Sustainable Development, Global Change and Ecosystems

Sustainable Surface Transport

Strategic Targeted Research Project

ARCHES

Assessment and Rehabilitation of Central European Highway Structures

Guideline for Smart Cathodic Protection of steel in concrete

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Drafted:	Rob Polder, Andrej Kranjc, Jan Leggedoor, Aljoša Šajna, Gerard Schuten, Irina Ošlaković Stipanović	30.06.2009
Reviewed:	Jan Eri	20.08.2009
Approved by ARCHES Management Group:		Tomasz Wierzbicki

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1. EXECUTIVE SUMMARY

Unplanned maintenance of ageing civil engineering structures and in particular concrete bridges is a major problem, among others for organisations responsible for road networks in Europe. Especially corrosion of reinforcement is widespread, which raises questions about structural safety. Repair of corrosion related damage is very costly and may cause a structure to become unavailable for prolonged periods of time.

As part of the ARCHES project, this report addresses Cathodic Protection (CP) as an alternative approach to remediation of corrosion with considerable benefits. Benefits may be lower cost over the whole life, shorter execution time, longer working life of the intervention and increased durability and safety.

Owners of large stock of bridges and/or their consulting engineers need to know the basics of corrosion of reinforcement, concrete repair methods and CP. Further elaboration of repair and CP are specialised activities that may be left to technical specialists. On a general level, owners should be aware of the long and successful track record of CP of concrete and its flexibility to suit the needs of individual structures.

This report first sketches the technical background and provides examples, focusing on bridges with corrosion damage and the solutions provided. Then a Technical Guideline is presented with the major steps described that have to be made to apply CP. Next, CP is placed in a wider framework of concrete maintenance, including economic considerations and case studies of whole-life costs. The Annex provides descriptions of trial CP systems that were investigated and some typical examples of bridges that suffer corrosion.

The conclusions to this study can be summarised as follows.

Cathodic protection (CP) of reinforcing steel has been applied to concrete structures with corrosion damage for over 25 years. World wide experience shows that CP prevents further development of corrosion damage in a reliable and economical way for a long time, provided that the CP system is designed, executed and maintained properly.

Since the 1980’s, CP has been applied to buildings, marine structures, tunnels and bridge decks and substructures in the US, Europe, the Middle East, Asia and elsewhere in the world. New anode materials have become available, of which in particular activated titanium and conductive coatings have proved their good performance over more than two decades. CP of concrete structures has been standardised in the US since the 1980s and in Europe since the year 2000.

As a wide variety of anode systems is available with a good track record and tailor made solutions can be provided for every type of structure.

The principle of CP is active and permanent intervention in the corrosion process. Therefore, monitoring is an essential part of operating a CP system. The main advantage is that monitoring proves the absence of corrosion on a regular basis.

Designing a CP system for a particular structure requires that proper information is available on the structure, in particular about the extent of damage and the layout of the reinforcement. CP can be applied to both reinforced and prestressed concrete structures. Application to post-tensioned structures can be done on a routine basis. It is recommended to perform a trial in case of slender (low cover) pre-tensioned elements. In all cases, very negative potentials of stressed high strength steel should be avoided, as they may cause loss of ductility. Dedicated monitoring of pre-tensioned steel or post-tensioning ducts in prestressed structures is needed to avoid overprotection. Numerical modelling of CP has been demonstrated to be a useful tool, both for predicting CP operation in general and the safety of potentials of prestressing steel in particular.

CP systems have a long working life. As growing experience has shown, typical life of conductive coatings systems is more than 10 years. The life of typical activated titanium systems is at least 25 years and probably much more. Corrosion of reinforcement will be completely absent during this period and probably many more years after the end of the working life. The long life of CP is in strong contrast to the life of conventional repairs. European studies and typical examples have shown that conventional repairs may have short lives of as low as five to ten years, among others due to poor execution quality. The working life of repairs is highly uncertain and the absence of corrosion and potentially severe loss of cross section cannot be guaranteed.

CP fits into a rational maintenance strategy in various ways. Both from technical and economical points of view, the recommended scenario is that corrosion and concrete damage are detected in a relatively early stage and that subsequently CP is applied. Waiting until damage has become extensive is unfavourable, as it will increase total costs. Applying CP before damage has appeared may be favourable in individual cases only.

Based on case studies, the cost of CP over the (remaining) life of a structure has been investigated and compared to other options. Generalising, it appears that CP may be instrumental in saving considerable amounts of money over the remaining life of a structure, say over periods of 10 to 25 years.

2. INTRODUCTION

2.1 General

General info, setup of sub-project

Cathodic protection (CP) has been applied successfully to stop corrosion of reinforcing steel in concrete structures across the world since about 25 years. However, the number of cases where CP has been applied is low in view of the huge potential need for durable protection of corroding structures. This has to do with CP being an “unknown” technology for most users and with economical aspects. Generally, CP is a significant investment with a relatively long life cycle, which has the potential of improving safety and serviceability and saving large amounts of money on the mid to long term. Consequently, CP is still a “promising” technique, despite its long track record.

This work package of ARCHES aims at innovative CP design, to demonstrate its potential benefits of and to lower existing barriers for applying CP in particular to bridges. The set-up of ARCHES WT3.2 is:

- to improve the economy of CP of concrete structures by making smart, that is “slender” but effective designs
- to support slender CP design using advanced computer modelling,
- to test slender CP systems on typical corroding bridges in New Member States
- to validate the modelling based on the testing, and
- to present the results in this Guideline.

Two trial CP systems were realised, one in Slovenia and one in Poland. Testing started in 2008 and results up to early 2009 are incorporated.

This document first describes the State of the Art of concrete CP and then goes into more detail about design of CP systems. Next, economical aspects of maintenance of concrete structures are illustrated, including the impact of CP and issues related to conventional repair. Examples of practical cases are described wherever relevant. The main text finishes with discussion and conclusions. All technical details of the trial systems and the numerical modelling are located in the Annexes.

2.2 State of the Art

2.2.1 General

Cathodic protection (CP) of reinforcing steel has been applied to concrete structures with corrosion damage for over 25 years. World wide experience shows that CP prevents further development of damage in a reliable and economical way for a long time, provided that the CP system is designed, executed and maintained properly. CP is particularly suited in cases where chloride contamination has caused reinforcement corrosion. Application started in the USA in the 1970's on bridge decks suffering from corrosion due to de-icing salt penetration resulting in severe damage to the concrete. Since the 1980's, CP has been applied to buildings, marine structures, tunnels and bridge decks and substructures in the US, Europe and elsewhere in the world. New anode materials became available, of which in particular activated titanium and conductive coatings have proved their good performance. A further development is application of CP to new structures where corrosion is anticipated within the service life, termed cathodic prevention. A European standard was published several years ago [CEN 2000]. By the year 2000, cathodic protection had been applied worldwide to about 2,000,000 m^2 of reinforced concrete on a few thousand (at least partially) corroding structures.

2.2.2 Corrosion of concrete reinforcement

The normal situation of steel reinforcement in concrete is passivity [Bertolini et al. 2004]. This is a state of almost negligible corrosion rate, caused by an atomically thin oxide film on the steel surface, which is stabilised by the high alkalinity in concrete (pH over 13). This passivation may be lost due to two causes:

- either carbon dioxide ingress, which reduces the pH to values about 9 (carbonation), causing a more or less uniform loss of passivation and general corrosion,
- the presence of chloride ions above a critical threshold, which locally break down the passive film starting pitting corrosion, despite a high pH; chloride may be either cast in or penetrate from the environment.

A third cause, stray current interference, will not be treated here. It may be relevant for bridges in railway systems with DC traction.

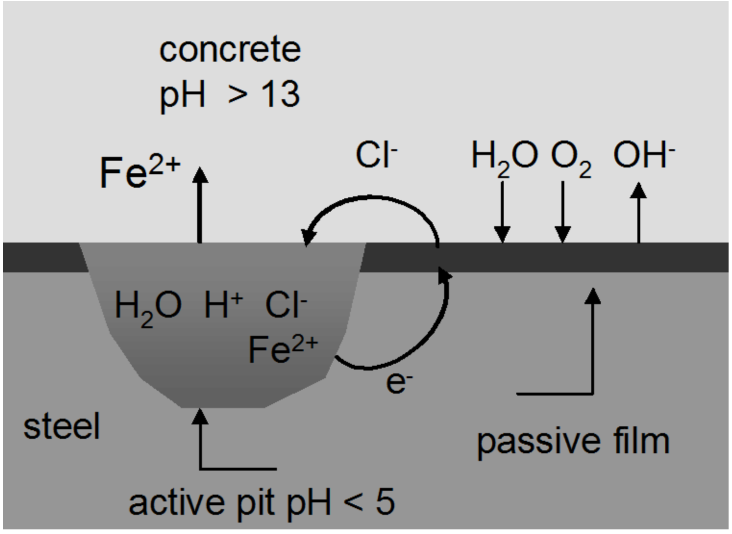

Figure 1 Schematic of pitting corrosion of steel in concrete

Corrosion is an electrochemical process, in which the electrical potential of the steel and the exchange of electrical current between steel and concrete pore solution play important roles. Figure 1 sketches the principles. In the passive state, the potential of the steel is relatively positive, due to a reaction of oxygen at the steel surface, consuming electrons (termed the cathodic reaction):

$$O_2 + 2\ H_2O + 4\ e \rightarrow 4\ OH^- \qquad (1).$$

The dissolution rate (corrosion rate) of passive steel is negligible.

