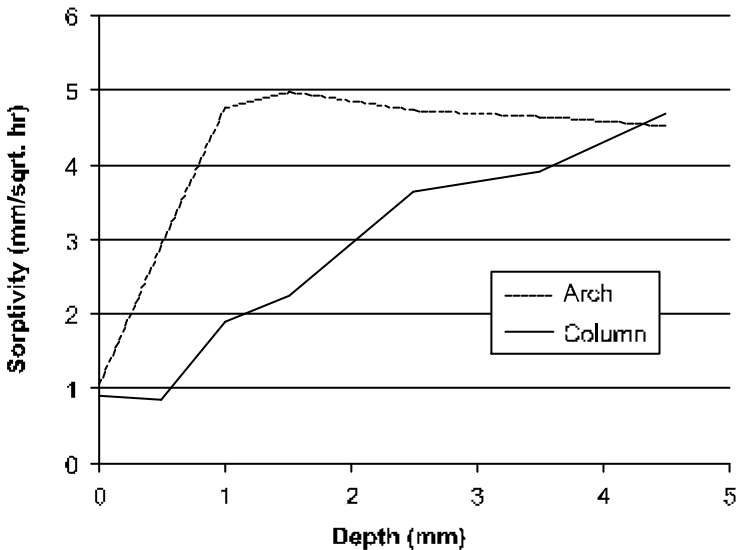


Figure 6: Sorptivity results from bridge cores

### c) Migrating corrosion inhibitors

A corrosion inhibitor is defined as a chemical substance that reduces the corrosion of metals without a reduction in the concentration of corrosive agents. Corrosion inhibitors work by reducing the rate of the anodic

and/or cathodic reactions thereby suppressing the overall corrosion rate. The effectiveness of migrating corrosion inhibitors is generally controlled by environmental, material and structural factors, shown in Table 5<sup>7</sup>.

**Table 5: Likely performance of migrating corrosion inhibitors in concrete**

<i>Likely inhibition</i>	<i>Corrosive conditions</i>	<i>Concrete conditions</i>	<i>Severity of corrosion</i>
Good	Mildly corrosive, low chlorides or carbonation	Dense concrete with good cover depths (> 50 mm)	Limited corrosion with minor pitting of steel
Moderate	Moderate levels of chloride at rebar (i.e. <1%)	Moderate quality concrete, some cracking	Moderate corrosion with some pitting
Poor	High chloride levels at rebar (i.e. > 1%)	Cracked, damaged concrete, low cover to rebar	Entrenched corrosion with deep pitting

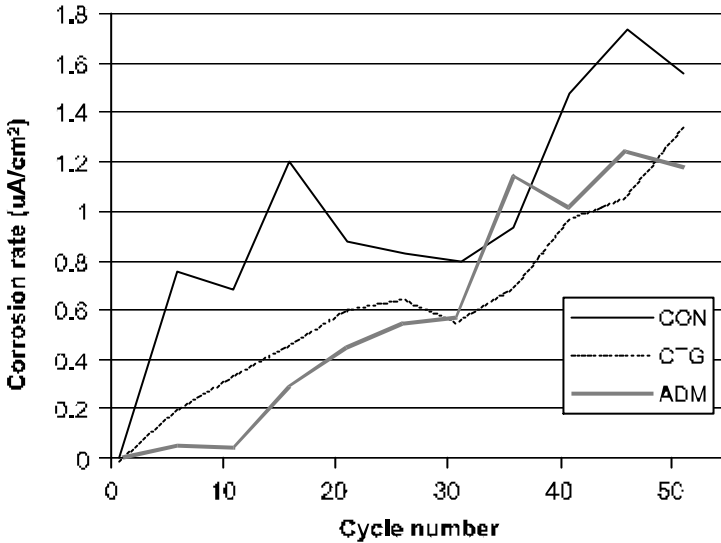
Migrating corrosion inhibitors are generally organic-based materials that move through unsaturated concrete by vapour diffusion. Organic corrosion inhibitors such as amino-alcohols are believed to suppress corrosion by primarily being adsorbed onto the steel surface thereby displacing corrosive ions such as chlorides. The adsorbed organic layer inhibits corrosion by interfering with anodic dissolution of iron while simultaneously disrupting the reduction of oxygen at the cathode.

