

Cost-Effective Remedial Techniques for Reinforcement Corrosion in Concrete Road Bridges

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Executive Summary

Aim of Research

This report represents the second stage of an evaluation of the long-term performance of proprietary patch repair systems, and electrochemical treatments used to rehabilitate concrete bridges suffering from corroding reinforcement. It follows on from, and complements, a description of electrochemical treatments reported by Freitag & Bruce in 2002. The combined findings of these two reports will enable New Zealand bridge asset managers to select repair technologies to suit the needs and circumstances of individual structures, thereby optimising cost and long-term performance requirements. The information will also allow bridge owners to furnish more accurate information for their long-term financial strategies.

Scope of Report

This report was written to disseminate information about current patch repair technology. It is designed to alert asset managers to features that might be relevant when they are considering the options for rehabilitating a structure, rather than to provide a comprehensive guide to the management and repair of affected structures.

The report describes the factors that need to be considered when developing an appropriate course of treatment for an individual structure, technical features of materials available in New Zealand for repairing damage caused by reinforcement corrosion in concrete, case histories illustrating the performance of patch repair and cathodic protection, and includes a comparison of immediate and life-cycle costs of patch repairs and electrochemical treatments.

The information presented is drawn from international reviews and local trade literature, and from discussions with asset managers in New Zealand and Australia during 2002-2003. Readers are directed to consult technical literature, standards and suppliers for more detailed information when considering specific applications.

Repair Strategies

Specialist technical investigation is required to identify the cause, extent and significance of deterioration and to predict the likelihood of future deterioration. Following this, a strategic approach taking into account the short-term and long-term needs of a structure will ensure the most cost-effective repair technique. In practice, immediate costs rather than strategic aspects usually take priority when deciding how to repair an individual structure.

Nevertheless, whatever the approach taken, an understanding of the principles behind various repair methods, and the benefits that each can deliver, will help to ensure that appropriate types of treatment are selected for the problems exhibited on a specific structure.

Patch Repairs

Patch repair materials are either batched on site, or supplied as a proprietary system of pre-batched materials designed to be used together. Proprietary products offer better quality control and are purpose-designed for optimum performance in different situations. Many suppliers provide specifications and trained applicators to support the performance guarantees of their products. Proprietary systems are therefore preferred over site-batched materials.

Individual products within proprietary repair systems include reinforcement coatings, bonding agents, patch repair materials, corrosion inhibitors, internal sacrificial anodes and surface treatments. The components selected for a given repair will depend on the individual circumstances of the repair.

As with electrochemical repair methods, an understanding of the principles behind individual products, and knowledge of their specific advantages and limitations, are necessary to ensure that appropriate products are selected. Specialist technical advice should be sought from suppliers of proprietary materials or from independent experts to ensure that the repair materials selected are appropriate for the treatment principles and methods adopted. Previous experience with individual products and contractors will also assist in ensuring that appropriate products are selected, and that they are applied correctly.

Repair Performance

Although patch repair often needs to be applied in repeat cycles, bridge owners generally consider the performance of patch repairs to be acceptable. The main problems reported are related to achieving repairs of acceptable quality for the lowest possible immediate cost.

Site evidence from previously repaired structures suggests that patch repair cycles are typically 5-10 years for NZS 3101:1995 exposure zone C (tidal/splash), 10-15 years for exposure zones B1/B2 (coastal perimeter and coastal frontage), and 20 years or more for exposure zone A2 (inland). The length of the repair cycle on a given structure is determined by the specific exposure conditions of the site, the quality of concrete and construction, the quality of the repair, and the level of ongoing damage that is considered acceptable. The length of repair cycles could be increased by incorporating surface treatments or internal sacrificial anodes in the repair system.

Electrochemical treatments are promoted as providing longer term protection than an individual cycle of patch repair. Owners of bridges protected by CP systems have not questioned its long-term performance, but they stressed the need for good management systems to support the long-term operation of CP systems. This includes ensuring that annual budgets include CP operating expenses, as well as systems for managing and correlating the outputs of CP monitoring and visual inspections. When installing a CP system for the first time, bridge owners should consider how they would manage more than one CP structure should they subsequently invest in more installations. The uncertainty in ongoing CP operating costs needs to be accommodated in analyses of life-cycle costs of repair options.

Australasian bridge applications of chloride extraction and realkalisation have been too recent to demonstrate the effectiveness of the treatments, so owners of these bridges were not contacted.

Assessment of Cost Effectiveness

The cost analysis of repair options for a particular structure (which may or may not include electrochemical treatments) should identify the factors that will have the most effect on costs if they vary from the initial estimate. It should then analyse the effects of the likely extreme values of these factors on the final cost. The ultimate decision can then be based on the likelihood of these extremes being reached. Any cost comparison must carefully consider repair volumes and issues relating to accommodating traffic during the repair process, and use data from reliable sources. Life-cycle cost analyses also need to consider the required future service life of the structure.

Immediate costs of patch repair will almost always be lower than the costs of installing electrochemical treatments. Life-cycle costs will also usually be lower for patch repairs, but can favour electrochemical treatments under certain circumstances for bridges with an indefinite required future service life. Such circumstances include structures severely damaged by carbonation-induced corrosion or which have extremely high exposure to chlorides such as in marine structures, and where significant costs related to access or traffic are incurred during repair. Technical advice should be sought from experienced specialist contractors, suppliers, or consultants to ensure that appropriate treatments and products are applied.

Future Developments

A model specification for patch repairs based on up-to-date technology would help to ensure a consistently high standard of patch repair being carried out on New Zealand bridges.

Electrochemical repair techniques, surface treatments and internal sacrificial anodes can offer cost advantages over the service life of the structure, but are unlikely to be selected other than for trials if only immediate installation costs are considered. Increasing development and adoption of bridge asset management practices based on life-cycle costing may encourage the use of electrochemical repair techniques, surface treatments and internal anodes, in specific circumstances.

Recommendations

It is recommended that:

- Bridge asset managers take a strategic approach when assessing repair options for a bridge affected by reinforcement corrosion. To ensure the most cost-effective repair technique, the immediate and long-term needs of a structure and the communities it serves need to be taken into account.
- A formal inventory system be utilised to record details of bridge condition and remedial treatments carried out. This will help to establish the long-term needs of individual structures.
- Remedial treatments be appropriate for both the technical and strategic needs of the individual structure concerned. Therefore repair systems and products should be selected with an understanding of the principles behind them, their specific advantages and limitations, their long-term performance, and the required service life of the structure. Ongoing maintenance associated with treatments such as surface treatments and cathodic protection also needs to be considered. Specialist technical advice should be sought to ensure that appropriate treatments are selected.
- A model specification for patch repairs based on up-to-date technology be adopted. This would help to ensure that a consistently high standard of patch repair is carried out on New Zealand bridges.
- Analyses of treatment costs include both immediate and long-term costs. They should include a sensitivity analysis to determine the factors that will have the most effect on costs should they vary from the initial estimate, and the effects of the likely extreme values on the final cost. Particular attention should be paid to costs associated with access and traffic control, both of which are sometimes overlooked, and with repair areas and volumes, which are easily under-estimated.

- Technical advice be sought from experienced specialists to identify the cause, extent and significance of deterioration, to predict the likelihood of future deterioration, and to ensure that appropriate remedial treatments are selected for individual structures.
- Electrochemical treatments be considered under certain circumstances, particularly for structures severely damaged by carbonation-induced corrosion or which have extremely high exposure to chlorides such as in marine structures, and where significant costs related to access or traffic are incurred during repair.

Abstract

Corrosion of steel reinforcement is the most serious and common deterioration problem affecting reinforced concrete bridges in New Zealand. Remedial treatments for concrete damaged by corroding reinforcement include electrochemical treatments (cathodic protection, realkalisation and chloride extraction), which potentially offer a permanent treatment, and patch repair, which is cheaper but does not always prevent future deterioration.

This research, carried out in 2002-2003, complements a 2001-2002 review of electrochemical treatments. It describes the features of current patch repair techniques, and uses New Zealand and Australian experience to compare the likely long-term performance and costs of both types of treatment for New Zealand bridges. This will enable New Zealand bridge asset managers to select appropriate repair technologies to suit the needs and circumstances of individual structures.

The research indicates that immediate costs of patch repair will almost always be lower than the costs of installing electrochemical treatments. Life-cycle costs will also usually be lower for patch repairs, but can favour electrochemical treatments under certain circumstances.

1. Introduction

1.1 Background

Corrosion of steel reinforcement is the most serious and common deterioration problem affecting New Zealand reinforced concrete structures. Steel will corrode when exposed to moisture and oxygen, and the corrosion process is accelerated in the presence of chloride ions. In New Zealand chloride ions are most often introduced into the concrete by exposure of the hardened concrete to sea water or sea spray, but also sometimes by the use of unwashed marine aggregates and admixtures containing chlorides.

Normally the surrounding concrete provides a relatively impermeable barrier to prevent moisture, oxygen and chlorides reaching the steel. It is also highly alkaline, which prevents the corrosion reactions occurring. But if insufficient concrete surrounds the reinforcement, or if the concrete is of poor quality and easily transmits air and water, then the reinforcement will start to corrode. The corrosion products occupy a greater volume than the original steel, and their formation will first crack, then break off, the overlying concrete. As the diameter of the corroded reinforcing bar diminishes, its strength reduces, and eventually the load capacity of the structure will diminish. It is therefore important to repair damage caused by reinforcement corrosion in order to maintain structural capacity and prevent damage to people, property, vehicles etc., by concrete falling off the structure.

All types of reinforced concrete structures, including bridges, are prone to corrosion damage. Those exposed to sea water or sea spray are at greater risk, as are surfaces exposed to rain or run-off. Road bridges are affected because many (60% of state highway bridges and 45% of local authority bridges) are in coastal environments, and because they all have surfaces exposed to water. Older bridges are more likely to be affected, first because construction quality was often poorer before 1950, and second because the corrosion damage increases with time. In New Zealand, over 30% (approx. 770) of state highway concrete bridges and 14% (approx. 1180) of local authority concrete bridges are more than 50 years old. Although many are still in acceptable condition, the incidence of deterioration caused by reinforcement corrosion is likely to increase as they age further, increasing overall maintenance costs for concrete bridges. Transit New Zealand's allocation for bridge maintenance in 2003-2004 estimates the cost of structural repair of New Zealand state highway bridges to be NZ\$5.7 million¹ for high and medium priority bridges, with more than \$3.3 million to be spent on routine maintenance, and \$5.5 million on other structures.

Technology for repairing concrete structures affected by reinforcement corrosion has advanced significantly in recent years, and several different approaches are commercially available. The electrochemical processes known as cathodic protection, chloride extraction and realkalisation have a high initial cost but are promoted as providing a permanent solution, while conventional systems, which combine patch repairs and surface treatments, are cheaper to install but have a

¹ All costs are in NZS.

limited life. Major patch repairs to a bridge can cost around \$40,000 per cubic metre of repair material. Cathodic protection costs about \$600 per square metre of treated concrete surface to install and requires ongoing monitoring and maintenance. Older technologies such as epoxy patch repair materials are still available and used even though they are no longer believed to be appropriate for repairing corrosion damage.

Different techniques are used depending on whether the corrosion was initiated by chlorides or by loss of alkalinity caused by exposure to carbon dioxide. Sometimes the repair needs to be permanent, and sometimes it may be needed to serve for only a finite time before major modifications or reconstruction. Consequently, when repair is required, asset managers need to identify the most effective and economic method for each particular case, taking into account the initial cause of the corrosion, the anticipated functional life required for the structure, and the long-term cost and predicted life of the repair. To do this, asset managers need to be familiar with the long-term performance and costs of these repair systems. However most systems are too recent for individual manufacturers or users to have built up extensive experience with them. Research by Bruce et al. (1999) suggested that bridge managers need more up-to-date information on corrosion treatments, and that lack of local evidence of performance is a major obstacle to the uptake of electrochemical techniques.

An independent review of electrochemical treatments was thus considered the priority for the first phase of this research, and the findings were reported in 2002 (Freitag & Bruce 2002). They confirmed the widespread use overseas of electrochemical repair techniques on bridge structures similar to those in New Zealand, although evidence of long-term performance was difficult to locate. The review showed that a critical influence on long-term performance is the selection of appropriate electrochemical repair techniques assisted by experienced electrochemical repair consultants.

The second phase of research, reported herein, reviews other remedial techniques such as patch repairs and surface coatings. Logistics preclude carrying out and monitoring repairs as long-term trials in New Zealand. As with the electrochemical treatments, many case histories in the international literature report the selection and application of particular systems, but fewer reports are available on their long-term performance. Investigations into the application and initial properties of generic repair systems carried out for the Road Research Unit of the National Roads Board in the 1980s (Freitag 1988) are still relevant to some repair systems but do not provide information on long-term performance. No large-scale investigations of current repair technologies are in progress in New Zealand, although some individual asset owners have initiated limited trials for their own purposes.

1.2 Scope of Research

The research reported here was carried out in 2002-2003. It represents the second stage of the project, and aims to evaluate existing evidence on the long-term performance of proprietary patch repair systems and electrochemical treatments used to rehabilitate concrete bridges suffering from corroding reinforcement. It is designed to improve knowledge about and confidence in using proprietary materials and techniques that represent state-of-the-art repair technology. The findings of this work

and the preceding review of electrochemical techniques (Freitag & Bruce 2002) will thus assist New Zealand bridge asset managers in selecting technically appropriate and cost-effective repair technologies to suit the needs and circumstances of individual structures. The information will also allow bridge owners to furnish more accurate information for their strategic asset management plans.

Some information was obtained from technical literature and reports, but much of it was gained by direct contact with New Zealand and Australian asset owners. They were asked about their experiences with various proprietary treatments, the process they used to select the repair system, the options they considered, how they made their selection, whether they experienced any installation problems, how they are monitoring the performance of the repair, and what the performance to date has been.

1.2.1 Report Contents

This report summarises the types of patch repair and surface treatments available and their performance characteristics. It describes the factors that need to be considered when deciding on an appropriate course of treatment for an individual structure, but is not intended to be a comprehensive guide to reinforcement corrosion, the theory and practice of its treatment, or the strategic management of affected structures.

Section 2 describes the basic principles of reinforcement corrosion.

Section 3 describes the factors that need to be considered when determining the strategy for treating reinforcement on a particular structure.

Section 4 describes the materials used in current patch repair products and systems.

Section 5 describes how New Zealand bridge asset managers currently manage the treatment of reinforcement corrosion.

Section 6 describes the long-term performance of treatments, using case studies of patch-repaired structures and the experiences of bridge asset managers and contractors who have applied electrochemical treatments.

Section 7 describes the factors that influence the cost of patch repair and electrochemical treatments.

More detailed information on concrete repair is available in Australian Concrete Repair Association (ACRA 1996), Broomfield (1997), Building Research Establishment (BRE 2000) and from the websites of the Corrosion Prevention Association and Concrete Repair Association, www.concreterepair.org.uk and www.corrosionprevention.org.uk. The *fédération internationale du béton* (fib 2002) describes issues relating to the management, maintenance and strengthening of concrete structures, including asset management considerations for owners and managers, and technical information to help consultants and contractors select appropriate remedial strategies. NACE has published a standard recommended practice for corrosion control of concrete structures (NACE 1998).

1.2.2 General Definitions

In this report, the following definitions are used:

“Patch repair”: is the partial replacement of concrete in an element. It includes materials applied by trowel, spray-applied, or cast into formwork.

“Electrochemical techniques” include:

- cathodic protection (CP);
- chloride extraction (also sometimes called desalination or electrochemical chloride extraction);
- realkalisation.

2. Corrosion Principles

‘Corrosion’ can be defined as the destruction or deterioration of a material by reaction with its environment. It can refer to the process or to the damage caused, and applies to metals and non-metals. In this report ‘corrosion’ refers to the corrosion of steel reinforcement in concrete.

Metals are normally found in nature as oxides, called ores. This is their most stable (or lowest energy) form. To obtain the metal, energy is applied to separate the chemical elements present in the ore. More energy is then applied to shape the metal into a product. The finished metal product has more energy than it needs to maintain equilibrium in the natural environment, and so tends to release this extra energy and return to its natural, low energy state. This is the process of corrosion. For example, heat and mechanical energy are applied to produce steel from iron oxide and, on exposure to moisture and oxygen, the steel corrodes back to stable iron oxides, releasing energy in the form of electrical corrosion currents.

Metallic corrosion occurs by electrical charges moving between sites on a metal surface to balance charges generated by chemical reactions at each site. Both electrical and chemical processes are involved, so corrosion is referred to as an ‘electrochemical’ process.

Metallic corrosion has four requirements:

- a metallic site with a high potential for releasing energy;
- a metallic site with low potential for releasing energy;
- a metallic connection that allows the flow of electrons (electric current) between them; and
- an electrically conductive solution (electrolyte) in contact with both sites that allows the flow of ions.

2.1 Anodes and Cathodes

The two metallic sites are called the electrodes. One, the anode, releases energy in the form of electrons. The other, the cathode, receives the electrons released at the anode. The difference in energy between the electrodes is the voltage, or electrochemical potential, and this is what drives the corrosion reactions. The amount of material lost from the corroding electrode is determined by the amount of corrosion current generated. This is a function of the voltage between the two electrodes and the electrical resistance of the materials through which electrons and ions must flow.