When passivation is lost (due to chloride or carbonation), iron passes into solution as ferrous ions, leaving excess electrons in the steel, which makes its potential more negative; this reaction is termed anodic:

$$Fe \rightarrow Fe^{2+} + 2\ e \qquad (2).$$

In the presence of chloride, the passivation is lost locally (at imperfections, air voids etc.) on the steel surface, resulting in (small) anodic spots compared to the large cathodic area (Figure 1). Potential differences between cathodic and anodic sites cause currents to flow in the concrete pore liquid, transporting more chloride ions to the corrosion pit, further accelerating the steel dissolution reaction. The overall reaction rate (in atmospheric concrete structures) is thought to be limited by the electrolytic resistance of the concrete.

The ferrous ions react with hydroxyl ions formed at the cathodes and with more oxygen to form various solid hydrated ferric oxides, commonly called “rust”:

$$Fe^{2+} + 2\ OH^- \rightarrow Fe(OH)_2 \qquad (3),$$

$$Fe(OH)_2 + 1/4\ O_2 + ½\ H_2O \rightarrow Fe(OH)_3,\ \ Fe_2O_3.H_2O \text{ etc.} \qquad (4)$$

These corrosion products are more voluminous than the original steel. The net effect is expansion, causing tensile stresses in the surrounding concrete. After relatively small amounts of steel have been transformed into corrosion products, rust stains may appear (Figure 2), the concrete cover cracks and spalling or delamination occurs (Figure 3). Cracking and spalling in themselves can be unacceptable, but they also have to be taken as a warning of further decay: when left to corrode, the steel bar diameter may decrease below structurally acceptable values. Concrete repair may be necessary and the corrosion protection must be reinstated, e.g. by CP.

Figure 2 Rust stains and cracking in bridge cross beam due to reinforcement corrosion

Figure 3 Spalling and delamination due to corrosion

2.2.3 Principles of cathodic protection of steel reinforcement in concrete

Cathodic protection is based on changing the potential of the steel to more negative values, reducing potential differences between anodic and cathodic sites and so reducing the corrosion current to negligible values [Polder 2005, COST 521 2003, Broomfield 1997]. The change of potential, called polarisation, is caused by forcing a direct current to flow from the concrete into the steel. In practice this is realised by mounting an external electrode, the anode, on the concrete surface, connecting it with the positive terminal of a low voltage direct current source, while connecting the negative terminal to the reinforcement cage, Figure 4. This is called impressed current CP. Sacrificial systems are also possible, where a less noble metal provides current to the steel by sacrificially corroding.

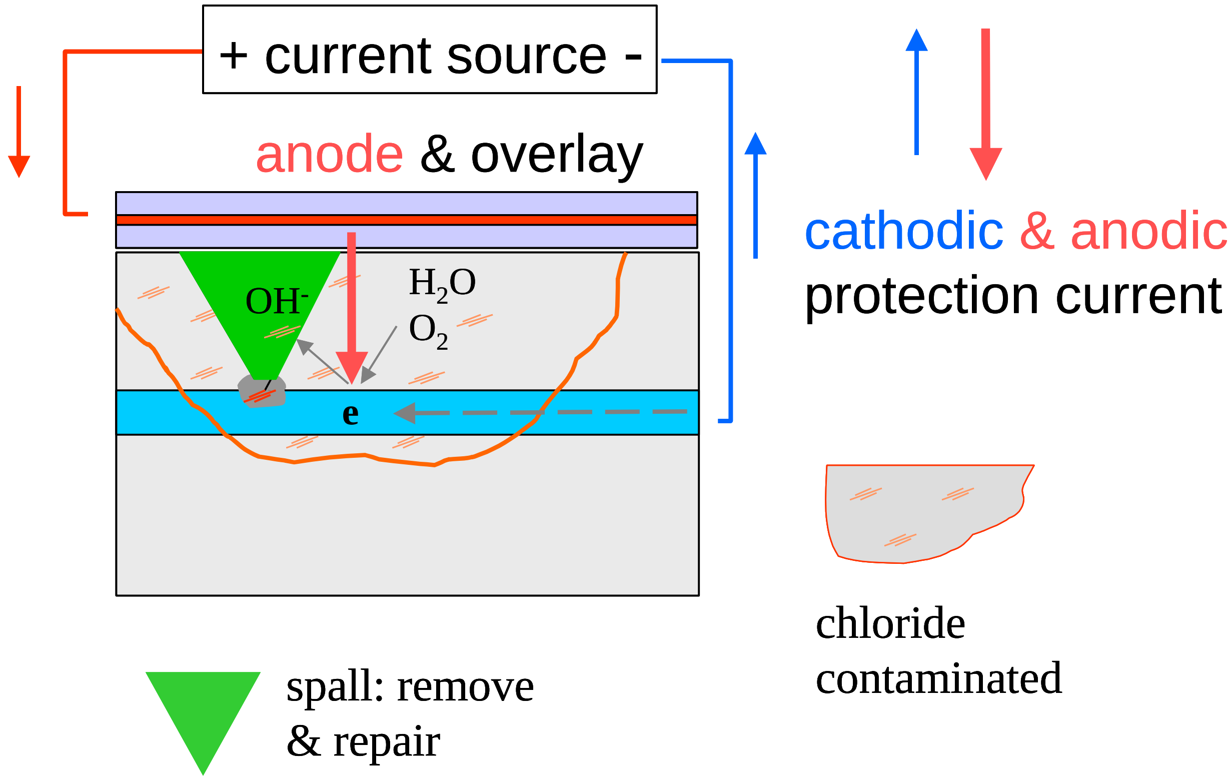

Figure 4 Principle elements of a cathodic protection system for concrete

Electrons flow to the steel/concrete interface through the reinforcement cage and increase the cathodic reaction, which produces hydroxyl ions from oxygen and water. The hydroxyl ions migrate through the concrete cover to the anode where they are oxidised to oxygen and electrons. The electrons flow through the anode cables back to the current source, which closes the electrical circuit. As a result of this current circulation, cathodic reactions are favoured and anodic reactions at the steel are suppressed. Relatively moderate current densities of the order of 10 mA/m^2 of steel surface area or even less are able to restore and maintain steel passivation and have various beneficial chemical effects. These effects make that the current need reduces over time. The required polarisation makes CP a permanent method: the current must flow during the remaining service life of the structure. Due to the beneficial effects, current interruptions of days to weeks do not harm the protection and thus are acceptable. For a uniform distribution of the protection current, the steel must be electrically continuous and the concrete must have a reasonably homogeneous conduction (resistivity). At any time and place, short circuits between the anode and the steel must be avoided.

2.2.4 Anode materials

The heart of any CP system is the anode material, which comes in various forms (Figure 5). For impressed current CP, these are:

- an electroactive mesh (mixed metal oxide activated titanium) shaped to fit the surface of the structure and subsequently covered with a cementitious overlay;

- a set of activated titanium wires or strips or titanium oxide rods placed in holes or slots and backfilled with a cementitious grout;
- a conductive and electroactive coating (loaded with carbon particles) covering the concrete surface;
- carbon fibres in various shapes, including woven mats embedded in a (polymer modified) cementitious overlay.