When assessing the suitability of repairs with migrating corrosion inhibitors, two important issues must first be considered:

- the likely penetration of the material into the concrete needs to be determined
- the severity of the corrosive environment at the reinforcement must be quantified

Migrating corrosion inhibitors are designed to move fairly rapidly through partially saturated concretes that allow vapour diffusion. Penetration has however been found to be poor in near-saturated concretes typically found in partially submerged marine structures. This poor penetration performance may be ascribed to high moisture and salt levels that prevent significant vapour diffusion through the concrete. It is critical therefore that satisfactory penetration of corrosion inhibitors is checked before undertaking full-scale repairs.

The performance of migrating corrosion inhibitors in controlling chloride-induced corrosion is largely dependent on chloride levels at the reinforcement. Work done by Rylands indicates that effective inhibition is not possible at chloride levels above 1.0% at the reinforcement<sup>8</sup>. This can be seen in Figure 7 where ribbed steel bars embedded at 25 mm in a grade 40 portland cement concrete were subjected to wetting and drying cycles with a salt solution for a period of 18 months.



**Figure 7: Corrosion rate measurements with time for grade 40 concrete**

Concrete blocks were either controls (CON) or contained organic corrosion inhibitor, either admixed during casting (ADM) or coated after 30 cycles (CTG). The chloride content at the level of the reinforcement was approaching 2% at the time of application of the migrating corrosion inhibitor and resulted in poor inhibition. Better inhibition is possible if treatment is done earlier when chloride contents are lower.

The effectiveness of migrating corrosion inhibitors appears to be enhanced when used in combination with hydrophobic coatings to reduce moisture levels in concrete. This has been noted in both laboratory trials and field monitoring of repairs. Such an approach has also been found to be effective in the repair of carbonation-induced corrosion damage.

#### d) Electrochemical techniques

Corrosion of reinforcement in concrete is an electrochemical process that occurs when embedded steel is depassivated by a reduction in concrete alkalinity or the presence of corrosive ions such as chlorides. Two repair techniques, electrochemical chloride removal and realkalization, attempt to restore passivating conditions by the temporary application of a strong electric field to the cover concrete region.

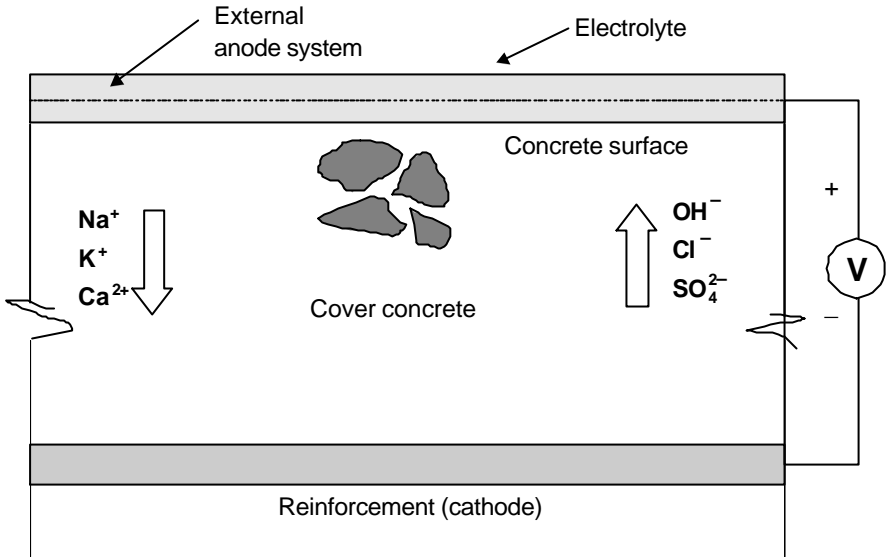
Realkalization is the process of restoring the original alkalinity of carbonated concrete in a non-destructive manner. The electrochemical treatment consists of placing an anode system and sodium carbonate electrolyte on the concrete surface and applying a high current density (typically  $1 \text{ A/m}^2$ ). The electrical field generates hydroxyl ions at the reinforcement and draws alkalis into the concrete. Alkaline conditions may be restored in the concrete in as little as one to two weeks using the system.