Features of the two electrodes, and the chemical reactions that occur at them are described in detail by Freitag & Bruce (2002) and are summarised below.

The *anode* has a greater potential to release energy. The metal at the anode dissolves, releasing energy in the form of electrons. The chemical reactions are called oxidation reactions. The anode has a positive charge because it loses negatively charged electrons.

The *cathode* has less energy. It accepts the electrons released by the anode, and takes them up in chemical reactions called reduction reactions. They do not necessarily involve the cathode metal itself. The cathode has a negative charge because it gains electrons. Cathodic reactions often involve the conversion of oxygen to hydroxyl ions.

When steel corrodes, the positive iron ions generated at the anode react with negative hydroxyl ions generated at the cathode, to form iron hydroxides. These eventually form the orange-coloured product we recognise as rust.

Any metal with a higher potential energy can release energy to any metal with lower energy, and thereby corrode. Similarly, any metal with lower energy can receive energy from any metal with higher energy, and thereby be protected from corrosion.

A metal's potential energy, or tendency to dissolve, depends upon its immediate environment, particularly the availability of oxygen, water, and contact with chemicals that it (or its ions) might react with. A single piece of metal is often exposed to slightly different environmental conditions over its surface area. If these differences are large enough, anodes and cathodes will be set up on its surface and it will corrode even if not in contact with another metal. Inclusions of other metals exposed on the surface of an alloy, or local differences in the metal's microstructure caused by working (welding, cutting, bending, etc.) can have the same effect.

'Galvanic corrosion' is the term given to corrosion driven by an energy difference between two different metals.

2.2 Electrical Conductor

The metallic conductor in reinforced concrete is the reinforcement itself.

2.3 Electrolyte

Moisture in the concrete provides the electrolyte through which ions are transported. The lower the moisture content, the greater the restriction on ion transport and consequently the higher the electrical resistivity. The resistivity of concrete is a major factor in determining the rate of corrosion. Resistivity is also related to the chemical composition of the water in the concrete pores (i.e. the 'pore solution').

2.4 Corrosion of Steel in Concrete

Steel does not normally corrode in concrete because the high alkalinity of Portland cement paste produces a protective film of iron oxide on the steel surface. The steel is then said to be in a passive state or passivated.

In normal alkaline concrete the oxide film is stable enough to protect the steel against small variations in environment, but it becomes unstable in the presence of chloride ions or if the alkalinity of the pore solution is reduced by carbonation². Thus chloride ion contamination (such as in marine environments), carbonation, and large variations in the concrete environment (e.g. areas of poor compaction) will lead to corrosion of the reinforcement. Depending on the circumstances, the oxide film can be reinstated, thereby repassivating the steel, by removing chloride contamination and/or restoring the alkalinity of the pore solution.

The amount of chloride required to depassivate the steel increases with the alkalinity of the pore solution, which is expressed as the hydroxyl ion concentration. Therefore sometimes chloride contamination is considered in terms of the chloride to hydroxyl ion ratio rather than simply as the percentage of chloride ion present.

In the usual situation in concrete, no other metals are present and local differences in steel composition, microstructure (Section 2.1), and environmental conditions cause steel reinforcement to develop anodes and cathodes and then corrode. Local differences in the immediate environment of the reinforcement include availability of oxygen and moisture, chloride ions, and alkalinity of pore solution in contact with the steel. The anode and cathode can be on the same piece of reinforcement or on different pieces, as long as a conducting material connects them. Corrosion may be very localised, producing a pitted appearance and local loss of cross section, or it may be randomly distributed causing loss of section and bond over the affected length of reinforcement.

Galvanic corrosion can develop if the reinforcement is in contact with fasteners, metal conduits, tie wire or other pieces of reinforcement made from metal having lower energy.

Corrosion will begin when the energy difference ('potential difference') between the anode and the cathode reaches a critical level. This event is called 'corrosion initiation'. It represents a particular reduction of pore solution alkalinity and/or level of chloride ion contamination being reached in the concrete in contact with the steel surface. The actual critical values for a piece of reinforcement will depend on the concrete quality and the relationship between alkalinity and chloride contamination. The time to corrosion initiation can be estimated by inputting information on concrete composition, pore solution alkalinity, and chloride contamination into predictive models.

Corrosion rate is determined by the availability of moisture, i.e. the electrolyte, and oxygen, which is involved in the corrosion reaction at the cathode. Thus corrosion rates are slow in concretes that are permanently saturated (e.g. piers that are permanently immersed in water) or never wetted and only exposed to atmospheric moisture (e.g. inner beams and deck soffits).

² "Carbonation" is a chemical reaction between atmospheric carbon dioxide and components of hydrated cement. It reduces the alkalinity of the pore solution to a level where the protective iron oxide film on the steel surface is no longer stable.

Corrosion rates are rapid in concrete exposed to intermittent wetting and drying by tidal movements (e.g. piers), splash (e.g. substructure and superstructure elements above the tidal zone but splashed in storm conditions), or rain and run-off (e.g. outer beams and deck soffits). Corrosion rate is more difficult to predict than corrosion initiation.

The time to corrosion initiation and the subsequent corrosion rate both need to be considered when assessing the service life of a structure. Long-term performance and cost of remedial treatments in relationship to the desired service life are discussed in Chapters 6 and 7 of this report.

3. Repair Strategies

3.1 Developing a Strategy for Managing Reinforcement Corrosion

When reinforcement corrosion has been diagnosed in a concrete structure, a strategy for managing the deterioration must be established to ensure that serviceability is maintained for the required period for an acceptable cost. It should include both repair and protection from further damage.

The strategy will be specific to the structure and will account for economic, technical/structural, performance, statutory and environmental factors. It will include the following information:

- the type of structure;
- its condition;
- its design life and required remaining service life;
- the type, cause and rate of deterioration;
- past and future use and service conditions (i.e. environmental and structural loading) and associated performance requirements;
- anticipated future need for the structure in its present form and location (i.e. ‘functional service life’);
- potential benefits from structural improvements;
- structural performance and sensitivity;
- consequences of failure and risks associated with spalling;
- previous treatments;
- compatibility between treatments needed for different parts of the structure;
- durability and repair history of nearby or similar structures;
- regulations affecting repair process (e.g. compliance with OSH, RMA, Building Code, and Historic Places Act requirements);
- availability of repair materials and applicators;
- possible effects on structural performance during repair;
- access to the site and to the affected parts of the structure;
- impact on users during remediation;
- power supply and facility to transmit data for remote monitoring;
- susceptibility to vandalism, impact or damage by natural exposure conditions;
- aesthetic requirements;
- long term performance of treatment and its effect on subsequent treatments;
- anticipated number, cost of and disruption caused by subsequent repair cycles;
- ability to maintain the treatment;
- ability to interpret and manage the data from monitoring systems;
- possible future need for additional protection and/or monitoring;

- immediate and long-term costs and budget constraints;
- value added by remediation.

This document does not consider the effects of statutory requirements on the strategy or methods selected. For example, the Historic Places Act (1993) considers all New Zealand structures built before 1900 to be archaeological sites and they must not be altered or repaired in any way without obtaining prior authority from the Historic Places Trust. Reinforced concrete was first used in New Zealand bridges in 1899 (Thornton 2001) so this is unlikely to be a common issue. However a bridge could still have a Historic Places Trust classification that requires a resource consent or consultation with the NZ Historic Places Trust.

The cost and performance of several remedial options should be considered when developing the strategy. Also an important consideration is the effect of the remedial options on additional treatment that might be needed in the future. For example water-repellent surface treatments will prevent the future application of electrochemical treatments.

Table 3.1 summarises the sequence involved in establishing a strategy for managing deterioration on a concrete structure. This table is an overview of the repair and remediation process outlined by DD ENV 1504-9:1997, a draft standard published by the British Standards Institution (BSI) that is intended to form the basis of repair specification in the UK and Europe (see Section 3.4.6 of this report). Sections 3.2 to 3.6 describe each stage in the process.

3.2 Condition Assessment

Once routine inspection has identified reinforcement corrosion damage, the first stage in determining a treatment strategy involves a detailed diagnosis of the cause, extent and structural significance of deterioration. This is critical to the selection of an appropriate management approach, and of principles and methods of treatments in the following stages. The findings should be used to predict the residual life of the structure with and without treatment. Procedures for inspection, diagnosis of defects and prediction of future deterioration are given elsewhere (e.g. ACRA 1996, Broomfield 1997, BRE 2000). Experienced specialists should be employed to carry out such investigations. They may be contractors, suppliers of remedial treatments, or independent consultants. Costings for various repair options may be prepared as part of this stage.

3.3 Options for Managing Deterioration

The second step is to choose an approach for managing deterioration caused by reinforcement corrosion. Options include:

- do nothing for now and monitor changes in condition;
- re-analyse and perhaps downgrade structural capacity;
- evaluate and possibly shorten the estimated remaining service life;
- replace cracked and spalled concrete (patch repair);

3. *Repair Strategies*

- prevent or reduce further deterioration (surface treatment, electrochemical treatment);
- improve, strengthen or refurbish all or part of structure (strengthen);
- rebuild all or part of structure;
- demolish all or part of structure.

Table 3.1 Process for establishing a strategy for managing deterioration on a concrete structure (based on DD ENV 1504-9:1997, published by BSI).

ACTION	ASPECTS TO CONSIDER	
1. Assess condition, identify cause and extent of defects	<ul style="list-style-type: none"> • present condition • original design approach • environment and contamination • construction conditions • past and future service conditions • past and projected future use 	
2. Select options for managing deterioration (e.g. do nothing, repair, demolish, etc.)	<ul style="list-style-type: none"> • intended use, design life • required residual life performance characteristics • long-term performance of treatment • additional future protection and monitoring • number and cost of repair cycles • cost and funding of alternative options, including future maintenance, compliance and access costs • substrate properties and preparation • appearance of treated structure 	
3. Identify treatment principle	Treatment principles are: <i>For concrete defects</i> <ol style="list-style-type: none"> 1. Protection against ingress 2. Moisture control 3. Concrete restoration 4. Structural strengthening 5. Physical resistance 6. Chemical resistance 	<i>For reinforcement corrosion</i> <ol style="list-style-type: none"> 7. Preserve/restore passivity 8. Increase resistivity 9. Cathodic control 10. Cathodic protection 11. Anodic control
4. Select method of treatment	<ul style="list-style-type: none"> • type, cause and extent of deterioration • effect of site conditions on treatment process • effect of site and service conditions on durability of treatment • facility to maintain the treatment • repair/replacement/monitoring approach selected • treatment principle • appearance of treated structure • cost and availability of products/systems of suitable type and quality 	
5. Select materials	<ul style="list-style-type: none"> • characteristics of materials for the specific application • cost and availability 	
6. Define inspection and maintenance requirements	<ul style="list-style-type: none"> • record work carried out • provide instructions for inspection, maintenance and repair during the remaining life of the structure • establish a system to manage maintenance of the treatment 	

The decision is rarely clear-cut except in the following circumstances:

- ‘do nothing’ is appropriate because the deterioration is so minor;
- demolition is appropriate because the structure is in such poor condition or because it no longer fulfils functional needs (e.g. road needs realigning and the

bridge is either no longer required or repairs/modification of the bridge is not economic).

A combination of the above options is more usual. The most appropriate approach will be determined by a combination of technical and economic needs. Economic analyses are usually used to assist in ranking the technical options.

This report describes materials for repair and surface treatment/coating, but not strengthening techniques.

3.4 Treatment Principles and Methods

The third step is to choose a treatment principle that is appropriate for the cause of and extent of deterioration, the required remaining service life, and the anticipated service conditions. Treatment methods are often selected at the same time, or indeed without consideration of the underlying principle, so are also described in this Section 3.4. However the principle behind each treatment will determine its long-term effectiveness in a given application, so no long-term strategy can be implemented without an understanding of the treatment principles.

The treatment principles 3.1-3.11 listed in Table 3.1 were developed by the British Standards Institution (DD ENV 1504-9:1997). The treatment principles outlined for reinforcement corrosion (principles 7-11) are based on controlling one or more of the four elements of corrosion (see Chapter 2): chemical reactions at the anode, chemical reactions at the cathode, transfer of electrons from anode to cathode through the reinforcement (i.e. the electrical conductor), or transfer of ions through the pore solution (i.e. the electrolyte). The treatments are designed to reduce the risk of future corrosion, as opposed to simply replacing the damaged concrete to restore the original appearance (principles 1-6). Table 3.2, which is also based on DD ENV 1504-9:1997, defines principles 7 to 11 in more detail and describes treatment methods based on each principle.

Table 3.3 shows which principles are suited to which causes of corrosion. Appendix A reproduces decision trees originally published by Broomfield (1997), which describe the process of identifying technically appropriate corrosion treatments (these were also reported in Freitag & Bruce 2002).

Table 3.3 Treatment principles for different causes of corrosion

Cause of corrosion	Treatment principle
Carbonation of cover concrete	7. Preserve/ restore passivity 11. Anodic control
Chloride contamination	7. Preserve / restore passivity 9. Cathodic control 10. Cathodic protection 11. Anodic control
Stray currents	8. Increase resistivity

3. Repair Strategies

Table 3.2 Principles and methods for remedial treatments (from BRE 2000, based on DD ENV 1504-9:1997).

Principle	Principle Definition	Methods based on Principle
7. Preserve or restore passivity	Creating conditions in which the surface of the steel can maintain or return to an electrochemically passive condition.	<p>7.1 Increase cover to reinforcement with additional cementitious mortar or concrete</p> <p>7.2 Replace chloride-contaminated or carbonated concrete</p> <p>7.3 Electrochemical realisation of carbonated concrete¹</p> <p>7.4 Realisation by diffusion</p> <p>7.5 Electrochemical chloride extraction¹</p>
8. Increase resistivity	Increasing the electrolytic resistivity of the concrete.	8.1 Limit moisture content of the concrete by surface treatments, coatings, or sheltering
9. Cathodic control	Creating conditions in which the cathodic processes are restricted so that cathode reactions can no longer balance corrosion at the anode.	<p>9.1 Limit oxygen content by saturating the concrete or by surface coating^{2,3}</p> <p>9.2 Apply a cathodic inhibitor^{1,2,3}</p>
10. Cathodic protection (CP)	Making the reinforcement into a cathode.	<p>10.1 Impressed current CP¹</p> <p>10.2 Galvanic (sacrificial) CP¹</p>
11. Anodic control	Creating conditions in which anodic processes are restricted.	<p>11.1 Paint reinforcement with coatings containing active pigments²</p> <p>11.2 Paint reinforcement with barrier coatings^{2,3}</p> <p>11.3 Apply an anodic inhibitor^{1,2,3}</p>

BRE (2000) made the following comments on the DD ENV 1504-9:1997 methods:

1. Methods may use products and systems not covered by the EN 1504 series of European standards.
2. BRE (2000) considers there is an absence of evidence that the method achieves protection by the appropriate principle.
3. Inclusion of methods does not imply BRE approval or confirmation of their effectiveness.

Corrosion can be caused by different factors on different parts of the same structure and the methods of treatment should be appropriate for each cause. For example on one estuarine bridge in Australia corrosion of the superstructure was attributed to carbonation and treated by realkalisation of the edge beams and patch repair to the internal surfaces, while corrosion in the chloride-contaminated pier columns and tie beams was treated by CP (Concrete in Australia 2002).

As with the treatment principle, the treatment method should also be appropriate to the cause and extent of deterioration, the required remaining life, anticipated service conditions and the materials available. The following factors need to be considered when selecting a method of treatment (from Table 3.1 (action 4) and fib 2002):

- Type, cause and extent of deterioration;
- Treatment principle;
- Repair/replacement/monitoring approach selected;
- Effect of site conditions on treatment process;
- Effect of site and service conditions on durability of treatment;
- Facility to maintain the treatment;
- Cost and availability of products/systems of suitable type, characteristics and quality;
- Chemical and physical condition of the substrate or contaminants;
- The ability of the structure to accept loading and movement during treatment and extra dead load resulting from the treatment;
- The ability of the treatment to withstand movements of the structure during and after treatment;
- The mechanical strength and stability of the structure during and after treatment;
- Compatibility between repair material and substrate;
- Compatibility between treatment and reinforcement/prestressing;
- Compatibility between different repair materials;
- Required appearance after treatment;
- Galvanic corrosion cells that might be created by the treatment;
- Electrical properties of structure and repair materials must be appropriate if electrochemical treatments are selected;
- The effect of the treatment on options for future treatments should they be needed;
- Quality systems to ensure that substrate preparation and use of proprietary systems comply with the manufacturer's written recommendations;
- Level of supervision available;
- Health and safety risks to workers and the public during the treatment.

A combination of methods that addresses more than one of these principles is often appropriate, for example patch repair to restore passivity (principle 7.2) supplemented by discrete internal anodes to cathodically protect adjacent rebar from incipient anode corrosion (principle 10.2; Section 3.4.1.1), and/or a coating to reduce

the moisture content of the concrete (principle 8.1) and thereby increase resistivity (principle 8.1).