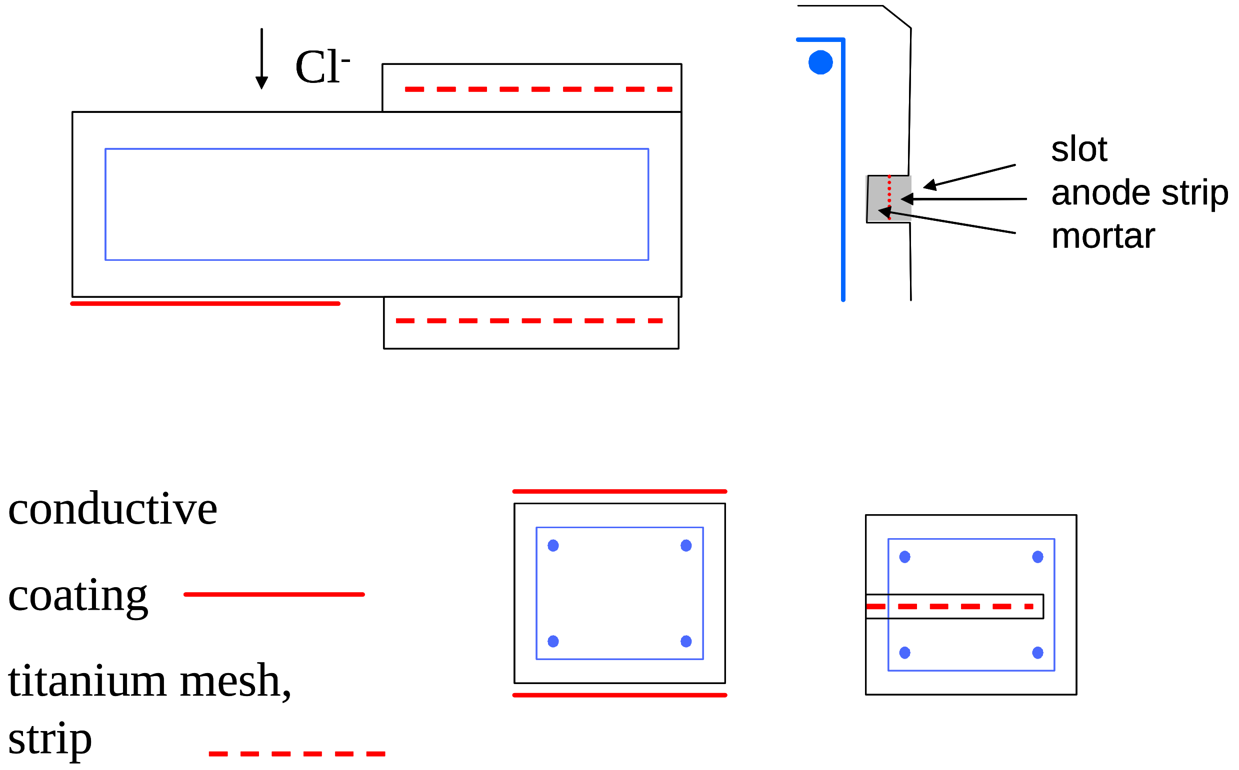

Figure 5 Anode materials and anode systems for CP of concrete

Activated titanium expanded mesh with a surface coating of mixed metal oxides is the most widely used and successful type of anode. It can be easily fixed to all surfaces, and is overlayed with a cementitious material, such as shotcrete (vertical or overhead surfaces) or flowable concrete (horizontal surfaces), Figure 6. The adhesion of an overlay depends on appropriate surface preparation and proper execution of the application technique. Titanium strip and wire systems involve inserting activated titanium strips into holes drilled into concrete elements or slots cut into the surface, which are then filled with cementitious grout or mortar. In general titanium systems have the ability to deliver a relatively large protection current without compromising their durability.

Figure 6 Three stages of application of a titanium mesh to the underside of a concrete slab; *left*: prepared surface; *middle*: titanium mesh attached (note repair patch and plastic fixing pins); *right*: shotcrete overlay applied

Conductive organic coatings are solvent based or water borne products, containing graphite particles to provide conduction. A series of metallic conductors is embedded in the coating to feed the current. A cosmetic top coat (similar to normal concrete paint) is applied, Figure 7. The adhesion of the coating system depends on appropriate surface preparation and a suitable application technique. This type of anode can be applied to complex shapes in any orientation and presents no addition of weight. Since about 1990, conductive coatings have shown to be able to provide a working life of over 10 to 15 years [Polder et al. 2006].

Carbon fibre materials including woven mats (textiles) constitute a new type of anode materials, which supposedly combine the advantages of titanium mesh and conductive coating. They are claimed to provide durable anode properties and can be applied in thin overlays of polymer modified cement mortar, thus adding little weight, Figure 8.

Sacrificial anodes have been used to a certain extent for CP of steel in concrete. They can be: zinc mesh and overlay; zinc sheet attached to the concrete using a conductive gel; flame sprayed zinc. Based on the limited use of sacrificial anodes, the shorter experience and questions as to their service life, no investigations have been carried out into sacrificial anodes in ARCHES.

Figure 7 Conductive coating anode applied on edge beam of bridge deck adjacent to abutment

Figure 8 Carbon fibre mesh applied in first layer of cementitious mortar (to be finished with another layer of mortar)

2.2.5 Monitoring of cathodic protection

The quality of protection offered by a CP system must be monitored by regular electrical testing (normally a few times per year) and by visual inspection (once a year). Test connections are provided in a control box that contains the power sources and in an increasing number of cases also a datalogger for remote monitoring, Figure 9. As a general measure of the quality of cathodic protection, the amount of polarisation that actually takes place in the structure is measured: as long as CP causes a certain minimum amount of polarisation, corrosion is suppressed to an insignificant level. The polarisation is tested for by interrupting the protection current and monitoring the subsequent change of the steel potential over periods up to 24 hours at several representative points in the concrete structure using embedded sensors (called reference electrodes), Figure 10. Upon switching off the current, the steel potential relaxes from polarised to non (or less) polarised. The polarised potential is measured within 1 second from switching off (called instant off). The potential measured after 4 to 24 hours after switching off is assumed to be the non-polarised (relaxed) potential. The difference in potential between instant off and at 4 or 24 hours is called depolarisation; this test is called depolarisation test. Empirically, a minimum depolarisation of 100 mV is considered to demonstrate sufficient protection for atmospheric concrete structures. For submerged or buried structures other criteria are applied, in particular a fixed minimum negative potential [CEN 2000].

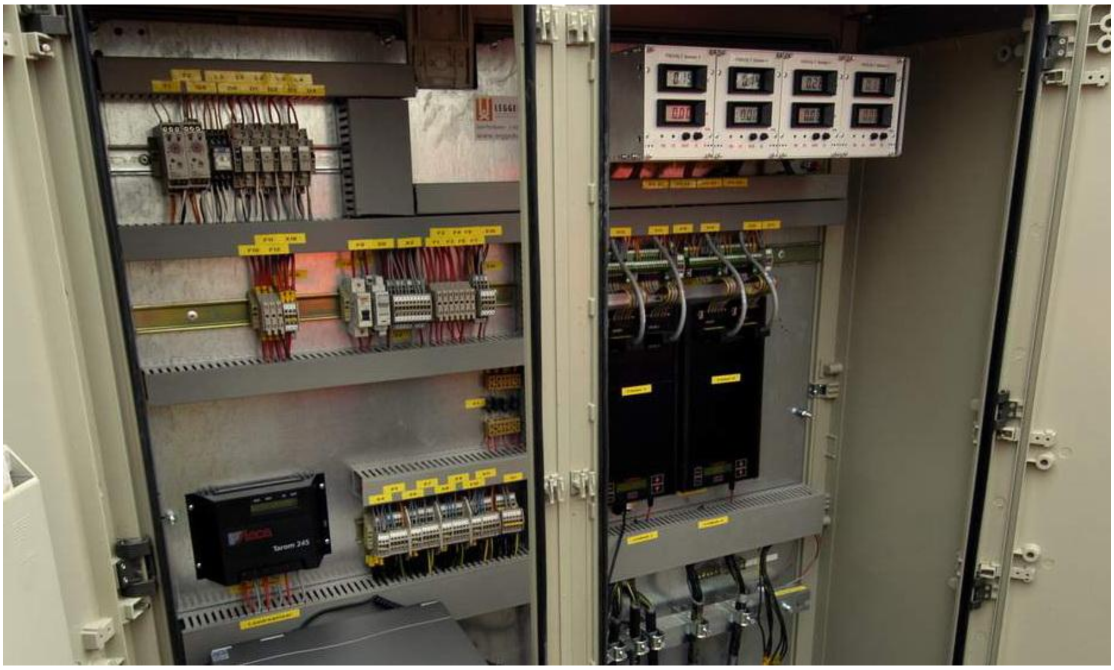

Figure 9 CP system control box with four power units (top right), datalogger/remote control (middle right), cable connections and testing points (left)