Electrochemical chloride removal (ECR) is a more time-consuming and complex technique and its suitability needs to be carefully assessed. Chloride removal is induced by applying a direct current between the reinforcement and an electrode that is placed temporarily onto the outside of the concrete. The impressed current creates an electric field in the concrete that causes negatively charged ions to migrate from the reinforcement to the external anode. The technique decreases the potential of the reinforcement, increases the hydroxyl ion concentration and decreases the chloride concentration around the steel thereby restoring passivating conditions. Figure 8 shows the basic principles of ECR.

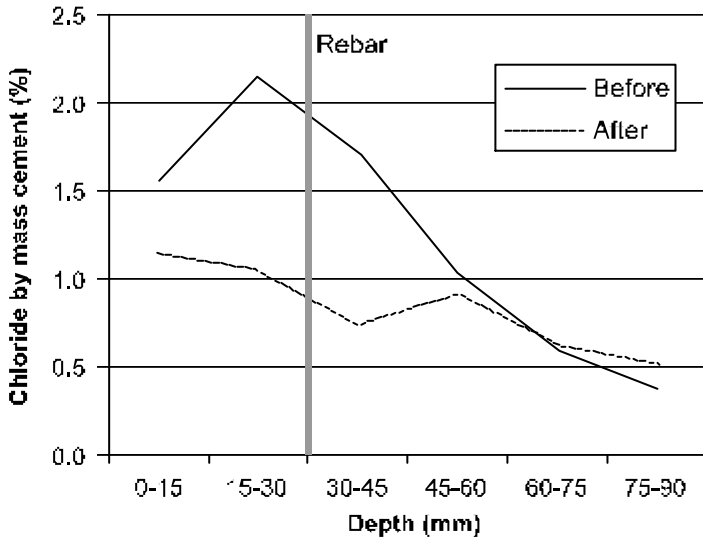
The effectiveness of ECR depends on several factors that include the following:-

- extent of chloride contamination in concrete
- structural configuration including depth and spacing of reinforcement
- applied current density and time of application
- pore solution conductivity and resistance of cover concrete
- presence of cracks, delaminations and defects causing uneven chloride removal

ECR typically takes 4-12 weeks to run at current densities within the normal range of  $1\text{-}2 \text{ A/m}^2$ . Results from ECR trials performed in the laboratory are shown in Figure 9 and indicate that complete extraction may take longer than 8 weeks at a current density of  $1 \text{ A/m}^2$  <sup>9</sup>. In some cir-



**Figure 8: Schematic illustration of electrochemical chloride removal technique**



**Figure 9: Chloride profiles before and after ECR treatment for 8 weeks**



cumstances chlorides beyond the reinforcement may be forced deeper into the concrete during the process. There is a risk that chlorides left in the concrete may diffuse back to the reinforcement and cause further corrosion with time.

The feasibility of using ECR depends on several factors such as:-

- the presence of major cracking, delaminations and defects that will require repair before ECR
- large variations in reinforcement cover that will cause differential chloride extraction and possible short-circuiting
- reactive aggregates requires special precautions to avoid possible alkali silica reaction; lithium salts should be used in these cases
- prestressed concrete structures may be susceptible to hydrogen embrittlement after ECR; special precautions are needed to eliminate this risk
- temporary power supplies of significant capacity are required during application of ECR