Treatment principles 7 to 11, and the associated methods, are described in Sections 3.4.1 to 3.4.5. Standard specifications covering repair are listed in Section 3.4.6. Individual repair materials are described in Chapter 4 of this report.

3.4.1 Preserving or Restoring Passivity

This method preserves or restores the passivating iron oxide film on the steel surface, either by preventing carbonation or chloride contamination reaching the steel, or by restoring the alkalinity and reducing the chloride ion content of the surrounding concrete. It includes patch repair, realkalisation by diffusion, electrochemical realkalisation and electrochemical chloride extraction.

3.4.1.1 Patch Repair

Patch repairs in the form of overlays can be applied over existing cover concrete to increase the cover depth, and thus prevent or delay the carbonation front or chlorides reaching the steel (principle 7.1). However the existing cover concrete must not be significantly carbonated or chloride-contaminated, and the effects of increased weight and changed profile of the structure must be assessed.

Patch repair is more often applied to replace chloride-contaminated or carbonated cover concrete (principle 7.2). The process is similar for both, although the long-term performance may differ. The procedure will vary with the needs and restrictions of individual sites, but will normally include cleaning of the concrete surface, location of defects, removal of damaged, carbonated or chloride-contaminated concrete, preparation of the exposed reinforcement, and reinstatement of the concrete. These processes are detailed elsewhere (e.g. ACRA 1996, BRE 2000). Chapter 4 describes the products and systems used.

Patch repair with an alkaline material will repassivate the steel embedded in the repair material. This steel is now in a different environment to steel that remains in the original carbonated or chloride-contaminated concrete. If it is electrically connected to steel in the original concrete a corrosion cell is set up, the cathode being the steel in the repaired area and the anode being steel in the original concrete. This is called the incipient anode effect. It is the reason why patch repair often does not prevent further corrosion, even if the repair material itself bonds well to the substrate and is highly impermeable to air, moisture and chloride ions.

It is rarely practical to remove all carbonated or chloride-contaminated concrete from around the reinforcement. The significance of the incipient anode effect that results will depend on the amount of moisture available. Corrosion rates will be relatively slow in concrete that is not frequently wetted or exposed to high relative humidity. On surfaces that are exposed to rain, run-off or high humidity, applying a water-resistant surface treatment to the repaired element can reduce the rate of subsequent corrosion adjacent to the patch repairs. Another way to control the incipient anode effect in patch repairs is to connect a sacrificial anode to the reinforcement at the edge of the original concrete.

Chloride extraction can be applied to prevent incipient anodes in concrete that was contaminated by chlorides when it was mixed (either from salt-contaminated aggregates or chloride-based accelerating admixtures). A CP system designed to protect the whole element will override incipient anode effects.

3.4.1.2 Electrochemical Realkalisation

Electrochemical realkalisation (principle 7.3) was described in detail by Freitag & Bruce (2002). Cracked and spalled cover concrete is replaced, then the reinforcement is connected to a power supply, and a temporary anode within an alkaline electrolyte applied to the concrete surface. Hydroxide ions are generated at the reinforcement, increasing the pore solution alkalinity. The process also draws the alkaline electrolyte into the concrete. After patch repair to replace spalled and cracked concrete has been carried out, the process takes up to two weeks. With an electrolyte formulated to chemically buffer subsequent carbonation, the effects can be permanent.

Because carbonated concrete that is still sound does not need to be removed, electrochemical realkalisation can be used where carbonation beyond the reinforcement is widespread and corrosion rates are significant. It can also be used as a preventive treatment if applied before the concrete is damaged, for example where cover depths are low over a wide area. It cannot be used on pre-tensioned concrete but can be used to protect reinforcing bars on post-tensioned elements if extreme care is taken to prevent damage to metal ducting and tendons.

On bridges in New Zealand its application is probably limited because carbonation is usually only significant in concrete that is not exposed to wetting. In this situation corrosion is slow and unlikely to necessitate repair. Carbonation-related corrosion attributed to localised low cover or poorly consolidated concrete can be effectively repaired by patch repair as described in Section 3.4.1.1. Electrochemical realkalisation is an option however where the damage is widespread. An example of where it might be appropriate is on a bridge superstructure with low clearance over a waterway, where the relative humidity could be high enough for corrosion rates to be significant.

3.4.1.3 Realkalisation by Diffusion

Realkalisation by diffusion (principle 7.4) involves applying an alkaline concrete, mortar or solution to the concrete surface so that alkalis can diffuse into the concrete. BRE (2000) recommends that this process should not be used where the average carbonation depth exceeds the depth of cover or 10 mm, whichever is less. Corrosion activity could be accelerated if moisture from the treatment reaches the steel before sufficient alkalis are available to passivate it. The process will take longer than electrochemical realkalisation.

Diffusion can also be used to redistribute alkalis already in the concrete by saturating then sealing it. This process takes longer than the application of an alkaline solution and the same limits on carbonation depth would apply.

Neither of these processes is recommended because of the difficulty in ensuring effective diffusion of alkalis.

3.4.1.4 Electrochemical Chloride Extraction

Electrochemical chloride extraction (principle 7.5) was described in detail by Freitag & Bruce (2002). Cracked and spalled cover concrete is replaced, then the reinforcement is connected to a power supply, and a temporary anode within an electrolyte applied to the concrete surface. Hydroxide ions are generated at the reinforcement, increasing the pore solution alkalinity. At the same time, chloride ions are repelled from the negatively charged reinforcement and attracted to the anode, where they are collected in the electrolyte and removed from the structure. The steel surface will repassivate if enough chlorides are removed and sufficient hydroxide ions are generated at the steel surface. It is not possible to remove all chlorides. The process takes up to two months after initial patch repair.

Even if the surface is protected to prevent further ingress of chlorides from external sources, redistribution of the remaining chlorides by diffusion within the concrete could eventually allow corrosion to recommence. Broomfield (1997) considers that the treatment could remain effective for 10-20 years if further ingress of chlorides is prevented. Repeat treatments can extend this period although they will be less effective than the initial one.

Chloride extraction is suitable where contamination is limited to the cover concrete, where it is not practical or economic to remove all chloride-contaminated concrete, where recontamination can be prevented or would take longer than the intended life of the structure or of traditional patch repairs, or where the concrete is relatively dry. It cannot be used on pre-tensioned concrete but can be used to protect reinforcing bars on post-tensioned elements if extreme care is taken to prevent damage to metal ducting and tendons. Bridges in the NZS 3101:1995 coastal exposure zone (B1/B2) could be appropriate candidates. It is not suitable for components such as piles or piers that are partly immersed in sea water, or for deeply contaminated concrete.

3.4.2 Increasing Resistivity

The rate of corrosion is largely controlled by the electrical resistance of the cover concrete. This is determined by its moisture content and its chemical composition. Treatments that are based on increasing resistivity (principle 8) seek to make the concrete more resistant to the ingress of moisture (thereby also increasing its resistance to chloride ion ingress). They include coatings, hydrophobic surface treatments, or shelters applied to protect the entire element or structure from rain, run-off, splash, spray or atmospheric moisture. (Local application of these systems will induce incipient anodes, as will patch repair materials that have a higher resistivity than the surrounding concrete because of their composition or water resistance.) These methods will remain effective for the life of the structure providing they are maintained.

Alternatively they can be used without maintenance, in which case they will increase the length of a repair cycle by the life of the treatment. They may be applied to undamaged concrete as a preventive measure, or in conjunction with patch repairs. They will however prevent the future use of electrochemical treatments that use anodes applied to the concrete surface should they be needed.

Water-resistant treatments must not trap moisture in the concrete or they will increase the risk of corrosion. They need to be permeable to moisture vapour to allow the evaporation of water present at the time of application, water that subsequently penetrates the concrete from the ground, water that penetrates the concrete due to inadequate detailing to control run-off, and water that penetrates the concrete because of failed services or joints. They will restrict the ingress of chloride ions, but evaporation of moisture in the concrete might be too slow to prevent corrosion in concrete that is already chloride-contaminated when the treatment is applied.

Buried or immersed concrete can only be protected by specialist waterproofing systems that are designed to withstand hydrostatic pressure and damage by backfill.

None of these methods will be effective unless the entire element is protected from the ingress of moisture from any source.

As well as providing a moisture barrier, coatings will resist the penetration of air and therefore carbon dioxide. Coatings must bond well to the substrate and be sufficiently elastomeric to withstand the anticipated movement of the concrete and subsequent cracking. Live cracks may need to be 'bandaged' before being coated, and attention should be paid to the treatment of joints. The coating must be correctly applied and maintained to prevent defects that would allow the ingress of air and moisture.

Hydrophobic surface treatments impregnate the concrete surface, lining the pores with water-repellent molecules that create a barrier to liquid water and aqueous solutions. They will not be effective if the concrete cracks or existing cracks widen after treatment. Unless they have a significant pore-blocking effect they could increase the rate of carbonation because they allow the concrete to dry and do not prevent the ingress of carbon dioxide.

Ventilated claddings can shelter the concrete from rain or splash, but should not be used if water could penetrate the concrete from another or unpredicted source.

3.4.3 Cathodic Control

Cathodic control (principle 9) involves controlling the rate of reaction at the cathode.

3.4.3.1 Saturating the Concrete

Corrosion is slow in saturated concrete because the water in the concrete pores physically blocks the supply of oxygen to the cathode. However this approach (principle 9.1) will not work unless the saturation is complete and permanent. It can be particularly risky if the concrete is chloride-contaminated unless it is permanently saturated. As a consequence it is not recommended.

3.4.3.2 Cathodic Inhibitors

Inhibitors can be introduced into the concrete as admixtures in the fresh repair material, or can be applied to the finished surface.

Cathodic inhibitors (principle 9.2) are chemicals that inhibit the cathodic reaction by forming a thin layer on the steel surface that acts as a barrier to prevent oxygen reaching the steel surface. They are less widely used than anodic inhibitors (section 3.4.5.2) or ambiodic inhibitors, which control both the cathodic and the anodic reactions by adsorption over the entire surface of the steel. For further discussion on corrosion inhibitors and their effectiveness see Section 3.4.5.2.

3.4.4 Cathodic Protection (CP)

Cathodic protection (principle 10) is described in detail by Freitag & Bruce (2002). The reinforcement is connected to an anode attached to the concrete surface or inserted into a cavity cut in the concrete, and polarised to overcome the potential difference driving corrosion (or to prevent such potentials from developing, in which case the process is called “cathodic prevention”). Impressed current cathodic protection (ICCP; principle 10.1) involves applying a current from an external power supply to polarise the reinforcement. Galvanic CP (principle 10.2), also called sacrificial CP, uses an anode made from a metal of higher potential energy than the reinforcement (see Section 2.1 of this report) to generate a current. Both processes generate hydroxyl ions at the reinforcement, providing further protection.

CP remains effective for as long as the equipment is installed and maintained, which can be for the life of the structure. It requires constant monitoring to optimise performance, regular maintenance and replacement of individual components that have a limited life. It is the only treatment for reinforcement corrosion that offers a permanent solution.

CP can be used to protect reinforcing bars on both pre- and post-tensioned concrete if extreme care is taken to prevent damage to metal ducting, strand and tendons. With appropriate care it can also be used to protect prestressing strand (NACE 2002). It is suitable for concrete in all exposure conditions. Spalled and cracked concrete needs to be replaced, but carbonated or chloride-contaminated concrete that is otherwise sound does not need to be removed.

Galvanic CP principles have also been adopted in products designed to prevent incipient anodes developing on rebar adjacent to patch repairs.

3.4.5 Anodic Control

Anodic control (principle 11) involves restricting the electrochemical reactions at the anode.

3.4.5.1 Reinforcement Coatings

Coatings containing zinc or other inhibitors can be applied to the reinforcement to inhibit cathodic reactions at the steel (principle 11.1). Their inhibiting capacity is limited by the amount of reactive inhibitor present, which could be insufficient to protect the steel for the life of the structure if, for example, other components in the patch repair system fail and are not replaced.

Alternatively, coatings that form a physical barrier to water, chlorides and air can be applied (principle 11.2). Such coatings electrically isolate the steel from the surrounding concrete. Corrosion activity can be intense at coating defects that expose

the steel, so extreme care must be taken to ensure complete coverage and bonding and to avoid subsequent damage to the coating. Some barrier coatings include inhibitors to protect the reinforcement where the coating is damaged, although again the amount of inhibitor at the damaged site determines its effectiveness. Barrier coatings could preclude the subsequent application of electrochemical treatments.

Reinforcement coatings are only essential where they form part of a proprietary system (Sections 4.2, 4.8) or for specific applications, such as protecting reinforcement exposed to sea water or sea spray before the cover concrete can be reinstated.

3.4.5.2 Anodic Inhibitors

Anodic inhibitors are chemicals that inhibit the anodic reaction (principle 11.3) by providing an insoluble protective film or an adsorbed layer on the steel surface.

Ambiodic inhibitors, which inhibit both anodic and cathodic reactions, are also available. These are large organic molecules that adsorb onto the steel surface to form a hydrophobic film. 'Vapour phase' or 'volatile' inhibitors are a type of adsorption inhibitor that has a high vapour pressure and can penetrate as a vapour (see discussion below about migrating corrosion inhibitors).

For anodic, cathodic or ambiodic inhibitors to be successful the active chemical must be present in sufficient quantity at the reinforcement. Pitting can develop if insufficient anodic inhibitor is present.

Migrating corrosion inhibitors are applied as admixtures in patch repair materials or to the concrete surface rather than being applied directly to the steel. Surface-applied inhibitors penetrate the concrete by capillary absorption as liquids, then by ionic diffusion through saturated pores. Vapour phase inhibitors penetrate to some degree by gaseous diffusion through empty pores although their dominant transport mechanism is thought to be diffusion in solution (Tritthart 2003). Inhibitors applied in the patch repair are transported primarily by diffusion through saturated pores. The degree of penetration depends on the quality and moisture content of the concrete. In good quality concrete with normal cover depths, the depth or uniformity of penetration may be insufficient. Compared to application directly onto the finished concrete surface, penetration of a surface-applied inhibitor is improved if the inhibitor is applied to holes or grooves cut into the concrete surface, or onto a substrate that has been prepared for patch repair. The greater the cover depth the more inhibitor needs to be applied. Vapour phase inhibitors could also evaporate out of the concrete. Some salt-forming (anodic) inhibitors have been observed to precipitate when in contact with cement and therefore are unable to penetrate to the steel (Tritthart 2003).

Concerns about inhibitors include the following (MacDonald & Broomfield (2003):

- Do they penetrate the concrete sufficiently and spread uniformly and effectively along the rebar network?
- Can they suppress corrosion and for how long?

- To what level of level of alkalinity, chloride contamination and corrosion rate are they effective, and do they remain effective?
- Can these features be measured?

Other potential issues with inhibitors could include odour with vapour phase inhibitors; staining, efflorescence or problems with coating the concrete after treatment; effect of admixture inhibitors on setting properties of patch repairs.

MacDonald & Broomfield (2003) report that knowledge about the effectiveness of corrosion inhibitors in concrete is limited. Nevertheless, on the basis of the above features, the inhibitors available, and the ease with which they can be applied to accessible surfaces, they consider them suitable when:

- The concrete has carbonated or contains less than 1% chloride by weight of cement;
- Cover depth is less than 20 mm;
- The cover concrete is permeable (as evidenced by carbonation or corrosion damage in less than 20 years from construction);
- A barrier coating is applied after application;
- Corrosion after application is monitored, e.g. by installing corrosion monitoring systems.

3.4.6 Standard Specifications

Treatments complying with standard specifications and tested to standard test methods offer some protection to the purchaser.

CP is covered by AS 2832.5-2002 (SA 2002), NACE (2000) and BS/EN 12696-2000 (BSI 2000).

The EN 1504 series of standard specifications (still mostly in draft status, and in the UK referred to as BS/EN 1504) covers products and systems for the protection and repair of concrete structures. It includes sections on definitions (part 1, which has been published), surface treatments (part 2), structural and non-structural repair (part 3), structural bonding (part 4), concrete injection (part 5), grouting of anchors or external voids (part 6), reinforcement corrosion prevention (part 7), quality control and conformity (part 8), general principles (part 9), site application and quality control (part 10). When adopted, the standard specifications will be used in all European countries that belong to CEN, and products supplied in New Zealand from European manufacturers will then be likely to comply with them. Some standard test methods have already been published to support these specifications, and others are in the development/approval process.

ACRA (1996) is an Australia/New Zealand guide to concrete repair and protection. It is not a standard but methods complying with its recommendations should be sound.

ACI Concrete Manual 546R-56 (ACI 1996) is a general guide to repair of concrete that includes information on repair of corrosion damage.

The Corrosion Prevention Association and Concrete Repair Association, both in the UK, offer guidelines on electrochemical and patch repair techniques through their websites *www.concreterepair.org.uk* and *www.corrosionprevention.org.uk*.

Suppliers of proprietary products and systems will usually provide model clauses that can be referenced in contract specifications. The conditions of the manufacturer's warranty usually require their products or systems to be used in accordance with the company's model specification clauses.