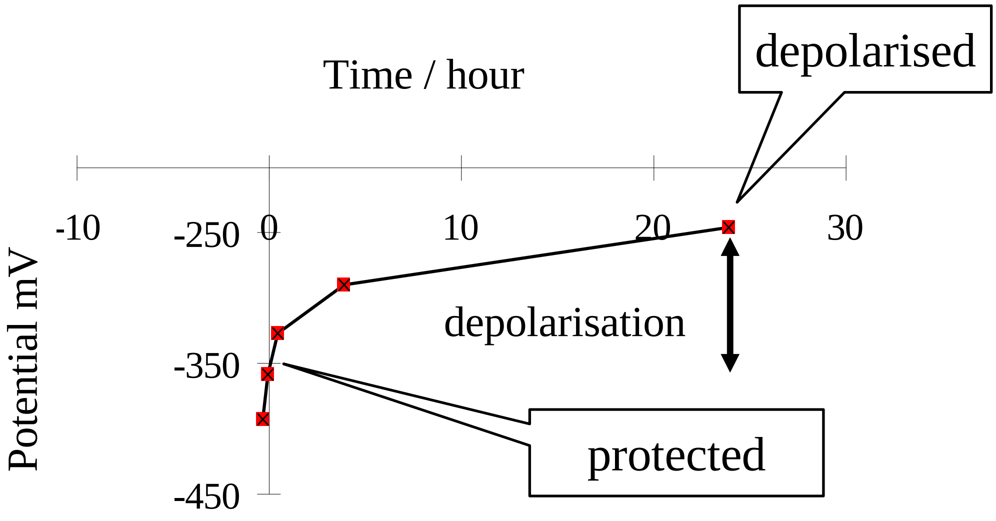

Figure 10 Principle of depolarisation test of CP of concrete

When during a test session, the level of depolarisation is found to be too low, the current or the voltage is increased. During the next testing session (three to six months later), the result of that increase will be observed. If necessary, the voltage is again increased. In general it is felt that taking small steps in this process is beneficial. Too high polarisation should be avoided ("overprotection") as it would reduce the working life of the anode system and its contact with the concrete.

From a practical point of view, the monitoring of CP systems is usually organised in a contract between the owner and the CP-contractor, by which the contractor carries out the testing and visual inspection, which are then reported to the owner. In an increasing number of cases, remote monitoring is installed. The most advanced type of remote monitoring also allows adjusting the voltage or current (remote control).

2.2.6 Cathodic prevention

Cathodic prevention (CPre) has been applied to large motorway bridges in Italy and to various other structures where early depassivation and corrosion were anticipated [Bertolini et al. 1996, COST 521 2003]. Although the principle is the same as for CP, CPre can be achieved more easily: it is easier to **prevent** pitting corrosion than it is to **suppress** ongoing pitting corrosion. This means that less current density is required and that “prevention current” spreads deeper into the structure than “protection current” does. Consequently, CPre systems can be designed and executed simpler, less anode material is needed, and/or their service life is longer. Installing anodes in the construction stage is much cheaper than it is to install them on an existing structure. CPre is monitored in the same way as CP, however the results are even more reliable due to the effects mentioned above. Generally, CPre is conceived in the design stage and applied in a fully integrated way when the structure is built. Anodes can be placed inside the concrete at optimised locations. This is the situation where the cost is minimal. However, in some cases CPre has been installed shortly after a structure had been finished. The difference with a fully preconceived and integrated case is that the anodes had to be placed on the surface. The costs generally will be higher than for a fully integrated system. The beneficial effects are the same.

2.2.7 Closing remarks

Overwhelming experience with concrete structures in various environments all over the world has shown that CP is very effective in stopping corrosion of reinforcement, even if high levels of chloride are present in the concrete and aggressive external conditions continue to be active. This has been documented in many case studies of successful application of CP, concerning tunnels, bridges, floor elements, foundations, balconies and façade elements [e.g. Polder 1998; Grefstad 2005; Nerland et al. 2007]. An increasing number of structures with carbonation induced corrosion has been provided with CP, in particular in Norway and The Netherlands. Technical guidance is given in International Standards and other documents.

3. GUIDELINE FOR DESIGN OF CP SYSTEMS FOR CONCRETE STRUCTURES

3.1 CP design

3.1.1 General

Design of a CP system should be based on proper knowledge of a structure including understanding its structural system, its condition (cause and extent of ongoing degradation processes, amount of damage) and physical characteristics (cover depth, concrete quality and composition, environment and exposure). In particular for CP, information is required on the severity and extent of corrosion, the size and position of reinforcement and its electrical continuity; and the electrical resistivity of the concrete. This information should be collected by studying the records of the case and by a dedicated inspection.

It should be realised that CP, as any technique, has a limited scope. If there are signs of inadequate structural capacity, a structural engineer should be involved. CP does not bring back steel that has already been lost due to corrosion; consequently, structural measures may be necessary in such cases.

Assuming that the required information is available, this chapter presents the following items: design considerations, anode system, power circuit, monitoring system, concrete removal, repair and overlay materials and the presence of prestressing. It finishes with a number of example cases.

3.1.2 Design considerations

The most general requirements to a CP system are that it should deliver sufficient protection to the reinforcement for a sufficiently long time.

"Sufficient protection" refers to a state of the reinforcement that cannot be designed for directly; it will be the result of sufficient current reaching the steel. What can be designed is a system that provides protection current. Protection current causes polarisation, of which a particular minimum amount reduces corrosion to negligible levels. Due to huge differences between structures (in terms of moisture, chloride, cover depth, cement chemistry), there is no specific current density that will produce the required polarisation. Fortunately, experience has shown that current densities between 2 and 20 mA/m^2 of steel surface usually produce sufficient polarisation to stop ongoing corrosion. Consequently, a design current density of 20 mA/m^2 of steel surface is usually applied. Design thus requires the steel surface density to be known, referring to the circumferential surface of the bars (not the cross section!). The effect of ribs is small and can be neglected. As a rule of thumb, bridges contain about 1 m^2 or more of steel surface per m^2 of concrete surface; buildings usually contain between 0.5 and 1 m^2 of steel surface per m^2 of concrete surface.

Similarly, the working life of CP systems cannot be designed for directly, as the degradation mechanisms involved are just beginning to be understood. General view is that keeping the current density at the anode/concrete interface below certain levels is sufficient to obtain a long life. For conductive coating systems, the maximum current density is 20 mA/m^2 of anode surface, usually equal to concrete surface. Titanium anodes can be used up to 110 mA/m^2 of anode surface (or double for short periods, e.g. up to months). Titanium anodes do not cover 100% of the concrete surface, so for mesh type anodes a maximum current density per concrete surface is specified depending on mesh width and wire thickness (based on the real titanium surface), e.g. 30 or 40 mA/m^2 of mesh surface, which usually equals concrete surface. For titanium strip or wire anodes, practical maximum values are based on their working surface. E.g. titanium strips of 20 mm width (and a particular mesh structure) have a maximum output of 5.5 mA per running meter of strip. For a slotted system (Figure 5), the number of strips in a square meter of concrete surface is adjusted to obtain sufficient current density.

Normally, CP design is based on the consideration that protection current (and thus the polarisation achieved) has a limited spatial distribution from the anode into the concrete to the steel. Both into the depth and sideways, about 0.2 m from the anode is the maximum (effective) limit of current spread that usually is counted on. Consequently, most anode systems are designed to cover the whole concrete surface of those elements where corrosion is visible or suspected. For internal anodes, the maximum distance to rebars is about 0.2 m.

Further design considerations are the addition of mass or thickness (overlays), the required service life of the system (mainly the anode), the appearance of the finished surface, the severity of environmental actions and the cost.

3.1.3 Anode systems

Anode types have been illustrated in chapter 2. The most important anode system properties that need to be considered in the design are added mass and service life. In Table 1 various properties have been highlighted for various anode systems. Based on experience the following remarks are made for each anode system.

Titanium mesh/overlay: heavy, robust, long experience, long life (> 25 year); high tolerance for external moisture; surface appearance changed by overlay; adhesion of overlay is critical, which has to do more with surface preparation and mortar (shotcrete) technology than with CP. Cabling can be placed in the overlay.

Titanium strips in slots or boreholes: lightweight, robust, long experience, long life (> 25 year); maintains most of original surface, anode-to-anode cabling must be placed in recesses (slots) in the concrete; high tolerance for external moisture; avoiding short circuits to reinforcement requires significant attention. A recent innovation is placing the strips on the concrete surface and surrounding them with a “dyke” of cementitious mortar (of about 50 mm width and 20 mm thickness). This type of anode system was applied on bridges in Norway and in a trial in Slovenia.

Conductive coating: lightweight, long experience, medium service life (> 10 to 15 year), over 20 years of positive experience; sensitive for excessive external moisture; changes surface appearance; cabling must be placed in recesses.