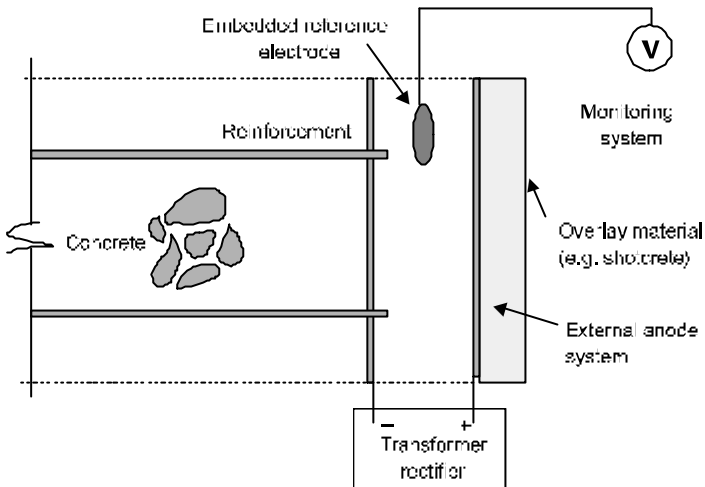
### **e) Cathodic protection systems**

Cathodic protection systems (CP) have an excellent track record in corrosion control of steel and reinforced concrete structures. The principle of CP is that the electrical potential of the steel reinforcement is artificially decreased by providing an additional anode system at the concrete surface. An external current is required between anode and cathode that diminishes the corrosion rate along embedded reinforcement. The current may be produced either by a sacrificial anode system or using an impressed current from an external power source.

Sacrificial anode systems consist of metals higher than steel in the electrochemical series (e.g. zinc). The external anode corrodes preferentially to the steel and supplies electrons to the cathodic steel surface. Sacrificial anode systems are most effective in submerged structures where the concrete is wet and resistivity is low. Warm temperatures are also generally required for sacrificial CP systems (i.e. above 20°C).

CP systems more commonly use an external electrical power source to supply electrons from anode to cathode. The anode is placed near the surface and is connected to the reinforcement through a transformer rectifier that supplies the impressed current (see Figure 10). Anodes may be conductive overlays, titanium mesh within a sprayed concrete overlay, discrete anodes or conductive paint systems. Anode systems are usually

designed for a minimum service life of 20 years but may last in excess of 50 years.



**Figure 10: Typical cathodic protection layout**

Before CP repairs are undertaken several factors need to be considered:

- reinforcement must be electrically continuous
- concrete cover must be uniformly conductive and free of delaminations
- alkali reactive aggregates and prestressing steel need special treatment
- power must be available to drive the impressed current in the structure

CP repair of concrete structures requires a thorough corrosion survey by a specialist and the design needs to be undertaken by a corrosion expert. Reliable CP systems are fully controlled and monitored by a series of embedded sensors in order to ensure optimum performance. This is essential since under or over-protection of the reinforcement may be potentially harmful to the structure or the CP system. Continuous monitoring of CP systems is usually done remotely by modem and the power consumption during operation is extremely small.

The first major CP repair of a reinforced concrete structure in South

Africa was done at the Simonstown Jetty in 1996 <sup>10</sup>. The structure was almost 80 years old and in an extremely poor condition with widespread chloride-corrosion damage. Several previous patch repairs had failed and the concrete was contaminated with chlorides making conventional repairs unfeasible. An impressed current CP system was installed with metallic ribbon anodes protected within a sprayed concrete overlay. The structure has been restored to full serviceability and should require no further repairs for at least 40-50 years.

#### **f) Demolition/reconstruction**

Deterioration of reinforced concrete structures is often so advanced that demolition and reconstruction becomes viable. This option should only be considered as a last resort since the total cost (capital costs plus loss of service and temporary works) is usually well in excess of repairs costs. Corrosion damage is also generally confined to near-surface regions and engineers often over-estimate the extent of damage to corrosion-damaged structures. Recent demolition of several bridge-decks along the Cape coast revealed that actual corrosion damage was less than anticipated.

Demolition and reconstruction is often preferred by engineers who have limited repair experience or lack confidence in new repair systems. It is crucial nevertheless that lessons are learnt from the old structure when designing the replacement. Guidance about ensuring durable reinforced concrete structures is given in Monographs 1 and 2.