3.5 Inspection and Maintenance Requirements

It is beyond the scope of this document to detail inspection and maintenance requirements of concrete structures after remedial treatment. However to implement a strategy for managing reinforcement corrosion on a structure, the following steps are needed:

- Instigate a regular inspection and monitoring programme to give early warning of imminent deterioration, and record observations and results systematically;
- Record the details of all repairs or preventive treatments carried out (date, method, materials, contractors);
- Develop a maintenance schedule for repaired and non-repaired parts of the structure, setting out actions required, personnel, responsibilities, timing, locations, methods;
- Develop a system and train personnel so that data from monitoring systems can be interpreted and managed;
- Co-ordinate inspections and monitoring so that observations by different personnel can be combined. Ideally records should be stored (or able to be sorted by) structure so that the complete history of an individual structure can be accessed easily.

Budgeting for the above processes will allow appropriate treatment to be developed, applied and maintained throughout the life of the structure.

Special provision needs to be made to ensure that CP structures are monitored and maintained correctly, and that monitoring outputs can be related to observations made during routine inspections.

Staff who carry out routine inspections should be instructed to examine and report the condition of repaired or treated sites as part of their normal procedure.

3.6 Summary

Management of reinforcement corrosion on any structure involves several stages of decision making:

- Assess condition, identify cause and extent of defects;
- Select options for managing deterioration (e.g. do nothing, repair, demolish, etc.);

3. *Repair Strategies*

- Identify treatment principle (e.g. preserving/restoring passivity, cathodic control, increasing resistivity);
- Select method of treatment (e.g. CP, patch repair);
- Select materials and specifications (e.g. specific proprietary products/systems);
- Identify future inspection and maintenance requirements.

Each of these stages requires consideration of features particular to the individual structure's condition, use, location and site conditions. In practice, immediate costs rather than strategic aspects may drive one or more of these decisions. Nevertheless, whatever the approach taken, an understanding of the principles behind various repair methods and products, and the benefits that each can deliver, will help to ensure that the treatment selected is effective for deterioration on a specific structure.

4. Patch Repair Systems

The selection of a repair strategy for an individual structure, and the principles by which repair materials prevent corrosion are described in Section 3 of this report. This section describes features of the different types of materials that comprise a patch repair system.

Most patch repairs will comprise several components: bonding agents, reinforcement coatings, patch materials and surface treatments. Each may be designed to address a separate treatment principle as defined in Table 3.2. Specialist technical advice should be sought from suppliers of proprietary materials or from independent experts to ensure that the repair materials selected are appropriate for the treatment principles and methods adopted.

Major suppliers of proprietary repair products in New Zealand at June 2003 are listed in Section 4.8 of this report.

4.1 Proprietary Materials

Pre-bagged proprietary products manufactured under an internationally recognised quality assurance scheme, or to meet recognised standard specifications such as those of BS/EN or ASTM, are recommended over site-batched materials because they are purpose-designed to meet specific requirements (described in the following sections). They are convenient, and consistent properties can be achieved between batches and applications. Many suppliers will offer warranties if the products are selected and applied according to their recommendations. Products that come with such warranties are recommended for large-scale repairs.

A wide variety of products are formulated to suit different application conditions and methods, service conditions and repair sizes. Requirements for application and long-term performance must be discussed with the supplier to ensure that appropriate products are selected. Independent expert advice should be sought when trialling a new or unfamiliar approach because it may compromise subsequent options should future treatment be needed. Independent expert advice should also be sought when the risk of repair failure must be minimised.

Most proprietary products are part of a system, the components of which are designed to be used together. Alternative materials should never be substituted without the approval of the supplier, otherwise the supplier's warranty will be invalidated. In addition to the patch repair mortar or concrete itself, systems may include reinforcement coatings, bonding agents, corrosion inhibitors, internal sacrificial anodes and surface treatments. These are described in Sections 4.2 to 4.7, and proprietary products in Section 4.8.

Suppliers may have approved or licensed applicators who have been trained in the appropriate application techniques. These contractors should be used wherever possible, because the supplier's warranty could be invalid if other contractors are used.

4.2 Reinforcement Coatings

Coatings applied directly to the reinforcement are designed to provide:

- a coating that is anodic to the steel and will corrode preferentially (principle 11.1), e.g. coatings containing zinc, or indeed galvanised rebar; or
- a physical barrier to prevent moisture, chlorides and air reaching the steel (principle 11.2), e.g. epoxy coatings; or
- an alkaline environment equivalent to that provided by the concrete substrate or patch repair (principle 7.2), e.g. cement-based coatings.

Shortcomings of the principles behind the first two types of coating are mentioned in Section 3.4.5.1. However reinforcement coatings are just one component of the patch repair system, and are not designed to be the sole means of protecting the reinforcement. Their effectiveness will depend on the performance of the entire system.

Reinforcement must be properly cleaned to ensure that the coating bonds to the steel, and that no contaminants remain on the steel surface.

Little information is available about the effect of reinforcement coatings on the bond between round reinforcing bars and the surrounding patch repair. Rebar coatings will reduce the bond between deformed bars and the surrounding concrete because they effectively reduce the profile of the deformations. ACI 546 warns that bars which have lost their original deformations as a result of corrosion will have less bond with most repair materials, and that their bond will be further reduced by coating. It recommends that rebar coatings be no thicker than 0.3 mm to avoid loss of bond development at the deformations.

Physical barrier coatings may prevent the subsequent use of electrochemical treatments.

4.3 Bonding Agents

Bonding agents are applied to the cleaned concrete substrate to ensure intimate contact between repair material and substrate, and to satisfy the absorption of the concrete substrate so that it does not suck so much water out of the patch material that it cannot hydrate. They are not applied between successive layers of patch repair materials. Bonding agents may be based on epoxy, polymer emulsion (latex), or cement. Bonding agents are applied at a specified time interval before the patch material, the interval varying with the bonding agent.

Epoxy bond coats impede the movement of moisture and diffusion of ions within the concrete near the repaired area, which could contribute to the development of incipient anodes. However, significant corrosion is unlikely because the flow of ions between anode and cathode is prevented, except at voids in the coating.

Polymer-emulsion bond coats may be redispersible or non-redispersable. Redispersable polymers (e.g. vinyl acetates such as PVA) can be applied several days before the patch repair material but cannot be used in structural applications or where the concrete is exposed to water or high humidity. Non-redispersable polymers (e.g. acrylics, styrene butadienes) give a stronger and more durable bond but must be applied immediately before the patch repair material because they will act as bond breakers if allowed to skin or dry.

Cement-based bond coats are slurries of cement and water and possibly sand and a polymer emulsion. Patch repair material is generally applied before the bond coat hardens, usually within a few hours.

4.4 Patch Repair Materials

Patch repairs protect the reinforcement by restoring passivity (principle 7.2). To maintain this protection they must remain bonded to the substrate without cracking, and without inducing cracking or other deterioration in the original concrete substrate. Properties of the patch material that determine its long-term performance are:

- *Adhesion*: the repair material must bond well to the substrate, preferably whether it is wet or dry;
- *Coefficient of thermal expansion*: the repair material must have a similar coefficient of thermal expansion to the substrate, or changes in temperature will induce failure at the interface or within the weaker material;
- *Shrinkage*: the repair material must have minimal shrinkage or it will induce failure at the interface or within the weaker material. Repair materials usually have a low water to cement ratio, and include shrinkage compensating components;
- *Modulus of elasticity*: the repair must have similar deformation characteristics to the substrate, otherwise loading parallel to the bond line will induce failure at the interface or within the weaker material;
- *Strength*: the repair material must be strong enough to withstand anticipated loads;
- *Electrical properties*: the repair material should have similar electrical properties to those of the remaining cover concrete to minimise incipient anode effects;
- *Permeability*: repair materials of different permeability than the substrate or remaining cover concrete can induce differences in electrical properties as described above. Repair materials that are less permeable than the substrate can trap moisture or moisture vapour at the interface, which will weaken the bond;
- *Chemical properties*: repair materials must be sufficiently alkaline to passivate the reinforcing steel. Repair materials that are designed to form an impermeable barrier rather than provide an alkaline environment will not protect the steel if damaged.

Epoxy patch repair materials have been widely used in the past because of their convenience, short curing time, good initial bonding characteristics, high strength, impermeability and resistance to chemical and physical attack. However they are not considered suitable for repairing damage caused by reinforcement corrosion because they do not meet the above requirements of thermal expansion, elastic modulus, electrical and chemical properties. Figure 4.1 shows epoxy patch repairs that have failed to prevent corrosion continuing under the patch. Debonding of epoxy patches caused by differences in dimensional stability between the patch and concrete substrate is also common. Epoxies are therefore best suited to repair of surface damage, edge damage at joints and surface defects, particularly where rapid setting is required, rather than to thicker repairs or to repair damage caused by reinforcement corrosion.

Other polymer materials that have been used to repair concrete include polyester, furan, vinylester, methyl methacrylate and styrene. They have the same drawbacks, in different degrees, as epoxies. Polymer repair materials of any type are not considered in the following discussion.



Figure 4.1 Epoxy patch repairs that have failed to prevent ongoing corrosion.

Portland cement-based repair materials are recommended for repairing corrosion damage because they are more compatible with the original substrate than other materials. They can be formulated with a range of properties to suit different applications. They can be applied by trowel, spray or by casting.

Options include:

- grouts, plasters, mortars or concretes depending on the depth, volume and accessibility of repair;

- different aggregate sizes to allow for different repair thicknesses and reinforcement spacings (maximum aggregate size should not exceed one third of the patch thickness);
- self-compacting concrete to allow sections to be cast in enclosed formwork;
- reduced density for high-build application;
- fibre reinforcement for resistance to cracking caused by plastic and/or drying shrinkage;
- shrinkage compensation for resistance to cracking and bond failure;
- different strengths for different load requirements;
- different setting times and strength gain characteristics for different application temperatures and retraffic requirements;
- supplementary cementitious materials for reduced permeability to moisture, increased chemical resistance and possible reduced heat evolution;
- polymer modifiers to improve workability, bond, and tensile strength, reduce permeability and the need for wet curing;
- waterproofing admixtures/additives.

Polymer-modified repair materials are particularly suited to relatively thin layers because they have good adhesion to wet and dry substrates, do not require the extended moist curing of other cementitious repair materials, and may be more resistant to shrinkage cracking because they retain moisture and have higher flexural/tensile strengths than unmodified materials.

They could have higher electrical resistivity than the surrounding concrete, which might increase the risk of incipient anode effects, although the reduced ion flow that results will minimise the risk of significant corrosion.

They can have lower elastic modulus than the concrete substrate so may not be suitable for repairs to members that will be loaded parallel to the patch-substrate interface, e.g. vertical faces of piers. To account for this, one supplier has a range of products modified with silica fume for structural (load-bearing) repairs, and a range of polymer-modified products for smaller or non-structural repairs, for example where significant structural damage has not yet developed.

PVA-based materials are not suitable for surfaces that are exposed to moisture. Acrylic, styrene-acrylic and styrene-butadiene based materials are water resistant. Styrene-butadiene based materials may discolour if exposed to UV light.

Concrete from a ready-mix supplier is suitable for recasting entire sections. When used as a patch repair it must be designed to minimise shrinkage and bleeding, which could otherwise cause bond failure at the interface between the repair and the original concrete. Although such a repair is likely to be mechanically bonded to the substrate by dowelling, loss of bond between the patch material and the original concrete could provide a passage for moisture and air to reach the reinforcement. As for other patch materials, the ultimate hardened properties of the concrete must be similar to those of the original concrete. Ready-mix concrete repairs may be more

successful over waterways than elsewhere because the high humidity minimises the risk of cracking or debonding related to shrinkage.

4.5 Corrosion Inhibitors

As discussed in Sections 3.4.3.2 and 3.4.5.2, corrosion inhibitors may be added to the patch material as an admixture, or applied to the concrete surface.

The most common anodic inhibitor is calcium nitrite. Others that have been tried include chromates, benzoates, alkali phosphates, silicates and carbonates.

Cathodic inhibitors are generally metal ions, including zinc, antimony, magnesium and nickel.

Ambiodic inhibitors include monofluorophosphate, and various amino alcohols and esters that penetrate as vapours.

Reports on the effectiveness of corrosion inhibitors used in repairs are conflicting. Part of the difficulty is in detecting their presence on the steel surface. Another problem is that surface treatments are often applied after corrosion inhibitors, and it can be difficult to monitor corrosion activity from a treated surface unless a monitoring system is embedded in the repair.

4.6 Internal Sacrificial Anodes

Sacrificial zinc anodes designed to prevent incipient anodes from forming were recently introduced by one supplier of patch repair materials. These anodes are supplied embedded in a proprietary mortar that prevents the zinc being passivated by the alkaline repair material. They are fitted with tie wires to electrically connect the zinc to the reinforcement, and are fastened to the cleaned reinforcing bar around the periphery of the repair before reinstatement of the cover concrete.

They can also be used to prevent incipient anodes forming within a repair if it is not possible to prepare all reinforcement surfaces within the repaired area to the same level of cleanliness.

As with other CP systems, the surface area of the rebar determines the number and spacing of the anodes, and the supplier's guidance on anode spacings must be followed for the system to be effective.

If monitoring anodes are also installed, the performance of the sacrificial anodes can be measured after completion of the repair to check that they were installed correctly, and monitoring can be continued for the life of the anode if required. These anodes are designed for use with a particular proprietary system and other patch repair materials must not be substituted.

The system is designed to bridge the gap between electrochemical techniques and patch repair, by enhancing the performance of patch repair at a smaller installation cost than a CP system. It is particularly suited to chloride-contaminated concrete. It

may be less effective for carbonated concrete, which has a higher electrical resistivity and therefore is less prone to incipient anode formation. Estimated life of the anodes is approximately 5-10 years depending on conditions, after which they can be replaced if desired. However it may be more cost-effective to use them just to extend the time to next repair.

The system was developed in the mid-1990s and introduced in New Zealand in 2000. As at May 2003 it had been installed on between five and ten structures in New Zealand, including wharves, buildings and bridges, but it is too soon to evaluate its effectiveness.

4.7 Surface Treatments

Surface treatments may be applied to the original surface and/or over a patch repair to prevent the ingress of moisture or other aggressive agents. They include penetrating sealers and surface coatings.

The durability and ease of maintenance varies between products, and these features should be considered when selecting a surface treatment.

Surface treatments will preclude the subsequent application of realkalisation or desalination.

4.7.1 Penetrating Sealers

Penetrating sealers are water-repellent materials that act within the pore system of the concrete. Penetration depth will differ with the size of the penetrating molecules, and with the pore size/continuity and moisture content of the concrete substrate. They are not water vapour barriers and so do not trap moisture in the concrete, but neither are they effective barriers against the ingress of carbon dioxide or oxygen which may penetrate more easily into the dry, treated concrete. Penetration depth is not critical to long-term performance unless the surface will be abraded or eroded during service.

Penetrating sealers include:

- *Stearates, silanes, siloxanes and silicones*: these line and block the pores with hydrophobic molecules. They can be used as stand-alone products but are also used as primers for surface coatings.
- *Silicate solutions*: these react with calcium hydroxide formed during cement hydration to produce calcium silicate hydrates. Various claims are made about the mode of action of the different products, but basically the reaction products block the pores to restrict the passage of air and moisture through the concrete. Consumption of the calcium hydroxide could increase the carbonation rate should the concrete dry sufficiently. ASTM (2002) has acknowledged the difficulties of evaluating the effects of silicate treatments on concrete, and has established a task group to develop standards to overcome confusion and disagreement that has arisen between producers, specifiers and users in the US.

Surface penetrants will line crack surfaces present at the time of application but will not seal active cracks.

4.7.2 Surface Coatings

Surface coatings provide a continuous barrier on the concrete surface to protect it from the ingress of water, aggressive solutions or air. Coating thicknesses can range from less than 1 mm to several mm.

Coating materials include:

- *Epoxy resins*: good chemical resistance, hard wearing but brittle and have poor UV resistance.
- *Polyurethanes*: good chemical and weathering resistance, flexibility and toughness.
- *Polyester, vinyl ester/acrylate*: excellent chemical and temperature resistance, cures at low temperatures.
- *Acrylics*: decorative, good weathering resistance and crack bridging properties, water-vapour permeable, resistant to ingress of carbon dioxide and chloride solutions.
- *Vinyls, synthetic elastomers, chlorinated rubber*: general barrier coatings, good weathering resistance, solvent sensitive.
- *Bitumen*: low cost waterproofing.
- *Cementitious*: barriers against carbon dioxide and moisture but poor acid resistance. Acid resistance and impermeability may be better in products containing fine silica. Flexibility can be improved by polymer modification. Good abrasion and impact resistance.
- *Inorganic silicate paints*: used largely to improve visibility rather than to protect the concrete. Preferred for coating in tunnels because it is not flammable.

A commonly recommended surface treatment to complete a repair is a silane-siloxane penetrating primer overcoated with a clear or pigmented acrylic membrane coating. The acrylic coating protects the silane-siloxane and provides a barrier to carbon dioxide while allowing evaporation of moisture in the concrete. Acrylic coatings are available as water- or solvent-based products. The solvent-based products tend to be more resistant to moisture and alkalis but are less flexible. Neither is suitable for surfaces that are subject to long-term water ponding.

Coating products that rely on the evaporation of solvent or water are more permeable to liquids and gas than '100% solids' products.