Carbon fibre mesh/overlay: moderate weight addition, short experience, probably long life, changes surface appearance; cabling in recesses or in the overlay.

Table 1 Anode system properties important for design of CP systems

Anode system Property	Titanium/overlay	Titanium in slot or borehole	Conductive coating	Carbon mesh/overlay
Mass added (and thickness)	Yes (typically 60 kg/m^2)	none	none	Some (30 kg/m^2)
Service life	>25 year	>25 year	10 – 15 year	25 year?
Cost	high	medium	medium	medium
Visual appearance	Surface covering	local	Surface covering	Surface covering
Typical applications	Bridge decks, substructures, walls, quays	Precast beams (buildings, bridges, columns)	Buildings (bridges)	Buildings and bridges

3.1.4 Power units and cabling

Power units should be designed for being able to provide sufficient current. As outlined above, the current needed cannot be predicted. Thus, power unit design is usually based on a safe value of 20 mA/m^2 of steel surface. A conservative approach would be design at this value, counting all steel in the structure. A more relaxed approach would be to count only the steel in the outer zones, or even to design at this value only for actively corroding steel; and at a much lower value for passive steel, e.g. 2 mA/m^2.

Nevertheless, power units as such are not expensive and having some overcapacity is beneficial at low cost. In any case, accidentally designing a too small power unit (which would not be able to provide sufficient current to obtain full protection), can always be corrected by replacing it with a larger one.

Cables should be designed for low resistance and a minimum loss of power, preferably at rather conservative design current densities. Replacing cables is costly, so here a relaxed approach is not recommended.

3.1.5 Monitoring system

Monitoring by depolarisation is based on having sensors at representative points in the structure. These sensors are usually so called reference electrodes (RE's) that should be embedded in the concrete near the steel. Representative means that they should preferably be placed near (formerly) corroding steel. There are several types of RE's, which fall in two categories: true reference electrodes and decay probes.

True reference electrodes are based on a well defined chemical reaction; they have a potential that is stable over long periods of time (years). Types are mainly silver/silverchloride/KCl (Ag/AgCl) and manganese/manganese dioxide (MnO_2). These types are more expensive than decay probes. They are prescribed for monitoring the potential of prestressing, but may be used for depolarisation testing.

Decay probes are based on various materials that maintain a stable potential over shorter periods (days), but not reliably over months to years. Typical types are based on activated titanium in cementitious mortar, graphite or lead. They are suitable for testing depolarisation (up to 24 hours) and relatively cheap.

Some guidance on the number of RE's is given in the Dutch recommendation [CUR 45, 1996]. It states that 1 RE should be present per 100 m^2 of concrete surface, with a minimum number of 2 per zone. As some RE's may fail in the course of years, it seems appropriate to install a redundant number. Coming back to install new RE's is costly. This is a typical issue of contractor's risk.

RE's are usually provided with their own reinforcement cables, in order to avoid ohmic drop and transient (switching on/off) effects, which may cause erroneous readings.

3.1.6 Removal and repair of concrete

Obviously, existing damage to the concrete has to be repaired before CP can be installed. This involves breaking out loose (cracked, spalled) parts, cleaning the steel superficially and applying a cementitious mortar. Polymer mortars and bonding agents are not allowed, as their high electrical resistance would block the protection current.

It should be noted that the amount of breaking out and the intensity of steel cleaning for CP are much less than with conventional repair. Conventional repair completely relies on the passivating properties of the mortar; so any chlorides remaining in corrosion products or in concrete that has not yet cracked and spalled, may cause re-activation of corrosion. This involves removal of concrete down to larger depths and over a wider surface area than for CP. Breaking out chloride contaminated concrete and cleaning of steel are critical for conventional repair. Carrying them out to the optimum is hard in practice. Consequently, conventional repairs may have low effectiveness and poor durability. This was found in a European survey, CONREPNET, which reported that the working life of (conventional) repairs is such that 50% has failed within 10 years [Tilly & Jacobs, 2007; Matthews et al. 2007].

3.1.7 Overlay and repair materials with CP

Overlays and repair mortars should allow ample current flow and should provide durable adhesion.

Adhesion of overlays (shotcrete) and repair mortars is obviously critical for CP, as it is for conventional repair. Surprisingly little is known on a theoretical level of adhesion and disbonding. Mortar properties as (drying) shrinkage are certainly important; materials should meet the relevant requirements from the EN 1504 Standard series "Products and systems for the protection and repair of concrete structures" should be met. However, most critical is probably the adhesion to the underlying concrete. In practice, substrate tensile strength, surface preparation and application quality seem to determine durable bond. Adhesion or lack thereof is taken care of by bond testing: a minimum of 1.0 MPa and a mean of 1.5 MPa are required according to [EN 12696]. The importance of surface preparation cannot be overstressed: in several cases, CP systems have shown poor performance due to bond loss of overlays, caused by insufficient surface preparation. As the cost of repair is high, this situation should be avoided to the benefit of all parties involved.

In order to allow protection current flow, overlay and repair mortars should have good electrical conduction, similar to the original concrete. Mortar conduction can be checked by resistivity testing. Repair mortar should have a resistivity not less than half and not more than twice the resistivity of the parent concrete. This is to prevent that repairs either carry much more or far less current than adjacent sections. Whether or not resistivity requirements should be specified for overlays is a matter of debate. If an overlay mortar (e.g. in which a titanium mesh is embedded) is applied in a constant thickness over the concrete surface, its resistivity does not really influence current distribution. A resistivity requirement does not seem to be critical.

Mortar resistivity is determined by cement type, (polymer) modifications and by water content. Portland cement and low-polymer modified mortars have low resistivities. Blended cements (slag, fly ash, pozzolana, silica fume) cause higher resistivities, just as in ordinary concrete. Polymer modifications may raise resistivity only slightly or considerably, depending on polymer content and type. Consequently, it is recommended that mortars are tested for resistivity. Simple tests are available, either based on cubes or mortar bars measured between steel plates [COST 521, 2002] or on specimens with embedded electrodes. The Dutch recommendation [CUR 45, 1996] specified 8 weeks of exposure in very wet (fog room) and semi-dry (20°C 80% RH) climates.

Concrete resistivity can be measured on site, e.g. using Wenner type probes [Polder 2000], or on cores taken from the structure, using the two electrode method [COST 521, 2002]. If the cement type is known, the resistivity can be estimated using Table 2.

Table 2 Concrete resistivity values based on cement type and exposure [COST 521, 2002], at 20°C for mature (>10 years) dense-aggregate concrete; conditions in [ ] are corresponding laboratory climates

Environment	Concrete resistivity $\rho_{concrete}$ (Ω m)	
	Ordinary Portland cement (CEM I)	Blast furnace slag cement CEM III/B (>65% slag) or CEM I with fly ash (>25%) or silica fume (>5%)
Very wet, submerged, splash zone, [fog room]	50 - 200	300 - 1000
Outside, exposed	100 - 400	500 - 2000
Outside, sheltered, coated, hydrophobised (not carbonated) [20°C/80%RH]	200 - 500	1000 - 4000
ditto, carbonated	1000 and higher	2000 - 6000 and higher
indoor climate (carbonated) [20°C/50%RH]	3000 and higher	4000 – 10,000 and higher

For example, a typical structure in wet environment made of CEM I may have a resistivity of about 200 Ωm. A repair mortar would be compatible with CP on such a structure if its resistivity is between 100 and 400 Ωm.

3.1.8 Execution and start up

During execution of a CP system, the design plan is to be realised, taking into account that the design is based on limited testing. In the execution phase the complete surface is accessible and the amount of damage, cover depth and other details can be checked more precisely.

The complete surface of the structure is sound hammered for loose parts. Loose parts are removed and the steel continuity is checked by measuring electrical resistance between exposed bars. If steel continuity is poor (demonstrated by resistance values over 1 Ω), continuity has to be provided. This is generally done by cutting slots perpendicular to the main bar direction and welding steel wires to all bars. Open spots (and continuity slots) are repaired. Over the complete surface a minimum cover depth to the steel has to be present, of at least 15 mm for conductive coatings and 20 mm for titanium mesh systems. This is to prevent short circuits between anode and steel, which would make significant parts of the system ineffective. The whole surface is cleaned and further prepared for receiving the anode system. Testing of the potential bond strength is required. Reference electrodes are installed and cables are placed in slots as far as needed.

Subsequently, the anode is installed. In case of a conductive coating, primary anodes (usually metal wires) are installed and the conductive coating is applied (in two or more layers); and finally a cosmetic top coat is applied. In the case of a titanium mesh system, the mesh is installed with primary anodes (usually titanium strips) and the overlay is applied.