### **ECONOMICS OF REPAIRS**

Repairs of reinforced concrete structures damaged by corrosion have often proved to be unsuccessful with further damage occurring after repair. Reasons for the poor performance of repairs include:-

- lack of understanding of deterioration processes
- inadequate investigation and testing prior to repairs
- inadequate funds to undertake satisfactory repairs
- ineffective or inappropriate repairs being specified
- poor supervision and implementation of repairs on site

Repairs are not generally anticipated by owners and funds for repairs are nearly always extremely limited. Economics largely dictate the timing and scale of repairs but unfortunately only short-term costs are often

considered. Whilst corrosion damage is to some degree unique to each structure some basic tenets hold for most cases.

- Performance of the concrete structure prior to treatment often dictates the likely performance after repair. Structures with high levels of damage and rapid rates of deterioration require more substantial repair than those less seriously affected.
- The timing of treatment is crucial since corrosion rates and damage increase with time. A structure that has been neglected and allowed to reach an advanced level of damage will not respond to 'quick-fix' solutions. Conversely a structure that is repaired early enough may be restored to full serviceability relatively cheaply.
- The effectiveness of treatments in retarding corrosion is not equal and may range from highly effective to detrimental (e.g. cathodic protection versus patch repairs)

Importantly, repairs costs need to be compared in a rational way by comparing life-cycle costs of the structure. Scott showed that when life-cycle costs are compared, a maintenance-free structural design is cheaper than cutting initial costs and deferring some money for repair and maintenance at a later date (data shown in Table 6) <sup>11</sup>.

**Table 6: Total life cycle costs of typical beam members exposed to marine environment**

<i>Option</i>	<i>1</i>	<i>2</i>	<i>3</i>	<i>4</i>	<i>5</i>
Original design	60 MPa 30% fly ash 55 mm cover	60 MPa 30% fly ash 30 mm cover	60 MPa 30% fly ash 40 mm cover	60 MPa 30% fly ash 40 mm cover	60 MPa 100%PC 75 mm cover
Repairs/ maintenance	None	Surface treatment at 10-year intervals	Patch repairs after 20 and 35 years	Cathodic protection after 20 years	Patch repairs after 15, 25 and 35 years
Relative costs	1.0	2.0	2.3	3.0	3.5

*Notes on repair options:-*

*Option 1. Durability design for maintenance free 40 year service life*

*Option 2. Based on anticipated life of surface treatment*

*Options 3-5. Based on the likely stage at which spalling damage becomes excessive*

*Option 5. Design required by SABS 0100:1992*

Strohmeier showed that repair costs escalate dramatically as deterioration proceeds and that repairs should be done as soon as distress is noted<sup>12</sup>. This research helped quantify what many engineers had long realized; that durability-based designs are cost-effective in the long-term and that delays in repairs cause an exponential increase in costs.

Engineers considering repair of concrete structures do not have the freedom to change either the original design or the timing of the repairs. Repairs therefore need to be considered on the merits, logistics, costs and risks of the many options that are available to rehabilitate the structure. To illustrate some of the issues that need to be considered, a practical example is given in the Appendix.

## **CLOSURE**

The notion that reinforced concrete structures require no maintenance or repair during their service life is gradually being dispelled. It has been said that owners will have to pay for durability at some point in the life of a structure. Inadequate designs with excessive cost-cutting will merely transfer the savings in capital costs to much more expensive repairs at a later stage. While accountants may encourage some deferment of capital costs into maintenance, experience suggests that investments in the form of design and construction for durability bring better rewards than allowing for maintenance. Despite this evidence, economic imperatives that attempt to maximise short-term profits, often impact detrimentally on the durability and service life of infrastructural developments.

Repair of reinforced concrete structures needs to be undertaken in a rational manner to guarantee success. An increasing number of repair options are available that must be considered in terms of cost, technical feasibility and reliability. Engineers need to understand all the relevant material, structural and environmental issues associated with concrete repairs in order to make intelligent choices.