Surface coatings can seal fine, non-moving cracks but specially designed elastomeric coatings are necessary to bridge active cracks. Before coating, active cracks need to be filled with flexible sealant and non-moving cracks need to be filled with a rigid sealant if the cracks are wider than the crack-bridging capacity of the selected coating. The coating should be flexible enough to withstand anticipated concrete movement caused by fluctuations in temperature or moisture content.

Many coatings based on organic polymers are pigmented, and can be used to conceal patch repairs and other surface blemishes as well as to impart an attractive appearance.

Uneven surfaces can be prepared for coating by applying a cementitious fairing coat up to about 4 mm thick. This will also provide some protection against moisture ingress but is not designed to be a protective barrier coating itself.

Coating performance can differ significantly between individual products of similar type. Performance also depends on the quality of application, which determines adhesion and coverage.

4.8 Proprietary Products

In August 2002, suppliers of patch repair materials listed in the Concrete Directory 2000 (Cement and Concrete Association of New Zealand (CCANZ 2000)) were asked to provide information on products that they would recommend for repairing concrete bridges damaged by reinforcement corrosion and, if possible, to identify bridges on which they had been used. This was to identify products that were likely to be used, rather than to compile a list of products available.

Portland cement-based patch repair systems specifically for concrete damaged by corroding reinforcement were offered by Building Chemical Supplies Ltd, Demden Ltd, Fosroc Ltd, Fraser Brown and Stratmore Ltd, MBT (NZ) Ltd, Rockbond Special Concrete Products, and Sika (NZ) Ltd. These systems consist of reinforcement coatings, bond coats, patching mortars and concretes and various surface treatments. Some products contained reactive fillers to reduce their permeability, some contained acrylic emulsions, and some were shrinkage compensated.

Building Chemical Supplies Ltd, Fosroc Ltd, Fraser Brown and Stratmore Ltd, MBT (NZ) Ltd, Rockbond Special Concrete Products, and Sika (NZ) Ltd offered a range of patch products and systems, with the following options available:

- low density for high-build applications;
- synthetic fibre-reinforced for thin applications;
- polymer-modified mortars and concretes;
- concrete for large scale repairs, mortar for small scale repairs (defined by one supplier as less than 0.15 m² x 75 mm deep);
- pumpable grout and concrete;
- various compressive strengths for different load requirements;
- free-flowing concrete for areas of congested steel, difficult access;
- wet-spray products for small repairs, dry-spray products for large repairs;
- normal- or rapid-setting;
- electrically conductive overlays for cathodic protection;
- mortars for underwater repairs;

4. *Patch Repair Systems*

- zinc-rich epoxy or cement-based reinforcement coatings;
- sacrificial anodes for preventing incipient anode formation;
- acrylic emulsion or solution curing compound;
- surface-applied migrating corrosion inhibitors.

Their coating systems included:

- biocide pre-treatments;
- silane siloxane primers;
- cementitious waterproof renders;
- cementitious mortars for filling, levelling and reprofiling;
- solvent and water-based acrylics;
- elastomeric acrylics for crack bridging;
- inorganic silicate paints for visibility/aesthetics.

Demden Ltd's system was based on proprietary waterproofing and repair products consisting of mixtures of Portland cement, hydrated lime and fine silica to which specified amounts of sand, cement, water and acrylic emulsion are added to make a bond coat/reinforcement coating, a repair mortar or concrete and a slurry topcoat. These products densify the pore structure of the hydrated Portland cement by reacting with cement hydration products to produce more calcium silicate hydrates. Fraser Brown and Stratmore Ltd offered a similar product.

Markham Distributing offered a system based on a silicate-based solution that is applied to the surface of the concrete or patch repair and absorbed into the pore system, where it reacts with calcium hydroxide to form a silicate hydro-gel of unknown composition that densifies the pore structure. Markham also claims that the product contains a corrosion inhibitor, blocks the ingress of oxygen, and removes chlorides from the concrete or binds them so that they are unable to promote steel corrosion. The Australian agent for the treatment, Allcrete Industries, recommends that standard cementitious repair mortars be used when this silicate product is to be applied: polymer-modified mortars will hinder penetration of the treatment to the extent that it must be applied to the prepared substrate rather than to the surface of the repair. Markham reports recent experience with two polymer-modified products that suggests that this effect depends on the specific polymer-modified products used. A silicate-based surface treatment is applied immediately after the silicate solution to complete the treatment.

Neither Cemix Products Ltd nor Nuplex Industries offered systems specifically for repair of corrosion-damaged concrete. Nuplex offered epoxy-based patching mortars, coatings and sealers. Cemix offered cement-based products with acrylic or styrene butadiene emulsions as gauging fluids, which could be suitable.

5. Present Approaches to Repair of Reinforcement Corrosion

Twelve bridge consultants representing eight Transit New Zealand regions and four local authorities were interviewed to find out about their management of reinforcement corrosion on concrete bridges. They were asked about their approach to concrete repair, their use of coating systems, how they select specific repair systems and products, how they monitored repair systems, problems encountered during the repair process, repair products specified, performance of products used to date, and the potential use of a national database of bridges that had undergone electrochemical treatments. Their responses are summarised below.

5.1 Maintenance and Repair Process

Bridge repair and maintenance is generally tailored to meet the client's cost and performance expectations. Work carried out is often a balance between best practice, engineering judgement of the risk and likely outcome, and the available budget. Repair technique selection is therefore typically governed by the initial costs as opposed to life cycle or whole-of-life costs.

Since electrochemical techniques are seen as being expensive, patch repair techniques are almost always adopted by default because of their relatively low initial cost. Conventional patch repair also represents accepted technology that has a known or expected performance and life cycle. Most practitioners, owners and contractors have some level of experience with patch repair, and practitioners are comfortable specifying these systems.

The need for rehabilitation is generally prompted from bridge inspections that, for structures operated by Transit New Zealand, are carried out two-yearly. Many territorial authorities and bridge consultants operate similar inspection regimes that may or may not be rigidly adhered to depending on available funds. For those structures included in ongoing inspection programmes, the resident consultant builds up an historical understanding of the structure and its performance. The deterioration can thus be managed systematically by intervention at an appropriate time. Intervention is generally durability enhancement rather than repair to overcome structural deficiencies.

Where damage is evident and the cause is obvious, for example impact or obvious lack of cover concrete, then patch repair is generally specified. Standard patch repair procedures are employed and often the contractor will select a suitable repair system. Used in this context, patch repair effectively represents reactive maintenance. Bridge owners generally appear to be reluctant to carry out any significant proactive maintenance.

Where the cause of deterioration is not obvious, or where elevated chloride ion levels are suspected, then further investigation is necessary. Investigation may range from a more detailed visual inspection to a full condition assessment involving the determination of concrete cover depths, depth of carbonation, chloride ion levels, and condition of reinforcement.

5.2 Use of Coating Systems

The approach to the application of coating systems ranges from practitioners who have never specified a coating to those who specify a coating for all reasonably sized repairs. In general, coatings would be recommended to enhance the life of repairs where extensive repairs have been carried out, particularly in severe marine environments, to reduce further chloride ingress or where access is difficult. Coating application as part of a repair was considered relatively expensive, and in many cases the extra cost was considered to be equivalent to a further cycle of patch repairs in the future. The general approach to coatings is to use simple, robust technology that essentially resists water penetration. Other coatings have been specifically applied for aesthetic and anti-graffiti reasons, and to mitigate the effects of alkali-silica reaction.

5.3 Selection of Specific Repair System

Transit New Zealand requires use of generic specifications in the tender process. However, practitioners will sometimes specify a preferred product or an acceptable equivalent, particularly where a specific property is required. For example a product that can cure at low temperature will be required for repairs that will be carried out in cold conditions. Specifications may also provide a list of acceptable products or suppliers from which a contractor must make their selection. Sometimes the contractor selects the product, for instance where minor repairs are carried out under a general maintenance and repair contract and the cause of the defect is obvious.

Some territorial authorities use an invited or pre-selected tender system to ensure that only those contractors with a good track record and adequate expertise will tender for the project. In one case a territorial authority paid the cost for two preferred contractors to price repairs on a number of bridges and then selected the lowest priced tender.

Where possible, practitioners generally seek to use recognised systems for which technical backup, approved applicators and contractors with a proven track record are available. Warranties on workmanship and materials are also often required and these are typically for 10 years.

The final selection of repair technique is generally based on the up-front cost and invariably patch repair is chosen. Under the Transit competitive pricing policy, selection of a tender for physical works is based on weighted attributes, which are typically 70% price, and 30% non-price attributes. Some consultants will base the final selection on track record but for some bridge owners it is likely that tender selection is made on price alone.

5.4 Monitoring of Patch Repairs

Patch repair performance is monitored indirectly by the inspection programmes in place for most of the New Zealand bridge stock. This is essentially monitoring by default and, if repairs are failing, this will be flagged. Specific monitoring of repairs on an individual structure is rare and would typically only occur where the repair cost was high (say in excess of \$500,000). An example is the Orowaiti Bridge in Westport, where extensive patch repair was carried out for the second time and Transit required that the performance of the repair be monitored.

5.5 Problems Encountered with Patch Repair Contracts

One of the main problems encountered with patch repair contracts is scheduling for payment. Payment can be based on one of several options including length of saw cut, volume of patch repair material used, area of patch repair as a function of depth or location of a repair if access is a problem. The scheduling system adopted should be robust, flexible to allow for variations, and equitable to all parties as often a fair schedule equates to a good job. Other problems typically encountered include:

- Poor preparation, e.g. incomplete removal of corrosion product from reinforcement and excavations being feather edged rather than sawcut;
- Poor application of repair products, e.g. inadequate compaction;
- Defining the extent of concrete excavation required. Often reinforcement has historical surface corrosion and a contractor may remove more concrete than necessary in an attempt to locate clean steel. Insufficient removal of concrete beyond corroded reinforcement is also a problem;
- Contractors wanting to use repair materials that are not acceptable to the engineer;
- The extent of repairs may require propping of beams, or limitations on the extent of concrete excavation allowed at any one time;
- Some contractors do not have staff with the appropriate skills to make sound engineering judgements;
- Some contractors may not have suitable knowledge or the skills to achieve an acceptable result;
- The level of supervision required in achieving an acceptable outcome.

Many of these problems may well be related to the acceptance of lowest price conforming tenders.

5.6 Systems Specified in the Past

Cementitious-based repair systems from Fosroc Ltd, Sika (NZ) Ltd and MBT (NZ) Ltd have been specified in the past, with specific systems sometimes specified for certain applications, particularly when coatings are involved. The use of a particular system depends to a degree on the local profile and marketing effort of the supplier, with one company sometimes dominating a local market. Epoxy mortars are also occasionally specified where reinforcement is not present, e.g. on kerbs. Fosroc Galvashield XP anodes are regularly specified by one practitioner.

Several pre-1980s repairs consisting of site mixed concretes exist. These concretes have typically been cast into over-boxed formwork, which both replaces damaged cover concrete and locally increases concrete cover. These repairs have generally been quite successful.

5.7 Performance of Patch Repairs to Date

According to the practitioners surveyed, where proprietary patch repair systems have been applied correctly then they have performed adequately. This experience spans up to 15 years. One of the major problems encountered is ongoing deterioration at the margins of repairs because of the formation of incipient anodes.

5.8 Database of Electrochemical Repairs

Practitioners were asked whether a database of completed electrochemical repairs on bridges would be useful to them and would encourage the use of electrochemical treatments.

In general the feeling was that this type of a database may be of general interest but would not be a particularly useful tool. The bridge repair industry in New Zealand is small enough that, if an electrochemical technique were applied to a bridge, it would be common knowledge very quickly. If an electrochemical technique were to be considered, the feeling was that specialist advice would be sought for that particular application. Concern was expressed about the mechanism of updating and maintaining the database.

Practitioners acknowledged that lack of knowledge was a barrier to the uptake of the technology, but also mentioned other well-known perceived barriers to electrochemical solutions such as cost, power supply and safety of remote monitoring equipment. They also pointed out that public safety issues (such as bridge width and road alignment) may override the potential use of electrochemical techniques, making replacement of a structure more cost effective in the long term.

5.9 Summary

Repair tends to be reactive rather than proactive, being prompted by observations during inspections and assessment of the risk associated with damage.

Patch repair is preferred over other methods because of its low initial cost and familiarity with the materials and techniques. Preferred products are sometimes specified. Proprietary patch repair systems are being used and are believed to be successful apart from incipient anode formation. Site-mixed concrete has also been used with similar success. Epoxy patching is used for small repairs not related to reinforcement corrosion. Coatings are used occasionally, for aesthetic purposes, to increase the life of repairs, to resist graffiti and to reduce the risk of alkali-silica reaction.

Immediate cost is still the main influence on the selection of repair methods, products and contractors. The main problems associated with repair contracts are related to scheduling for payment, and achievement of acceptable quality for minimum price.

A move to asset management practices that encourage long-term maintenance strategies for individual structures, rather than simply minimising up front costs, may help to improve the quality of repairs being achieved.

6. Case Studies of Long-term Performance of Treatments

This chapter presents case studies of treated structures to describe the long-term performance of patch repairs or electrochemical treatments. The intention was to examine structures treated five to ten years ago, a period long enough to establish whether a patch repair system could at least deliver a 10-year service life, and the likely costs involved after application of an electrochemical treatment. Treatments applied more than 10 years ago were considered to be irrelevant because the technologies used then have been superseded.

For some treatments there appeared to be no structures treated more than five years ago.

The case studies are based on site investigations of structures by the researchers and on discussions with owners of structures and with other consultants.

6.1 Long-term Performance of Patch Repairs

It is generally accepted that, if they are correctly applied, patch repairs will maintain a bridge in a satisfactory condition for about 10 years before more repairs are necessary. If not correctly applied, or if inappropriate systems are used, the repair life can be much shorter. One Australian bridge owner who was interviewed about use of CP (discussed in Section 6.2) assumed a life of 5 years for patch repairs in a marine environment and 10 years for patch repairs elsewhere.

Structures that had been repaired with different patch materials were reviewed to assess the performance of the materials. Structures were selected because suppliers of repair materials had identified them as ‘bridge applications’, or because the researchers had access to the original condition assessments. Performance was considered in terms of the cause and extent of the original defect and the presence of cracks or other corrosion-related damage in the repairs or in the original concrete.

The case studies (details of which are presented in Appendix B) suggest the following expectations of patch repair performance for the exposure zones defined by NZS 3101:1995.

- In the A2 zone, proprietary patch repairs will offer at least 10 years without further corrosion damage, as evidenced by the performance of repairs on the Fairfield Bridge. A repair cycle of 20 years is suggested as a basis for long-term costing.
- In the B1/B2 zones, proprietary patch repairs can offer up to 10 years without further corrosion damage depending on the type of repair and the standard of its execution, as shown by the repairs on the Kilbirnie Fire Station. However new corrosion damage within 10 years is more likely, as demonstrated by the Paremata, Orowaiti, and West Coast bridges. A repair cycle of 10-15 years is suggested as a basis for long-term costing.

- This study examined insufficient C zone concrete to establish the time to cracking after patch repair, but the above evidence and previous experience suggests that 5-10 years is a reasonable estimate for both time to cracking and for a repair cycle as a basis for long-term costing.

The actual length of the repair cycle on a given structure will depend on site conditions, quality of concrete and construction, and the level of damage that is considered acceptable. Surface coatings and treatments and sacrificial anodes incorporated into the repair can increase the length of the repair cycle.

6.2 Long-term Performance of Electrochemical Treatments

Although many papers have been published about the installation of electrochemical treatments, little information is available about their long-term performance. This could be because such information is commercially sensitive, or because it is perceived as not worthwhile to write about something that has become a routine operation. Papers that do report medium-term performance tend to report monitoring data and technical details of interest to operators and technical experts, rather than observations and maintenance details that might be of interest to the clients paying to operate and maintain the systems. To overcome this shortage of published material, representatives of three Australian bridge-owning authorities, and three Australian consultants/suppliers were asked about their experiences with CP systems: how they selected a treatment, the cost and effort involved in operating and maintaining it, and whether it was achieving the desired outcome. Their comments, not all related to long-term performance, are condensed below. Many of the CP systems installed or monitored that they described were less than 5 years old.

The long-term performance of chloride extraction and realkalisation was not evaluated because few structures have been treated sufficiently long ago for their owners to develop broad experience with these treatments. The long-term performance of CP systems in the UK and US were not reviewed because many of these applications are to address corrosion problems caused by de-icing salts rather than by marine conditions. Similarly, applications of CP on wharf structures were not reviewed because wharves are in more aggressive environments than most New Zealand bridges.

The owners' responses reflect the number of installations they have and how they are monitored:

- Owner 1: several CP installations, monitored on-site by contractor;
- Owner 2: several CP installations, monitored remotely by contractor;
- Owner 3: few CP installations, monitored on site with contractor.

6.2.1 Selection of Treatment Type

CP was selected by the owners on the basis of whole-of-life cost analysis. It was most advantageous for large structures with high capital replacement costs. On smaller systems the long-term cost of CP was closer to replacement cost so was not so beneficial. Location of the structure, volume of repair, access, initial costs, and availability of funding over the life of the structure will all impact on the choice of treatment.