Before energising the system, electrical checks are made. The electrical resistance between anode and steel is checked. A resistance well over 1 Ω demonstrates the absence of short circuiting. However, a large zone may have a resistance near or below 1 Ω without a short circuit.

Consequently, testing should be done in sufficiently small parts to be able to detect short circuits. The resistance of a particular part can be estimated using the concrete resistivity from Table 2 taking into account the size and the concrete cover (between reinforcement and anode) by

$$R = \rho \,.\, d / A \qquad (5),$$

with R resistance (Ω), ρ resistivity (Ω m), A surface (m^2) and d cover (anode-reinforcement (m). As an example, for CEM I concrete in an exposed situation (ρ = 200 Ω m) and 25 mm cover depth, a part without short circuit with at least 1 Ω will have a size of 5 m^2 or less. This requires that in the execution stage, small parts be kept electrically separated and tested; after successful testing they can be connected. Sheltered structures will have a higher resistivity and consequently, separate parts to be tested can be larger.

Further tests before start-up are measuring potentials and resistances of embedded reference electrodes. Potentials provide baseline data for later comparison under CP. Electrode resistances provide a baseline for later checking their functioning. Depending on type of electrode and concrete, their resistance is in the order of a few 100 Ω to 100 kΩ. A reference electrode resistance (at start up or later during service) of 1 MΩ or more suggests poor electrolytic connection to the concrete and the steel or damage to the cable. A large decrease of resistance suggests damage to the cable insulation or excessive leakage near the electrode.

At start up, a moderate voltage is applied to the system, for example 1.2 V. The total current and reference electrode potentials are monitored for at least a few hours. The current should become relatively stable in a few to 24 hours. Potentials will become progressively more negative over a period of days to weeks.

Over a period of four weeks to three months, regular checks should be carried out until the system operates satisfactorily. At each check, depolarisation is tested for four to 24 hours. If any values are below the required 100 mV, the voltage should be increased. It is recommended to do so in small steps of typically 0.2 V. If many depolarisation values are above 250 mV, the voltage should be decreased. During checks, resistances are measured for reference electrodes. Such measurements provide insight in their proper functioning. Monitoring the resistance between anode an cathode provides information over drying out of the concrete.

In the course of the early operation/intensive testing period, sufficient depolarisation will be achieved at all measuring points and the current will become relatively stable.

3.1.9 Service phase

During the service or operation phase, regular electrical checks are carried out as foreseen in the design, usually every three to six months, in addition to a complete visual check of the whole structure at least once a year. Some variation of parameters, in particular current, may occur with seasonal variations (temperature, precipitation). Over the years, most systems have a tendency to lower current and more positive steel potentials. This expresses that conditions at the steel are becoming less aggressive. However, large variations and in particular sudden increase of current should be taken as indications of unforeseen changes. In particular leakage over parts of the structure may cause a local drop of concrete resistance, increase of current and drop of potential. In extreme cases and in particular for conductive coating systems, the anode may become damaged. Another cause of undesired changes may be short-circuits caused by work on the structure, e.g. drilling holes for fixings anchors etc. During visual checks, signs of leakage (traces of running water, defect drainages and so on) shall be looked for carefully. Routine elements of annual visual inspections are checking the integrity of all surfaces (coating, overlay) and cabling; and the absence of signs of corrosion and cracking.

Normally, the company carrying out the checks will report once a year to the owner about the operation of the system. When irregularities are observed, interim reporting and corrective actions may be required.

3.1.10 CP and prestressed structures

A recent European concerted Action, COST 534, has paid attention to application of electrochemical protection methods to prestressed structures. A brief summary of the conclusions with regard to cathodic protection is give here. For more information reference is made to the Final Report (Part IV) [Polder et al. 2009a]. In principle, both cathodic prevention and cathodic protection can be applied safely to prestressed structures. The main concern is avoiding overprotection of prestressing steel (or post-tensioning ducts). Both CP and CPre work by applying relatively low levels of current density and steel polarisation. So, provided that they are properly designed and monitored, these methods will protect the steel and will not promote hydrogen evolution; thus, the probability of damaging the prestressing is negligible. Numerical modelling has preliminarily confirmed this view and can be used to predict the safety of prestressing steel in particular cases [Polder et al. 2009b]. The report provides two examples, one of a successful trial on pretensioned beams in a marine quay; and one of a full scale project on pretensioned bridge slabs and reinforced cantilever column heads.

3.1.11 Service life

Recently, the durability of CP systems has been investigated. From a study on samples taken from structures with CP after up to nine years of operation, one proprietary conductive coating system was concluded to have a service life of well over ten years [Polder et al. 2001]. Experience and test results show that activated titanium systems have service lives of 25 years and more [Polder et al. 2002]. Further work on quantifying the service life of CP systems was carried out, both from empirical data and using physical models [Polder et al. 2006], which can be summarised as follows. CP system service life should be based on a probabilistic approach, parallel to service life design of concrete structures. A failure analysis of components and operational factors was carried out. Oxidation and acid formation may cause degradation of anode materials, concrete or failure of adhesion of anode or overlays. These degradations are proportional to the protection current density. Based on theoretical considerations, field data and laboratory studies, degradation models and limit states were proposed for these time-dependent degradations. Accelerated corrosion of anode-cabling connections and failure of reference electrodes are also potential causes of failures. An inventory of 70 CP systems installed in The Netherlands between 1987 and 2004 has provided data on degradation and failures. Conductive coatings are sensitive to local moisture ingress, for example due to leaking joints, which promotes locally increased current densities and premature loss of anode integrity and/or adhesion. The majority of conductive coating CP systems was found still working satisfactorily after ten years. Failure of anode-copper cable connections is limited to a few cases. This type of failure must be avoided by careful design and execution of connections.

Modern types of reference electrodes appear to operate satisfactorily for a long time. An update of the CP system inventory in 2009 allows to state that 90% of documented CP systems in The Netherlands showed to have a working life of 13 years or more.

3.2 Advanced Numerical design

Based on recent developments in numerical modelling of corrosion processes in concrete including electrochemical phenomena [Redaelli et al. 2006], numerical design of CP systems has become possible. A finite element computer model is made of a typical part of the structure to be protected, which ideally includes the geometry, polarisation processes at the anode and at the steel and the ohmic drop in the concrete. Using such an electrochemical numerical model, current densities and potentials can be calculated at every point in the modelled structure. They can be used to verify the spread of protection for various anode configurations and to predict local polarisations that can be tested experimentally [Polder et al., 2008a,b; 2009b].

A critical problem in modelling of CP is formed by the input variables. In particular the corrosion current density of the steel is an important variable. This parameter is strongly influenced by the flow of protection current. Once CP has become effective, all steel will be passivated and only passive steel should be modelled to predict the distribution of current and polarisation. However, at start up some steel will be actively corroding. Consequently, there is a period of transition from active to passive. A conceptual model was set up to predict the length of this transition period. The model results were compared to start up data from the test site in Slovenia, which showed satisfactory agreement [Polder& Peelen, 2009]. It appears that passivation took place in about four hours. Pending further confirmation, this would mean that steel in concrete under normal CP operating conditions can be modelled as being passive.

As a side note, attention should be paid to the difference between cases where damage has fully developed at the moment when CP is applied, as compared to cases where no concrete damage has developed yet (but where the steel is already actively corroding). In the former case, some passivation may result from the cleaning of the steel and subsequent alkaline repair mortar application (as in the Slovenian trial system). In the latter case, the only active force to passivate the steel is the CP current. Thus, it may take longer before passivation is complete in cases where no repairs have been carried out. Tentatively this could take a week. On the long run of the CP operation, however, this may not make any difference at all.

4. ECONOMY OF CP AS A MAINTENANCE STRATEGY

4.1 General

Generally, maintenance of concrete structures is aimed at guaranteeing structural safety and reliability, or minimum acceptable performance, over the entire service period; and at doing so at the lowest total cost. Degradation processes (such as chloride penetration induced reinforcement corrosion) tend to reduce reliability with time. Maintenance actions (inspections, protective measures, repair) have the effect of increasing reliability. Such maintenance actions cost money, the present value of which decreases when they are taken further away in the future due to the effect of a positive rate of interest. Maintenance actions thus should be taken as early as needed for reliability reasons and as late as possible for economic reasons. In this chapter CP is discussed as part of concrete structure maintenance and its effect on present and future costs is considered.