High quality repairs require a thorough investigation into the causes of deterioration, appropriate repair specifications and competent execution of the repair work. This can only be done when structural investigations are carried out by independent experts, specifications are drawn up by engineers with specialist repair expertise and repairs are undertaken by competent contractors.

## **APPENDIX I: Repair example**

A 60-year old bridge structure is in need of major repairs arising from widespread corrosion damage. The bridge spans a tidal estuary with direct exposure to seawater splash and spray action. Concrete is heavily contaminated with salt and chloride levels at the reinforcement are around 1.0% by mass of cement. Damage in the form of cracking, spalling and delaminations are widespread over much of the structure and are the result of chloride-induced corrosion. Urgent repairs are essential to restore full serviceability to the bridge.

Rough estimates of service life of the various options are based on recent experience in South Africa and specialist publications<sup>13,14,15</sup>. Whilst the projected performance of the various repairs is a subjective assessment, the figures serve to illustrate the many issues that need to be considered when costing repairs.

For the purposes of costing the repair options, the following assumptions are made:-

- unescalated 2001 costs are used due to uncertainties about future discount, inflation and tax rates
- site establishment costs are fixed at R 250 000 for each repair option
- total area of concrete under repair is 2000 m<sup>2</sup>
- unit rates for repair include allowance for labour, materials, access and supervision
- repairs are focused on chloride-induced corrosion damage only

The following repair options are considered for the bridge.

- A Localized repairs of corrosion-damaged areas with only cosmetic consequences. Assuming 15% of the structure requires patching and that concrete is only broken back to the reinforcement, a unit rate of R250/m<sup>2</sup> is used. Given the limited nature of the repairs and the likelihood of incipient anode formation an effective life of 8 years is considered possible.
- B More extensive mechanical break-outs and patching are done with all corroded reinforcement being exposed, cleaned and a good quality repair material used for patching. Approximately 30% of the structure is treated at a unit rate of R280/m<sup>2</sup>. Despite the effort made to repair

the structure, corrosive conditions still exist at the reinforcement and further corrosion damage limits the effective life to 12 years before more repairs must be considered.

- C Conventional corrosion repairs are done but a migrating corrosion inhibitor is applied to the repaired concrete surface together with a hydrophobic coating (silane/siloxane). Mechanical breakout is limited to damaged areas of concrete and not all corrosion on reinforcement is removed resulting in a unit rate of R300/m<sup>2</sup>. This includes the cost of the migrating corrosion inhibitor and coating at R40/m<sup>2</sup>. The chloride level at the reinforcement (1.0%) is at the upper level for corrosion inhibitor performance resulting in an effective service life of only 15 years.
- D Electrochemical chloride extraction is applied to the concrete to remove chloride from around the steel. The cost of the system is approximately R750/m<sup>2</sup> for a six week application and includes repair to damaged concrete. Unfortunately not all the chloride is removed from the concrete resulting in an effective service life of 25 years.
- E Cathodic protection is applied to the structure to protect the embedded reinforcement. The cost of the system is R900/m<sup>2</sup> at installation and a nominal maintenance and monitoring fee of R5000 per year. The anode system is designed to last 50 years thereby dictating the effective life of the system.

Present value costs for the various options are shown in Table A1. From these findings it is clear that initial repair costs and total repair costs over 40 years vary significantly. Option A is most cost-effective when only short-term costs are considered but most expensive in the longer-term. For a structure that only has to last another 20 years, option C may be preferable whereas for 40 years further service, option E is most economical for the hypothetical example.