For example on a large marine structure with extensive damage the up front costs of patch repair can be more than repair of damage and installation of CP because more chloride-contaminated concrete needs to be removed for patch repair.

6.2.2 Selection of CP System

Owner 1's choice of CP system was based on the systems available at the time, contractors' proposals during the tendering process, evidence of performance, aesthetic implications, risk of damage to the system by impact from traffic on and beneath the structure, and long-term maintenance costs.

Owner 1 identified under-design, inadequate investigation, use of inexperienced personnel and inadequate quality control/assurance to minimise price as causes of cost blow-outs at installation. He estimated that design, excluding investigations, would comprise 10% of the total cost, with the principal components of installation costs being access and labour. He found trials useful to select systems and establish operating criteria but impossible to carry out as part of a contract, and pointed out that the criteria set by AS 2832.5-2002 were intended to ensure satisfactory performance without the need for trials. He used a competitive tender to determine the price of the CP system, with associated repairs based on provisional quantities.

One supplier stressed the need to contract to companies that employ experienced individuals to install them because, although AS 2832.5-2002 prescribes operating criteria, it does not specify how to achieve them.

6.2.3 Monitoring

Owner 2, with several structures with CP installations, found it increasingly difficult to interpret and manage data provided by the different hardware and software systems installed on different structures at different times by different suppliers. He was considering reviewing the monitoring contracts so that all structures were monitored by the same system.

Owner 1, also with several structures with CP, noted the importance of ensuring that maintenance personnel are aware of CP installations, and that the expenses to monitor and maintain CP systems are included in annual budgets. He found this to be difficult in a large organisation unless one person manages their monitoring and maintenance over an extended period, and felt that developing management systems to do it created more difficulties. Owner 3 thought that it would be useful for the same person to be responsible for routine inspection and CP monitoring so that results could be correlated.

Structures can be monitored by the owner or by a contractor. Remote monitoring can cost approximately AUD\$3000 per annum for monthly remote monitoring plus approximately AUD\$5000 for an annual on-site inspection of the CP system. On-site monitoring can cost around AUD\$2000 to \$5000 per annum for monitoring twice a year, depending on size and location of the structure. These costs do not include remedial costs such as replacement of electrical or electronic components. Owner 1 observed that the structure could incur costly damage if a CP system is not monitored and maintained regularly.

Covino et al. (2002) report annual costs of US\$1900 for monitoring and electricity supply for an impressed current CP system on a bridge in Oregon. This is a similar order of magnitude to the costs reported by the Australian owners.

Owner 2, who employs contractors to monitor remotely, felt that it would be cheaper and simpler for his own organisation to do the monitoring itself on-site if it had the technical capability. Owner 3, who monitors on-site alongside a contractor, was considering changing to a remote monitoring system, which would allow continuous optimisation of the system while it is operating. This would increase anode life while still protecting the reinforcement. He thought using a contractor who was already monitoring for other clients with the same systems would be more efficient.

Owner 1 actively correlates monitoring outputs with observations made on-site when monitoring the CP system. Owners 2 and 3 monitor the actual condition of structures with CP as part of the routine maintenance and inspection programme, without inspectors being told to inspect for specific features on the bridges with CP installed (although owner 2 does “keep an eye on” the structures himself if he is in the area). Thus they have no formal correlation of remotely monitored CP results with site observations.

6.2.4 Hardware Durability

Anode durability is discussed by Freitag & Bruce (2002) and is not included here.

Owners acknowledged that vandalism was a problem. Recommended preventive measures included installing, as a deterrent, signs warning of danger from high voltages, chasing into the concrete surface all wiring less than 5 m from the ground, and enclosing all other hardware in stainless steel cases attached to the structure, and putting the cases where they were visible from roadways.

Most systems were mains-supplied and maintenance of power supply was not a problem.

Owner 2 had found that digital mobile telephone technology was cheaper for remote monitoring because of high landline rental charges.

Unreliability and rapid obsolescence of hardware and software was a concern for Owner 2, who had found that, although the design life of a system might be specified to be at least 20 years, in practice some components fail within 10 years. He doubted that backup for hardware would be available for the intended 20+ years service that the systems were intended to provide. Frequent changes within the industry supplying CP systems and monitoring equipment and services were seen as adding to the potential problem.

One supplier said that all hardware on the structure should last for 30 years except for the electronics, which should last around 10 years depending on their complexity. He said that manual systems have remained in operation for more than 13 years but that fully computer-controlled systems might deliver less than 10 years. Consequently he recommended that allowance be made to replace critical components in computer-controlled systems every 7-10 years. He suggested allowing

a percentage of installation cost per year to cover monitoring and maintenance. Another source indicated that electronic components may require replacement every 25 years. The Corrosion Prevention Association expects electrical components and cabling to be renewed after about 30 years (CPA 2002).

6.2.5 Effectiveness of CP

One supplier said that up to 7 years may be needed to ensure that an installation is operating satisfactorily, and that over 90% of problems compromise aesthetics or the efficiency or cost of the system, but do not increase the risk of corrosion.

All owners said that monitoring outputs met operating criteria and, in the absence of reported concrete deterioration, they believed this to mean that the reinforcement was being protected from corrosion.

One supplier reported that one bridge recently inspected was still in acceptable condition 10 years after installation of CP. McGee et al. (1998) reported that an CP system based on a mixed metal oxide mesh anode embedded in a cementitious overlay was performing satisfactorily after five years, with no physical deterioration of the concrete or the components of the CP system. Other systems had been installed more recently.

6.2.6 Summary of Experience with CP

Concern was expressed about the long-term management of a CP installation rather than the actual performance of CP.

A common theme in the responses is that the owner needs a good management system to support CP installations. This includes ensuring that annual budgets include CP operating expenses, as well as systems for managing and correlating the outputs of CP monitoring and visual inspections. Thus when installing a CP system on a structure for the first time, a bridge owner should consider how they would manage more than one structure with CP should they subsequently invest in more installations.

Experienced contractors should be used to install and maintain CP systems, and provision made for transfer of monitoring/maintenance contracts by ensuring that the hardware and operation is properly documented.

Some uncertainty exists about the long-term operating costs, and this needs to be examined when analysing costs for repair options.

7. Cost Effectiveness of Repair Techniques

This Chapter 7 describes the factors that are taken into account when comparing the costs of different repair options. It assumes that immediate or imminent replacement for strategic reasons is unnecessary, that replacement costs exceed repair costs, that repeat cycles of patch repair will not eventually reduce the structural performance of the bridge, and that repeat cycles of repair can be carried out. In practice, these factors (and others described in Chapter 3) must be considered because they could override the comparative long-term or short-term costs of repair options.

Irrespective of cost, remedial treatments must be technically appropriate. The situations under which patch repair and each of the electrochemical treatments are suitable are described in Chapter 3, Appendix A, and by Freitag & Bruce (2000). Technical advice should be sought from experienced specialist contractors, suppliers or consultants to ensure that appropriate treatments are applied.

7.1 Definitions of Service Life

The concept of a design life or service life is a design and asset management tool used to predict when some intervention is likely to be needed to maintain the structure in a serviceable condition. For a concrete structure subject to reinforcement corrosion, it can represent corrosion initiation or a defined level of damage, such as first cracking or loss of a particular amount of steel cross section.

The Transit New Zealand Bridge Manual (2003) defines bridge design life as “the life beyond which the bridge will be expected to become functionally obsolete, or to have become uneconomic to maintain in a condition adequate for it to perform its functional requirements”. The allowance for some maintenance within the 100-year design life of a bridge suggests that, as a design life definition, the time to first cracking best reflects the intent of the Bridge Manual. Therefore in the following discussion the ‘service life’ or ‘life cycle’ of a repair or structure represents the time to first cracking. Logistics mean that the actual time to repair could be up to five years longer than this.

7.2 Immediate Costs

Immediate direct costs that need to be included in cost analyses include:

- mobilisation (may be significant for remote sites);
- access (may be significant for long or high structures, or for working over a railway, road, river or estuary);
- traffic control (may be significant where lane closure is required to reduce the load on the structure or to allow access to the repair site, or may involve extending the duration of the contract to allow for restricted hours of access to avoid disrupting traffic);
- temporary support for the structure (may be significant if large volumes of concrete need to be removed);

- containment of debris (may be significant over waterways or in urban areas where debris must be collected as it is created);
- equipment and labour for removing damaged and contaminated concrete;
- equipment, materials and labour for replacing cover concrete;
- equipment, materials and labour for application of electrochemical techniques, including trials, application and, for chloride extraction and realkalisation, decommissioning.

Many of these costs will be the same for patch repairs and electrochemical treatments. The higher unit cost of electrochemical treatments may be offset by lower volumes of chloride-contaminated or carbonated concrete needing to be replaced, with reduced effects on the load capacity of the bridge during treatment. Cheaper repair materials can be used with electrochemical techniques than for patch repair.

Indirect costs, which should also be accounted for, include:

- re-routing of traffic if structural capacity is significantly reduced during repair;
- restricted hours available for working to avoid traffic disruption.

Immediate and indirect costs are site-specific and will depend on the size and capacity of the structure, its location, composition and volume of traffic, site features, and the cause, extent and severity of deterioration.

7.3 Whole-of-Life Costs

Unless replacement of the bridge is scheduled within a repair cycle, repeat cycles of patch repair will be needed as deterioration continues in unrepaired areas, possibly accelerated by incipient anode effects adjacent to repairs.

The number of repair cycles will depend on:

- the rate of deterioration, which is determined by the quality and depth of cover concrete, remaining chloride contamination, and rate of recontamination, carbonation rate, and the availability of moisture;
- the extent of damage considered to be acceptable, which is determined by aesthetic requirements, safety hazards posed by spalling, and structural requirements; and
- the future length of service required from the bridge, which is determined from projections of future traffic volumes and loads.

Repair volumes can vary from cycle to cycle.

The error in estimated whole-of-life costs of patch repair will be large because of the difficulty in predicting these factors. Whole-of-life costs should aim to identify a likely range of costs, starting with a sensitivity analysis to identify the factors that have the greatest influence on the costs. Then the effects of extreme and average values of these factors should be tested on the whole-of-life costs.

Coatings or surface treatments applied as part of the repair often may not be maintained because of the cost of access to re-coat. Under these circumstances they can be considered to increase the length of the repair cycle during which they were applied.

Similarly, use of *sacrificial anodes* as part of a patch repair system to prevent incipient anode formation should increase the length of the repair cycle by the life of the anode. The life of such anode systems currently available is estimated at 10 years, thus increasing the repair cycles to 20, 25 and 30 years for the purpose of cost comparison.

Chloride extraction and realkalisation should result in lower ongoing costs than patch repair because only the initial treatment is needed. However coatings or surface treatments applied to the treated surface will need to be maintained regularly throughout the life of the structure to prevent further chloride ingress or carbonation.

CP has ongoing costs of monitoring and maintenance. A minimum of \$2000 per annum should be allowed for monitoring, while maintenance costs should allow for replacement of electronic components every 10 years, anodes as indicated by the supplier, and other hardware every 30 years. While some access costs will be incurred during maintenance, the duration of work will be shorter than for initial installation or patch repair. Costs related to the removal and replacement of damaged concrete, and related reduction in capacity will not be incurred during such maintenance.

7.4 Comparison of Costs of Patch Repair and Electrochemical Techniques

To compare the cost of conventional patch repair to the cost of an electrochemical treatment, a scenario based on the repair history of an existing bridge has been created as an example. The case study approach is intended to ensure that inputs are realistic, rather than to justify a particular approach. The outcomes of the analysis are not relevant to the strategic needs of this particular bridge, but they illustrate what they could be on a different structure with similar problems. Although this particular bridge was affected by chloride-induced corrosion, the data were also used to examine costs that might be incurred in repairing damage caused by carbonation-induced corrosion.

The bridge was built in the 1950s and crosses over a major arterial road and a railway. Construction is cast in-situ, reinforced concrete incorporating five beams with intermediate diaphragms and full width piers. It is within 100 m of the shoreline and exposed to prevailing onshore winds from the south. The NZS 3101:Part 1:1995 exposure classification is B2 (coastal frontage).

Reinforcement corrosion has caused significant concrete deterioration on the bridge, although reinforcement section loss is not significant. Deterioration is typical for a bridge in this type of situation, with more extensive reinforcement corrosion damage on the seaward beams, although the soffit has deteriorated more than usual in some areas.

Numerous patch repairs had been carried out in the past but generally these had been unsuccessful and many have failed. A condition assessment of the structure was carried out in 1999 and subsequent repairs were completed in 2001. The principal cause of corrosion was identified as external chloride-ion contamination from wind-borne salt-laden aerosols, with lack of concrete cover and variable concrete quality being contributing factors. CP or chloride extraction both could be appropriate technical solutions given the cause, extent and severity of deterioration.

Data from the condition assessment of the structure and from the associated pricing of repair options provided the basis for the cost analysis. The costs do not include professional fees or traffic control. No allowance has been made for accrued benefits or other factors such as safety concerns, road alignment, future work or strategic importance of the structure. The volume of past and current repair did not affect structural performance, and the need to support the structure during repair was eliminated by limiting the amount of concrete that could be removed at a given time.

7.4.1 Conventional Patch Repair

A unit cost rate of \$37,000/m³ has been adopted for patch repair. This assumes a pre-packaged proprietary repair system applied according to the manufacturer's instructions. The system does not include a surface coating or treatment or internal sacrificial anodes. The rate is indicative only and is based on the average of the estimated cost and three tender prices received for repair on the bridge described in Section 7.4 above. It includes a nominal sum for site establishment and access but no allowance for road/rail traffic control.

The volume of repair material needed is difficult to estimate from the area of spalling because surface corrosion on the rebar usually extends beyond the area of damaged cover concrete. The condition survey provided a baseline estimate of repair volumes. The analysis initially used three repair volumes for comparison: one representing an extra 20% over the baseline estimated volume for every repair cycle; one representing 100% extra volume over the baseline estimate for every repair cycle; and one representing 100% extra volume over the baseline estimate for the first repair, followed by 20% extra over the baseline for subsequent repairs. This last combination is considered the most realistic of the three volumes for a well-designed and well-executed repair, and when damage is not allowed to reach the same extent before the next repair cycle.

Repeat repair cycles of 10, 15 and 20 years were used in the analyses, representing the typical cycles identified in Section 6.1 of this report for the NZS 3101:1995 exposure classifications C, B1/2 and A2 zones respectively (see also comment in Section 7.1 of this report, about time to cracking and time to repair). In practice, the 15-year cycle could represent concrete in the B2 zone with a relatively slow deterioration rate. It could also represent concrete in the A2 zone with a relatively rapid deterioration rate or in an area where it is desired to minimise visible damage or hazard caused by spalling concrete. Similarly, the 10-year cycle could represent concrete in the C zone with a relatively slow deterioration rate, or concrete in the B2 zone with a relatively rapid deterioration rate or in an area where it is desired to minimise visible damage or hazard caused by spalling concrete. These periods could be increased to 20, 25 and 30 years for repair cycles that include a surface treatment.

7.4.2 Cathodic Protection

The unit rates quoted for the application of a CP system to various structures differ with access requirements and the size of the area to be protected. This analysis is based on the installation costs of impressed current CP (ICCP) at \$600/m². This cost rate is indicative only and includes some allowance for site establishment and access. The costs estimated here are based on an area of 1200m². An additional nominal sum of \$10,000 has also been allowed to cover the costs of the preliminary design and of a pilot installation to prove and fine-tune the system on-site.

Before the application of a CP system, cracked and spalling concrete needs to be removed. There is no need to excavate behind the bar and chase the reinforcement until clean steel is reached, so the volume of repair is smaller than that required for conventional repairs. The required level of steel preparation is also reduced, and cheaper repair materials can be used, so the unit rate for repair is reduced from \$37,000/m³ for conventional patch repair down to \$20,000/m³ for CP.

CP systems also require ongoing monitoring and maintenance as indicated in Sections 6.2.3 and 6.2.4 of this report. The analysis includes annual allowances of 1%, 2% and 5% to account for these operating costs. A 1% allowance is typical. The higher allowances are included to demonstrate the effect of unexpected operational problems.

7.4.3 Chloride Extraction

A unit cost rate of \$700/m² has been adopted for chloride extraction and this includes an allowance for site establishment and reasonable access. Whether or not a surface treatment is applied as part of the process does not significantly alter this unit rate. Since the repair requirements are similar to those for a CP system, a unit rate of \$20,000/m³ has been applied to the original repair volumes. Chloride extraction was considered to provide permanent treatment with one application, providing that a coating is applied as part of the treatment when the structure is exposed to an external source of chlorides, and that the surface treatment is properly maintained for the remaining life of the structure. However no allowance was made for maintenance of surface treatments because the required frequency will depend on the treatment and exposure and is therefore difficult to predict.

7.4.4 Realkalisation

A unit cost rate of \$400/m² has been adopted for realkalisation, which allows for site establishment and reasonable access. Whether or not a surface treatment is applied as part of the process does not significantly alter this unit rate. Again the repair requirements are similar to those for a CP system so the \$20,000 /m³ rate has been adopted for the patch repairs. As with chloride extraction, no allowance was made for maintenance of coatings or surface treatments. Realkalisation is considered to provide permanent treatment with one application. Depending on the electrolyte, a surface treatment may be beneficial, but no allowance was made for maintaining such a treatment.