4.2 Maintenance strategies

As illustrated in previous chapters, corrosion of reinforcement causes damage to concrete and loss of steel cross section. Its effects may be cracking and spalling on relatively short notice and loss of structural capacity on the mid-to-longer term. All of these phenomena can be taken as reduction of performance of the structure. Inspection, protection and repair are actions that aim at keeping this loss of performance within acceptable limits. The baseline is the reliable safety as prescribed by laws, standards and owners' policies, the latter including serviceability issues. Following structural formalism (see e.g. Eurocodes), reliability is expressed by the reliability index β, which is related to the failure probability by a negative power relationship (for example, $\beta=4$ approximately corresponds to failure probability $P_f = 10^{-4}$).

Now, several strategies can be followed to keep the structure within safe and serviceable limits. Carrying out frequent inspections provides ample time for carrying out repairs. Postponing inspections may cause the need of repair on short notice. Taking (relatively inexpensive) preventative actions in an early stage may postpone full development of damage to much later; waiting until damage has developed on a large scale may require intensive and costly repair on short notice. Taking repair measures with a short life requires more repair cycles; protection with a long life may be necessary only once in the life of the structure. It will be clear that the total cost levels of each of these strategies over time will differ. Estimating the cost of various maintenance strategies may be termed maintenance cycle costing (MCC). An example of the latter type is briefly illustrated here.

For an example structure, two maintenance strategies are available: short life repair (strategy A) and long life repair & protection (strategy B). At some point in time, deterioration is observed and either strategy A or strategy B is applied. When maintenance is carried out, the reliability index β of the structure is brought to a relatively high level. In scenario A, it decreases rather rapidly down to the minimum level (due to degradation of the repairs, e.g. by corrosion reappearing) and a new round of repairs is necessary. In scenario B, the reliability decreases only slowly and remains above the target minimum level for a longer time. At some point, however, another round of application is needed, which may last until the end of the required life. Figure 11 illustrates the development of reliability for two maintenance options as a function of time. Input data are: the structure reaches its minimum reliability (β=2.0) in year 40; repair option A has a working life of 20 years, option B of 40 years; restoration due to both options brings the reliability back to β=4.0.

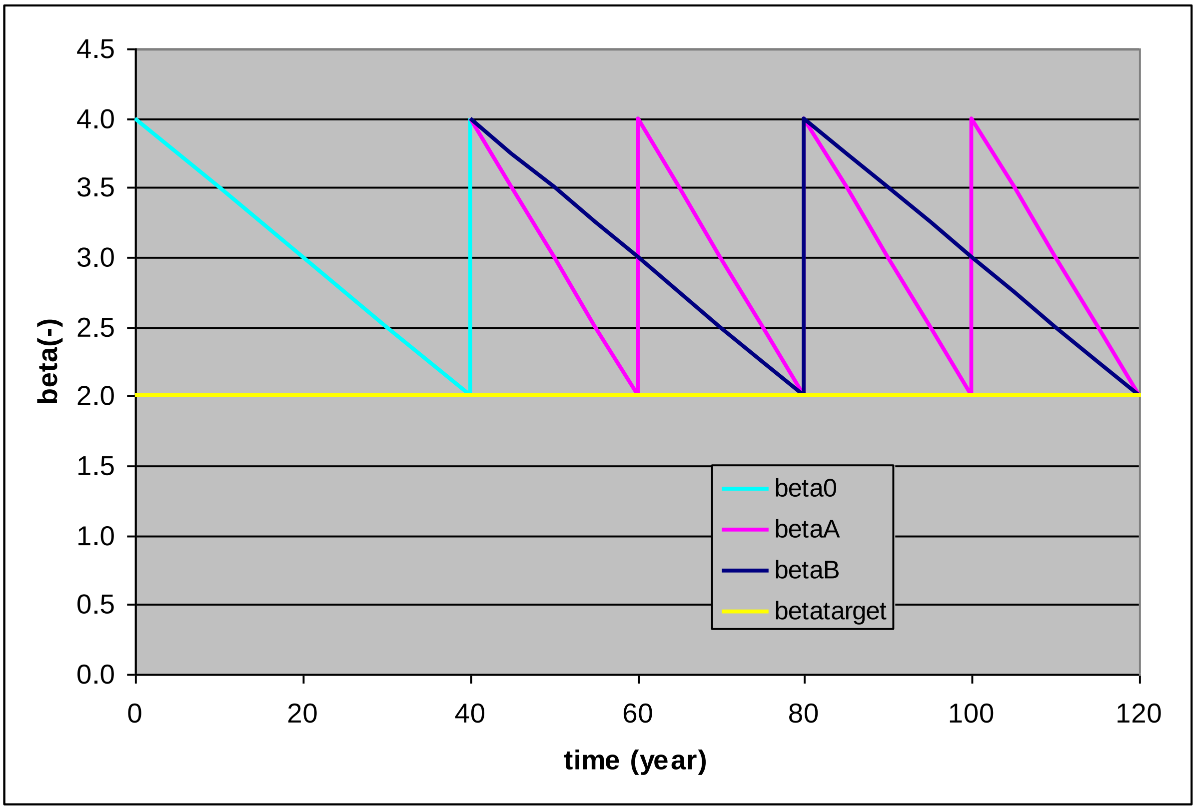

Figure 11 Schematic of reliability with time for a structure with two options, A and B, for repair and protection applied in year 40

For optimal economic decision making, the total cost of both options over the remaining life must be considered. This includes the rhythm of spending (option A every 20 years, option B every 40 years). However, spending money in the future should be expressed as (Net) Present Value, taking into account the (effective) rate of interest. In order to make these calculations more realistic, particularities of the two options are mentioned. Input data are: option A, conventional repair, working life 20 years, cost 100 monetary units (mu) when spent in year 1; option B, cathodic protection, life 40 years, installation cost 125 mu (in year 1), maintenance cost (control measurements) 1 mu per year; rate of interest 3%. Figure 12 presents the results. The cost of A or B in year 40 relative to year 1 is reduced by the effect of rate of interest (to about 30 and 40 mu, respectively). In the example, the total (cumulative) cost of option B is lower than those for option A, despite the fact that a single application of option B is more costly than for a single application of option A.

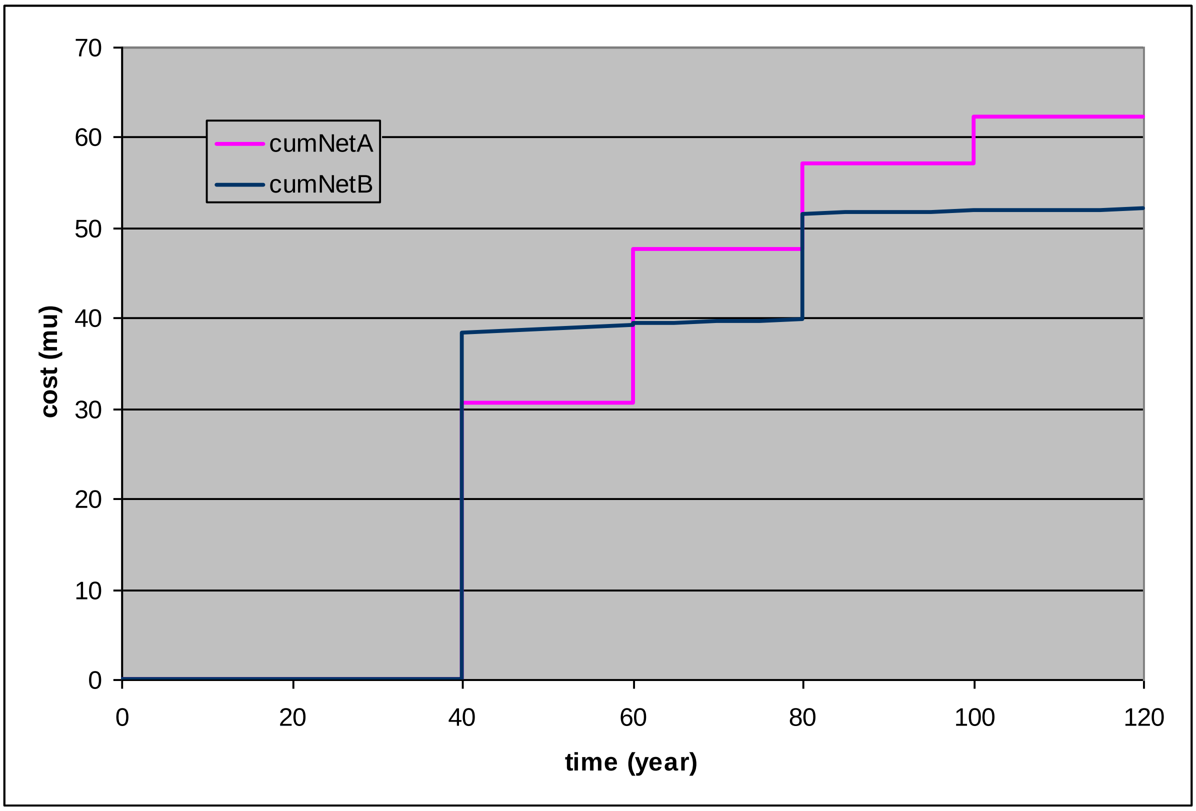

Figure 12 Schematic of total (cumulative) cost in present value development in monetary units (mu) for two repair and protection options, A and B; rate of interest 3%