**Table A1: Total present value costs (million rands)**

<i>Timing</i>	<i>Option A</i>	<i>Option B</i>	<i>Option C</i>	<i>Option D</i>	<i>Option E</i>
Initial	0.75	0.81	0.85	1.75	2.05
20 years	2.25	2.43	1.70	1.75	2.15
40 years	3.75	3.24	2.49	3.50	2.25

## APPENDIX 2: Diagnostic sheets

**Table A2: Checklist for investigation of structural deterioration**

<i>Item</i>	<i>Example</i>	<i>Observation</i>
	<b>Background data</b>	
Structure name	Identification, reference number	
Location	Physical address or location	
Environment	Severity and type of exposure	
History	Age, design data, repairs	
Date inspected	Date	
	<b>Original condition</b>	
Surface condition	Honeycombing, bleeding, voids, popouts	
Early cracking	Plastic settlement or plastic shrinkage	
Concrete quality	Surface hardness, density, voids, colour	
Rebar cover	Covermeter survey, mechanical breakout	
Structural effects	Overloading, dynamic effects, structural cracking	
	<b>Present condition</b>	
Surface damage	Abrasion, chemical attack, spalling, leaching	
Staining	Rebar corrosion, AAR gel, efflorescence, salts	
Cracking	Width, pattern, location, causes of cracking	
Rebar condition	Visual examination of bar, rust and pitting damage	
Carbonation	Indicator test on cores or mechanical breakouts	
Delamination	Size, frequency, severity of delamination	
Previous repairs	Integrity of repairs, signs of damage near repair locations	



**Table A3: Conditions and features of different forms of reinforcement corrosion**

<i>Type of corrosion</i>	<i>Environment or causative conditions</i>	<i>Significant features of deterioration</i>
Chloride-induced	Marine environments Industrial chemicals Admixed chlorides (older structures)	Rapid and severe corrosion Distinct anode & cathode regions Corrosion damage may affect structural integrity
Carbonation-induced	Unsaturated concrete Polluted environments Low cover depths to steel	General corrosion along rebar Moderate corrosion rates except when wet & dry faces are close Corrosion damage generally only affects aesthetics
Stray current	DC power supplies Railway systems Heavy industries, smelters	General corrosion of rebar exposed to moist conditions Corrosion not confined to low cover depths Large crack widths possible
Chemical induced	High sulphate groundwaters Fertilizer factories Industrial plants Sewage treatment works	Corrosion generally associated with near saturated conditions Concrete deterioration occurring together with corrosion
Secondary forms	Primary cracking due to alkali aggregate reaction, delayed ettringite formation, structural cracking	Corrosion localized in regions where cracks intersect rebar Other forms of distress evident in concrete (i.e. AAR gel deposits)
Artificially induced	Bimetallic corrosion Partial sealing of concrete High temperatures (>200 <sup>o</sup> C) Patch repairs of corrosion	Generally very localized intense corrosion due to well defined anode/cathode regions

**Table A4: Diagnostic sheet for concrete deterioration (all forms)**

<i>Type of deterioration</i>	<i>Visual evidence / associated factors</i>	<i>Confirmatory testing</i>
Reinforcement corrosion	Rust stains, cracking along reinforcement, spalling of cover concrete, delamination of cover concrete	Cover depth of rebar Carbonation & chloride testing Exploratory coring Electrochemical testing
Alkali aggregate reaction	Expansive map cracking, restrained cracking following reinforcement, white silica gel at cracks	Core analysis for gel and rimming of aggregates Petrographic analysis Aggregate testing
Shrinkage/creep	Characteristic cracking, excessive displacements, time dependent movements, exposure to drying conditions	Concrete core analysis Loading and structural analysis Aggregate and binder analysis
Chemical attack	Surface attack, salt deposits on surface, expansive internal reactions causing cracking, exposure to aggressive waters	Chemical analysis of concrete Core examination for depth of attack and internal distress
Softwater attack	Surface leaching of concrete, exposed aggregate, exposure to moving waters in conduits	Chemical analysis of water Core examination for leaching damage
Fire damage	Surface discolouration, concrete spalling, thermal cracking, buckling, loss of strength, microcracking	Core examination for colour variations, steel condition Petrographic analysis Specialist techniques
Structural overload	Major cracking in areas of high stress, localized crushing, excessive deformations and deflections	Loading and structural analysis Core testing for compressive strength and elastic modulus

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