7.4.5 Cost Comparison

The costs of the initial repair or installation are presented in Table 7.1.

Repair volumes were based on the estimate of 4.6m³ made during the condition assessment. For conventional patch repair this volume was multiplied by factors of 1.2, 2, and 3 to allow for the extra volume that inevitably has to be removed during the repair contract, once the extent of actual corrosion and contaminated concrete is revealed.

Although coatings and internal sacrificial anodes are available as part of some proprietary repair systems they are not considered in this analysis.

Table 7.1 Initial or installation costs.

Treatment	Installation cost for applicable repair volumes			
	4.6 m ³	5.5m ³	9.2m ³	13.8m ³
Realkalisation	\$572,000	N/A	N/A	N/A
Chloride extraction	\$932,000	N/A	N/A	N/A
CP	\$822,000	N/A	N/A	N/A
Conventional patch repair	N/A	\$204,240	\$340,400	\$510,600

N/A not applicable

The costs in Table 7.1 indicate that the initial cost of conventional patch repair is significantly less than the installation cost of any of the three electrochemical techniques, even though the repair volume is greater than for the electrochemical techniques and the repair materials more expensive.

Life-cycle cost comparisons are summarised in Table 7.2. The life-cycle analyses used the Single Payment Present Worth Factor (SPPWF) and the Uniform Series Present Worth Factor (USPWF) according to Transfund New Zealand Project Evaluation Manual PEM2 (1997). Costs were analysed at the standard discount rate of 10%. Undiscounted values and values based on a reduced discount factor are also included to demonstrate the effect of using different discount factors, as may be used overseas where life-cycle cost analyses support the use of electrochemical techniques.

Costs were analysed on the basis of required future service lives of 30 years and 50 years (determined by future strategic needs rather than deterioration), with no patch repair carried out in the final year. Thirty years is normally considered the longest period for which such analyses are realistic because the discount factors could be unrealistically high for longer periods.

Table 7.2 shows that, for this particular scenario, the life-cycle costs of the repair options at preferred/normal discount rates and costs were chloride extraction \$932,000, CP \$890,000 for 30 years and \$893,000 for 50 years, and patch repair \$389,000 for 30 years and \$404,000 for 50 years, both over a 15-year cycle. This

points clearly to patch repair as the cheapest option, and this is the conclusion reached by most comparisons by asset managers using data quoted in tenders.

The other values in Table 7.2 show how these amounts are affected by the assumptions of discount factor and CP operating costs.

The difference between the undiscounted life cycle costs of CP and patch repair decreases with decreasing discount rate. Although not appropriate for current Transit New Zealand cost analyses, the 5% discount rate is perhaps the most realistic of the three rates assessed because it can be treated as a net discount rate which allows for increases in cost. However it still results in a lower life-cycle cost for patch repair. Thus the use of different discount factors in other countries does not explain how overseas owners justify the use of CP by its lower life-cycle costs.

Table 7.2 Comparison between life-cycle costs of patch repair and electrochemical processes. (Asterisks indicate values that are preferred or normally used)

Required future service life	Treatment	Cost and discount factor			
		10%*	5%	0%	
30 years	Realkalisation ¹	\$572,000			
	Chloride extraction ¹	\$932,000			
	CP ¹ 1% pa * ²	\$890,000	\$933,000	\$1,038,000	
		2% pa ²	\$958,000	\$1,043,000	\$1,254,000
		5% pa ²	\$1,161,000	\$1,375,000	\$1,902,000
	Patch repair ³	10 yr cycle	\$450,000	\$543,000	\$749,000
		15 yr cycle	\$389,000	\$439,000	\$545,000
20 yr cycle		\$371,000	\$417,000	\$545,000	
50 years	Realkalisation ¹	\$572,000			
	Chloride extraction ¹	\$932,000			
	CP ¹ 1% pa * ²	\$893,000	\$953,000	\$1,182,000	
		2% pa ²	\$965,000	\$1,085,000	\$1,542,000
		5% pa ²	\$1,179,000	\$1,479,000	\$2,622,000
	Patch repair ³	10 yr cycle	\$466,000	\$619,000	\$1,157,000
		15 yr cycle	\$404,000	\$509,000	\$953,000
20 yr cycle		\$375,000	\$446,000	\$749,000	

1. Based on 4.6m³ volume of repair at time of treatment.
2. CP monitoring and maintenance costs, expressed as an annual allowance that is calculated as a percentage of the installation cost.
3. Based on 9.2m³ volume of repair in the first repair and 5.5m³ for all subsequent repairs.

Use of different allowances for CP operating costs has a relatively small effect on its life-cycle cost.

The final cost of repair to the bridge used in this scenario was higher than the original estimate. The main reasons were the volumes of old concrete to be replaced being greater than anticipated and costs associated with working with high volume roads and rail. The effects of repair volume and increased unit rates were examined again in the next stage of the analysis.

The volume of repair clearly affects the result of the analysis directly. Tables 7.1 and 7.2 show that the cost of patch repair increases in proportion to the repair volume. Although tender prices might be based on the initial estimate of spalling/cracking with a small allowance for extra repairs, final costs could be significantly higher, with actual repair volumes two to three times the initial estimate. Increasing the repair volume to 13.8m³ followed by subsequent repairs of 9.2m³, increases the 10% discounted cost of patch repair from \$389,000 to \$592,000 for 30 years and from \$404,000 to \$616,000 for 50 years, reducing the difference between the costs of CP and patch repair.

The costs of working around traffic are highly site-specific. In some circumstances they will include traffic control and delay costs but are more likely to include the cost of avoiding disruption to traffic. On this bridge it meant restricted working hours, with access being established and disestablished at the start and finish of each working period. This increases the duration of the work programme, and thus increases the unit rate. The effect of increasing the unit rate of patch repair to \$44,000/m³ (the average rate of the two tenders that were not accepted) and the repair component for electrochemical treatments to \$25,000/m³ to allow for restricted working hours changed the relative life-cycle costs of treatments for chloride-induced corrosion to those shown in Table 7.3.

Table 7.3 also shows how the life-cycle cost of CP becomes more favourable for shorter repair cycles. A 5-year cycle was introduced to represent concrete in the C zone with a relatively fast deterioration rate. It is perhaps more applicable to wharf structures than to bridges, but could apply to a bridge over an estuary with a low clearance over the water level.

Table 7.3 Effect of increased repair volumes and unit rate of repair, and of shorter repair cycles on treatment costs (10% discount factor).

Treatment	Required future service life		
	30 years	50 years	
Realkalisation ¹	\$595,000	\$595,000	
Chloride extraction ¹	\$955,000	\$955,000	
CP ²	\$912,000	\$916,000	
Patch repair ³	5 yr cycle	\$958,000	\$1,010,000
	10 yr cycle	\$823,000	\$856,000
	15 yr cycle	\$704,000	\$733,000
	20 yr cycle	\$667,000	\$676,000

1. Based on repair volume 4.6m³, no coating or surface treatment.

2. Based on operating costs of 1% pa, repair volume 4.6m³.

3. Based on repair volumes of 13.8m³ for initial repair and subsequent repairs of 9.2m³.

Although the data in Table 7.3 still show patch repair to be cheaper than CP for the 15-year repair cycle typical for bridges in the B1/B2 zone that are affected by chloride-induced corrosion, the difference between the two costs is much smaller than in the original calculations. This shows that assumptions such as repair volumes and costs associated with traffic using the affected structure, or a carriageway beneath it, can significantly affect the outcome of an analysis. Any analysis of life-cycle costs therefore should examine the likely range of costs, preferably with attached risk factors.

Table 7.3 demonstrates how the cost effectiveness of CP increases as the repair cycle shortens. This means CP is a more realistic option in more aggressive environments, and explains why it is used overseas on bridges exposed to deicing salts, and on marine structures in New Zealand and overseas. It also explains why it has not been a favourable option for any New Zealand bridges considered to date. Similarly, at present installation costs chloride extraction is less likely to compare favourably with the alternative of a 15-year cycle of patch repair, particularly if one adds the cost of maintaining an associated surface treatment. Chloride extraction is not recommended for concrete that is in directly in contact with sea water, so is unlikely to be considered as an alternative to a 5-year repair cycle even though the costs are comparable.

Table 7.3 indicates that the life-cycle costs of a 20-year patch repair cycle are comparable with those of realkalisation. Hence realkalisation might provide a cost-effective alternative to patch repair for a bridge in the A2 exposure zone which has been severely damaged by carbonation-induced corrosion, although the repeat repair volumes used in this particular analysis may be over-estimated.

7.5 Summary

The purpose of the iterations in the cost comparison presented in Section 7.4.5 of this report was to show how changes in some factors affect the outcome of a cost analysis much more significantly than others, and how such changes can alter the actual outcome.

If the extent and/or severity of corrosion damage suggest that an electrochemical treatment would be technically appropriate, the cost analysis for the repair options should include a sensitivity analysis. This analysis would determine which factors will have the most effect on costs, should the costs vary from the estimated value, and would be followed by analysis of the effects of the likely extreme values. The ultimate decision should then be based on the likelihood of these extremes being reached.

The cost comparison presented above also demonstrates the importance of carefully selecting consultants and contractors. Tenders that are significantly lower than their competitors may not have made adequate allowance for repair volumes or the need to accommodate traffic or other factors in the work programme. Long-term cost analysis based on estimates from inadequately experienced consultants or tenders from inadequately experienced contractors can give misleading outcomes.

Life-cycle cost analysis must take into account the required future service life of the structure, and the effect on life-cycle costs should that service life subsequently be exceeded. A decision based on any projected future service life is not appropriate unless likely reasons for replacing the bridge have been established, and the risk of changes in circumstances that would alter these reasons has been assessed.

Immediate costs of patch repair will almost always be lower than the costs of installing electrochemical treatments. Life-cycle costs will also usually be lower for patch repairs, but can favour electrochemical treatments under certain circumstances. Such conditions may arise for structures severely damaged by carbonation-induced corrosion, or which have extremely high exposure to chlorides such as in marine structures, and where significant access-related costs or traffic-related costs are incurred during repair. Technical advice should be sought from experienced specialist contractors, suppliers or consultants to ensure that appropriate treatments are applied.

Electrochemical treatments are unlikely to be selected other than as trials or demonstrations if only immediate installation costs are considered. However, increasing development and adoption of asset management practices based on life-cycle costing may encourage the use of electrochemical repair techniques, surface treatments and internal anodes, in specific circumstances.

8. Summary and Conclusions

A strategic approach taking into account the immediate and long-term needs of a structure will ensure the most cost-effective repair technique. Specialist technical investigation is required to identify the cause, extent and significance of deterioration and to predict the likelihood of future deterioration. In practice, immediate costs rather than strategic aspects usually take priority when deciding how to repair an individual structure. Nevertheless, whatever the approach taken, an understanding of the principles behind various repair methods, and the benefits that each can deliver, will help to ensure that appropriate types of treatment are selected for the problems exhibited on a specific structure.

A wide variety of repair products is available, many of them targeted for specific applications. Proprietary products offer better quality control and performance guarantee, and are purpose-designed for optimum performance in different situations and therefore they are preferred over site-batched materials. Individual products within proprietary repair systems include reinforcement coatings, bonding agents, patch repair materials, corrosion inhibitors, internal sacrificial anodes and surface treatments. The components selected for a given repair will depend on the individual circumstances of the repair.

As with repair methods, an understanding of the principles behind individual products, and knowledge of their specific advantages and limitations, are necessary to ensure that appropriate products are selected. Specialist technical advice should be sought from suppliers of proprietary materials or from independent experts to ensure that the repair materials selected are appropriate for the treatment principles and methods adopted. Previous experience with individual products and contractors will also assist in ensuring that appropriate products are selected, and that they are applied correctly.

The performance of patch repairs observed to date (2003) is generally considered acceptable by bridge owners, with the main problems reported being related to achieving repairs of acceptable quality for the lowest possible immediate cost.

Site evidence from previously repaired structures suggests that patch repair cycles are typically 5-10 years for NZS 3101:1995 exposure zone C, 10-15 years for exposure zones B1/B2, and 20 years or more for exposure zone A2. The length of the repair cycle on a given structure is determined by the specific exposure conditions of the site, the quality of concrete and construction, the quality of the repair and the level of ongoing damage that is considered acceptable. The length of repair cycles could be increased by incorporating surface treatments, or internal sacrificial anodes in the repair system.

Owners of bridges protected by CP systems have not questioned its long-term performance, but they stressed the need for good management systems to support the long-term operation of CP systems. Such systems need to ensure that annual budgets include CP operating expenses, as well as manage and correlate the outputs of CP monitoring and visual inspections. When installing a CP system on a structure for the

first time, a bridge owner should consider how they would manage more than one CP structure should they subsequently invest in more installations. The uncertainty in ongoing CP operating costs needs to be accommodated in analyses of life-cycle costs of repair options.

The cost analysis of repair options for a particular structure (which may or may not include electrochemical treatments) should identify the factors that will have the most effect on costs if they vary from the initial estimate. It should then analyse the effects of the likely extreme values of these factors on the final cost. The ultimate decision can then be based on the likelihood of these extremes being reached. Any cost comparison must carefully consider repair volumes and issues relating to accommodating traffic during the repair process, and use data from reliable sources. Life-cycle cost analyses also need to consider the required future service life of the structure.

Immediate costs of patch repair will almost always be lower than the costs of installing electrochemical treatments. Life-cycle costs will also usually be lower for patch repairs, but can favour electrochemical treatments under certain circumstances for bridges with an indefinite required future service life. Such circumstances include structures severely damaged by carbonation-induced corrosion or which have extremely high exposure to chlorides such as in marine structures, and where significant costs related to access or traffic are incurred during repair. Technical advice should be sought from experienced specialist contractors, suppliers or consultants to ensure that appropriate treatments and products are applied.

Electrochemical repair techniques, surface treatments, and internal sacrificial anodes can offer cost advantages over the service life of the structure, but are unlikely to be selected other than for trials or demonstrations if only immediate installation costs are considered. Increasing development and adoption of asset management practices based on life-cycle costing could encourage the use of electrochemical repair techniques, surface treatments and internal anodes, in specific circumstances.

9. Recommendations

It is recommended that:

- Bridge asset managers take a strategic approach when assessing repair options for a bridge affected by reinforcement corrosion. To ensure the most cost-effective repair technique, the immediate and long-term needs of a structure and the communities it serves need to be taken into account.
- A formal inventory system be utilised to record details of bridge condition and remedial treatments carried out. This will help to establish the long-term needs of individual structures.
- Remedial treatments be appropriate for both the technical and strategic needs of the individual structure concerned. Therefore repair systems and products should be selected with an understanding of the principles behind them, their specific advantages and limitations, their long-term performance, and the required service life of the structure. Ongoing maintenance associated with treatments such as surface treatments and cathodic protection also needs to be considered. Specialist technical advice should be sought to ensure that appropriate treatments are selected.
- A model specification for patch repairs based on up-to-date technology be adopted. This would help to ensure that a consistently high standard of patch repair is carried out on New Zealand bridges.
- Analyses of treatment costs include both immediate and long-term costs. They should include a sensitivity analysis to determine the factors that will have the most effect on costs should they vary from the initial estimate, and the effects of the likely extreme values on the final cost. Particular attention should be paid to costs associated with access and traffic control, both of which are sometimes overlooked, and repair areas and volumes, which are easily under-estimated.
- Technical advice be sought from experienced specialists to identify the cause, extent and significance of deterioration, to predict the likelihood of future deterioration, and to ensure that appropriate remedial treatments are selected for individual structures.
- Electrochemical treatments be considered under certain circumstances, particularly for structures severely damaged by carbonation-induced corrosion or which have extremely high exposure to chlorides such as in marine structures, and where significant costs related to access or traffic are incurred during repair.

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Appendices

Appendix A: Methodologies for Selecting Corrosion Treatments

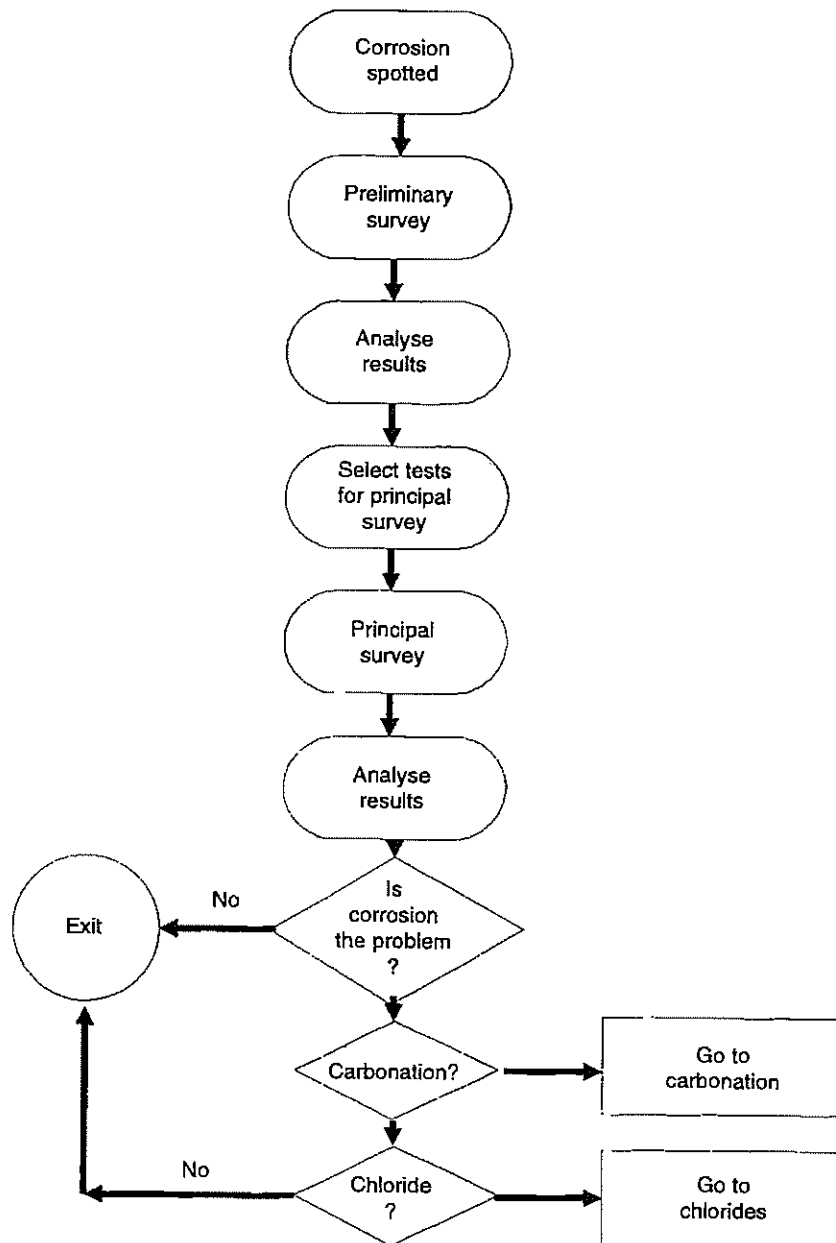


Figure A1 Methodology for selecting a repair or rehabilitation method (Broomfield 1997).

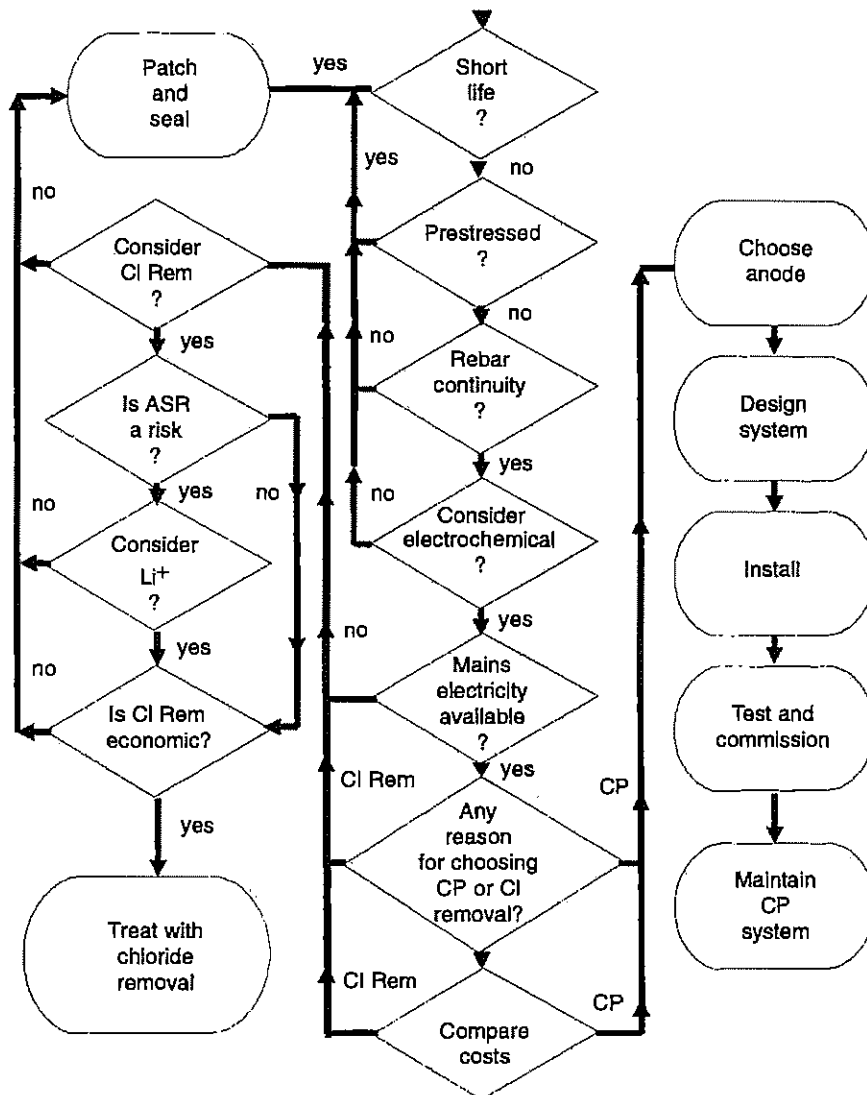


Figure A2 Methodology for selecting a treatment for corrosion caused by chloride contamination (Broomfield 1997).

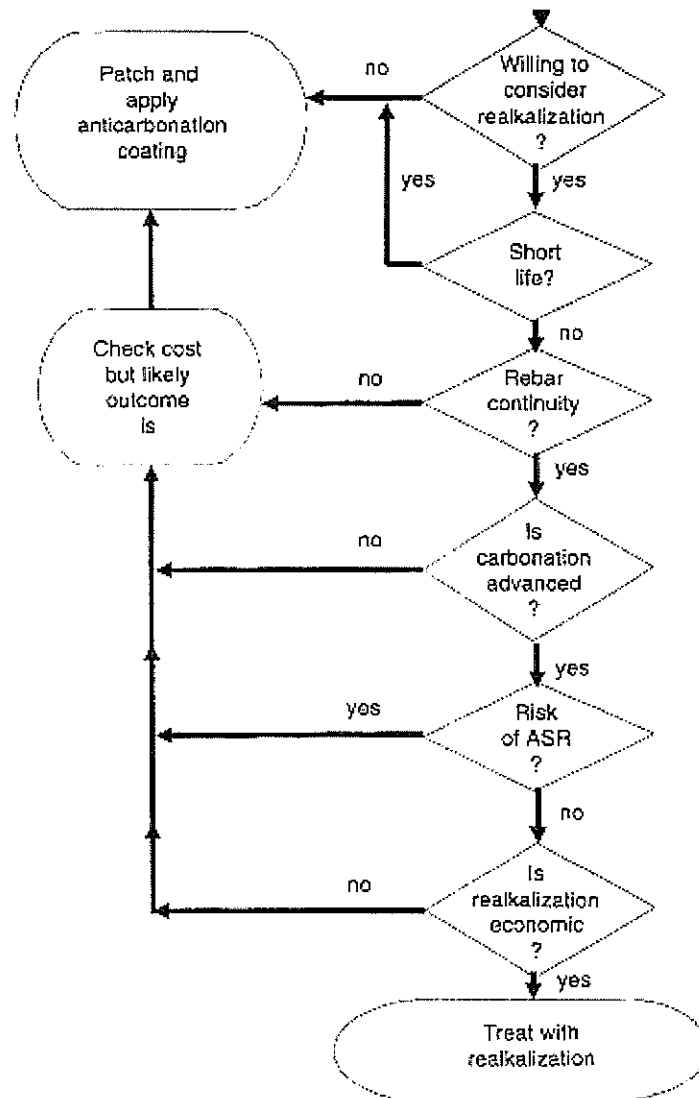


Figure A3 Methodology for selecting a treatment for corrosion caused by carbonation (Broomfield 1997).

Appendix B: Case Studies of Patch Repaired Structures

Kilbirnie Fire Station

The Kilbirnie Fire Station in Wellington was built in 1972 and is in the B2 exposure zone. A condition assessment carried out in 1992 identified the following as contributing factors to extensive reinforcement corrosion damage exhibited at several locations on the site:

- Lack of cover concrete;
- Poor quality concrete;
- Chloride ion contamination from a chloride-based accelerator added to the fresh concrete, use of contaminated aggregates, and external environmental contamination.

Following the condition assessment, repair contract documents were circulated to invited tenderers and the contract was let to the lowest price tender of \$184,000 (compared to the estimated price of \$214,000). The contract specified a complete proprietary cementitious repair system, and listed examples of suitable products. The coating system was specified as the Fosroc Nitocote Dekguard system or proven equivalent.

Patch repairs were subsequently carried out in 1994/95 using cementitious shrinkage-compensated fibre-reinforced mortar, and pourable micro-concrete repair systems (MBT Emaco S88C and Emaco S88 Pourable respectively). An acrylic coating system (MBT Barracryl Elastic) was applied to the entire structure following repair to prevent further ingress of chloride ions. Major repairs such as on columns were carried out using cast repairs in formwork. Hand-applied mortar was used for smaller repairs.

Performance of the repair system to date (8 years after application) is satisfactory. However some cracking caused by ongoing reinforcement corrosion is evident at several locations, generally where hand-applied repairs have been carried out.

Paremata Bridge

The Paremata Bridge was built in 1936 at the head of a tidal inlet on the west coast just north of Wellington. Much of the pier height is in the C exposure zone, the piers being founded in seawater. The top of the piers and the superstructure are in the B2 zone. The bridge had been repaired using sprayed concrete at least once before a detailed assessment made in 1992, during which ongoing deterioration in both the original concrete and the sprayed repairs was observed, principally in the piers and beams, and particularly the seaward beam. The assessment concluded that the most significant factor in ongoing reinforcement corrosion was elevated chloride ion levels.

The repair contract was open tender and was let to the lowest price-conforming tender for \$391,000, compared to the estimated price of \$610,000.

Patch repairs were carried out in 1994/95 using site-batched repair materials incorporating an additive described as a mortar improver (Sikachem 810). An acrylic coating system (Sikaguard 550) was applied to the entire structure following repair.

After approximately 8 years many of the repaired areas appear to be sound but ongoing corrosion is evident at several locations. Deterioration includes cracks that appear to be unrelated to repairs, cracks adjacent to patch repairs (suggesting incipient anode action), and failure of patch repairs. Some surface rust staining was also noted on a large patch on the side of a pier.

Orowaiti Bridge

Built in 1938, the Orowaiti Bridge crosses a tidal lagoon and is located approximately 1 km from the coast at Westport, South Island. The lower two thirds of the piers are in the C exposure zone, and the upper parts of the piers and the superstructure are in the B1/B2 zone. Damage caused by reinforcement corrosion was repaired with shotcrete in 1970, cast concrete in 1981, and proprietary repair materials in 1996.

The cast-concrete repairs carried out in 1981 consisted of a site-mixed concrete poured into formwork that extended 50 mm out from the face of the beams. Concrete was excavated beyond the depth of the reinforcement but not beyond the longitudinal extent of cracking. A cement slurry was applied to the reinforcement and surrounding concrete prior to application of the repair material. These repairs have effectively increased the depth of cover concrete at the beam face in the area of repair. Although incipient anodes had developed toward the ends of these repairs, the repairs themselves are still intact.

A condition assessment carried out in 1992 found that the primary cause of reinforcement corrosion was inadequate concrete compaction. Although the site is subject to onshore winds from the north and west, chloride ion levels were not excessive. Extensive patch repairs were carried out in 1996 using either shrinkage-compensated pourable micro-concrete (Fosroc LA 55) or a cementitious-based lightweight shrinkage-compensated polymer-modified mortar (Fosroc HB 25). Following repair, a coating system, (Fosroc Dekguard Elastic) based on a silane-siloxane primer and an elastomeric acrylic topcoat, was applied to the entire superstructure of the bridge.

Eight years after the 1996 repairs, the Orowaiti Bridge is in reasonably good condition but eight areas of cracking caused by ongoing reinforcement corrosion were noted. This deterioration is both in and adjacent to areas of previous repair, although the pigmented coating prevented definition of individual repairs.

Other West Coast Bridges

Six other West Coast Bridges built in 1936 between Greymouth and Hokitika have similar repair histories to the Orowaiti Bridge:

- South Beach Overbridge;
- New River Bridge;
- Serpentine Creek Bridge;

- Acre Creek Bridge;
- Kapitea Bridge;
- Waimea Bridge.

These bridges are all within 1 km of the coast and therefore within the B1/B2 zone.

All these structures are suffering ongoing deterioration caused by reinforcement corrosion, particularly at the seaward edge of the beams. A limited condition assessment was carried out on these bridges (apart from Serpentine Creek and Acre Creek) in 1998. Although the chloride ion levels measured were not excessive, the most likely cause of deterioration is chloride contamination from unwashed beach aggregates used to make the concrete and subsequent external contamination from salt-laden winds. Local areas of poor compaction, low cover, and carbonation beyond the depth of reinforcement may also be contributing factors at some locations.

The first round of major repair was carried out on the bridges between 1980 and 1982 to repair widespread delamination of cover concrete. Although voluminous corrosion product was visible on the surface of exposed reinforcement, little section loss had occurred. Cast-concrete repairs were performed in a manner similar to those carried out in the same era on the Orowaiti Bridge. An undefined silicon-based product was sprayed over all of the beams following the repair. The repairs have performed well and were all still intact in 2003.

Isolated minor repairs were carried out in the late 1980s, mainly where reinforcement had corroded between and adjacent to the earlier repairs. This phase of repair included epoxy mortar repairs and cement-based mortar placed over epoxy bond coats. The repairs were applied simply to replace spalled concrete, without excavating behind or along the steel to remove contaminated concrete. Several of these repairs are still intact.

Following the condition assessments in 1998, a further round of repairs was carried out in 1999. The contract documents specified that a proprietary repair material conforming to several stated criteria was to be used. Repair types were specified along with typical breakout dimensions for each type of repair.

Deterioration was typically located in original concrete or between repairs but cracking often extended into previous repairs, indicating incipient anode corrosion. Concrete excavation extended behind the reinforcement and a nominal 200 mm beyond the visible extent of longitudinal deterioration. Edges were cut square following excavation, and reinforcement was water-blasted, then sand-blasted to SA2½. Reinforcement was coated with a zinc-rich anti-corrosion primer (Nitoprime Zincrich) prior to the application of repair materials. Patch repairs were then applied, comprising a combination of Fosroc LA55 micro-concrete and Fosroc HB 25 repair mortar.

In general these repairs have been executed to a high standard under a high level of supervision with the surface condition of the reinforcement often checked before the application of repair materials. Four years after application the repairs are still in good condition, although there are new cracks in the original concrete, often adjacent to a previous repair.

Fairfield Bridge

The Fairfield Bridge is a three-span bowstring arch bridge, with two approach spans, crossing the Waikato River in Hamilton. The site is in the A2 exposure zone. Construction was completed in 1937 and by the mid-1950s deterioration was visible. An investigation in 1981 concluded that cracks in the end blocks and some of the arches were related to alkali aggregate reaction. A further condition assessment in 1991 attributed ongoing deterioration principally to poor quality concrete, lack of compaction, and poor drainage detailing.

Following the condition assessment, above-deck areas of the structure were repaired in 1992. The contract documents specified a proprietary repair method and coating system conforming to several stated criteria. The typical types of defects likely to be encountered were also listed. MBT Emaco S88 micro-repair concrete was used and following repair the above-deck structure was coated with MBT Barracryl Elastic as an anti-carbonation coating.

In 1994 the below-deck elements were repaired by a different contractor under a different contract using a combination of Fosroc LA55 micro-concrete and Fosroc HB 25 repair mortar. A coating system (Fosroc Dekguard S) incorporating a silane-siloxane primer and an aliphatic-acrylate top coat were used to coat the beams, deck soffits and the piers in the vicinity of the water level. Dekguard Elastic was used to coat the rest of the piers.

Because of the large volume of repair required and the short timeframe of the contract, the balustrades were repaired using sprayed concrete applied to a female mould.

After approximately 10 years the repairs are still in good condition with no obvious deterioration of repairs or parent concrete.

Rail Bridge, Queensland

Piers on a rail bridge in Queensland built in 1909 were made from poor quality concrete that was highly permeable and had allowed reinforcement to corrode. The bridge crossed land that was to be flooded to increase the capacity of a water supply reservoir. Because this would be the last chance to repair the concrete easily, the piers were repaired with a Xypex repair system to both restore and waterproof the concrete. The work was carried out in 2002 and the owner reported that after 12 months were reportedly still sound, whereas from past experience he would expect to see recurrence of damage within 12 months.

Various Bridges and Wharves

Markham Distributing identified several New Zealand bridges and wharves that had been treated with the 'Aquron' system, between 1999 and 2003. One application was to a marine structure that was affected by reinforcement corrosion and the rest were treatments to prevent corrosion damage, or applications to demonstrate how well the product can penetrate cover concrete. At the time of writing it is too soon to evaluate the effect of the treatment on actual steel corrosion at these sites, although results from one study indicate that some of the desired effects on the concrete have been achieved (GHD Pty Ltd report to Allcrete Industries, 2003).