Additional costs may occur that could be important. For repair or protection measures to be carried out, the traffic may have to be taken from a bridge. Consequently, the time it takes to carry out the repairs has some additional societal cost, for example due to traffic jams or for temporarily providing replacing roads. It is difficult to generalise such costs, which among others will depend on the location of the bridge, its importance for the local economy and the availability of alternative routes. This means that the time needed to carry out repair and protection work (during which the bridge is taken out of service) may have an influence on additional costs. If the execution time is different for two repair options, they have different additional costs. Another source of cost differences is the need for temporary support. Heavy conventional repair, where a considerable amount (depth) of concrete has to be removed, may necessitate temporary support with additional costs. Generally this is not needed with superficial repairs as needed with CP.

4.3 CP as a strategic option

Cathodic protection can be beneficially used in a strategic maintenance policy by considering the following scenarios.

4.3.1 CP with early detected corrosion

For this scenario, the starting situation and follow up actions taken are as follows:

- a bridge shows limited signs of corrosion (small amount of rust staining, localised damage), due to leaking drainage
- an inspection including cover depth and potential mapping is carried out to assess the extent of corroding reinforcement over the surface
- a limited structural analysis is carried out to locate structurally important reinforcement
- small repairs are carried out where needed
- leakage is stopped and water running across the concrete is diverted
- CP is applied to (potentially) corroding areas, in particular where structurally important reinforcement is located
- CP monitoring and visual inspection carried out every year.

Pros and cons for this case are:

PRO relatively low cost, reliability is kept at desired level, monitoring is included, short execution time;

CON costs are made relatively early in the bridge's life, cost of maintenance of CP system.

Concluding, this approach is highly recommended, as it prevents growth of damage to serious levels, maintains structural safety and reliability and avoids costly repair work in the future.

4.3.2 CP with fully developed damage

For this scenario, the starting situation and actions taken are as follows:

- a bridge shows widespread corrosion damage (cracking, spalling, rebars exposed)
- in-depth inspection is needed including cover depth and potential mapping, chloride penetration, carbonation, rebar section loss
- full structural analysis is carried out to evaluate the safety level and need for placing additional reinforcement
- extensive repairs are carried out
- leakage is stopped and water running across the concrete is diverted

- CP is applied to repaired parts and where future corrosion is anticipated
- CP monitoring and visual inspection carried out every year.

Pros and cons for this case are:

PRO costs are made relatively late in the bridge's life, reliability is kept at desired level, monitoring is included;

CON relatively high cost, situation tends to be urgent, cost of maintenance of CP system, longer execution time.

Concluding, this approach is not recommended as it involves relatively large and costly repair work. Unfortunately this scenario may be unavoidable when the condition of a structure has been neglected for a long time.

4.3.3 Cathodic prevention

For this scenario, the starting situation and actions taken are as follows:

- a bridge does not shows any signs of corrosion
- an in-depth inspection including cover depth mapping and chloride profile analysis is carried out to analyse the chances of corrosion in the future (preferably for a specified remaining life)
- a limited structural analysis is carried out to locate structurally important reinforcement
- no repairs are needed
- leakage and water running across the concrete are prevented
- CPre is applied to areas where corrosion is anticipated, in particular where structurally important reinforcement is located; such a CPre system can be light, as less current is needed and its distribution is favourable
- CPre monitoring and visual inspection carried out every year.

Pros and cons for this case are:

PRO relatively low cost, reliability is kept at high level, monitoring is included, short execution time;

CON costs are made relatively early in the bridge's life, cost of maintenance of CP system.

Concluding, this approach is only recommended for bridges that are critical in the main transport network, where high additional costs are involved (providing alternative routes, economic losses). It is not recommended for unimportant bridges or where alternative routes are available; the cost of CPre is then better saved for later.

4.3.4 Monitoring of CP

The need for monitoring of CP has been viewed as a disadvantage. On the short term this may look justified. On the other hand, however, the need for monitoring that comes automatically with CP should be seen as an advantage. Most importantly, it provides information on the condition of a structure that has already shown to be vulnerable to deterioration (corrosion damage). Thus, for a relatively low amount of costs, the absence of corrosion is watched at many points in the structure, precluded by the presence of the CP system and thus the structure's safety and reliability are guaranteed. In addition, the annual cost of monitoring and inspection is known beforehand and thus highly predictable.

4.4 Conventional repair

Conventional repair methods have many possibilities, as repairs can be made in small patches, over large surfaces, used to increase cover depth and for various types of defects, provide specific surface finishes, using various materials and application techniques. Limitations exist with regard to the effectiveness and durability of repairs. In particular their durability depends strongly on the quality of execution with regard to surface preparation, steel cleaning and curing of materials. Most importantly, as many execution issues require intensive labour, the quality of repair is directly related to the cost. In tendering situations, market pressure tends to reduce contract cost at the expense of execution quality and, consequently, durability. From a strategic point of view, the durability (the working life of a repair) is highly uncertain, the point in time when repairs have to be reapplied is uncertain and thus the cost over the remaining life of the structure is highly unpredictable.

4.5 Service life aspects

Some practical cases allow comparing the working life of conventional repairs and CP under similar conditions.

Case 1 is a pre-WWII bridge deck in Rotterdam. Two cycle tracks (North, South) had serious reinforcement corrosion and cover damage in 1986. The North track was repaired conventionally in 1986 by standards of the time. The South track was provided with CP (as the first system in The Netherlands) in 1987. In 1998, the North track had redeveloped corrosion and damage (causing serious serviceability problems) to such an extent that the whole slab had to be replaced (by a steel deck) at high cost. In 1999, the South track CP system was found not to be working properly anymore; reinforcement corrosion was virtually absent, however. Moderate maintenance was carried out, comprising replacing the anode connections, the reference electrodes and the power units. The cost of this operation was a small fraction of the original CP system cost. After this renovation, the CP system was again working properly. This case shows that after some 12 years, conventional repair had led to complete failure of the repaired cycle track, causing the need for replacement. CP had protected the structure, but was in need of maintenance that was possible at relatively low cost, without compromising safety and serviceability over that period. More details on this case are provided in [Schuten et al., 2007].

Case 2 concerns a building, but is nevertheless illustrative for the subject. It is an apartment block in Groningen, The Netherlands, built up of frames and slabs precast-on-site forming a three-story gallery, built in 1957. Apparently some chloride was mixed in the concrete (for accelerating hardening). Corrosion and concrete damage appeared in the 1980s, which was repaired using polymer mortars and a coating was applied to the whole surface. In a few years, corrosion and damage reappeared. In 1993, the owner wanted a real solution. Cathodic protection was applied to the underside of the gallery slabs using a conductive coating. For economic reasons (CP was more expensive than repair in those times), the frames (small columns and beams) were repaired conventionally using cementitious mortars. By 1999, rust staining and cracking had reappeared on the columns, while the slabs (with CP) were free of any signs of corrosion. An inventory of CP systems in 2004 revealed that the CP system still effectively working. Six similar buildings were provided with CP on slabs and columns/beams in the subsequent years. Details of the CP system have been published [Polder 1998].

Case 3 is another building, an apartment house on the seaside at Noordwijk, The Netherlands. Corrosion and concrete damage in cantilever beams (supporting gallery slabs) had been caused by penetration of chlorides from sea spray. 500 beams on the sea side had been provided with CP (using a conductive coating) in 1995. In 2004, this coating needed some maintenance. No damage to the concrete or the reinforcement had developed over that period. For economical reasons, cantilever beams with visible damage on the land-side were repaired and coated (with a conventional coating). Over ten years, repairs were needed every 2 to 3 years for individual beams that showed signs of corrosion. Although no precise numbers are available, this example shows that for the conventionally repaired parts, repeated repairs were necessary, which involved repeated costs of scaffolding and regular disruption to the inhabitants.

4.6 Economy of case studies

4.6.1 General

The costs of various repair and protection options are discussed for two cases. One is a bridge (substructure) that has been the ARCHES CP trial site in Slovenia. The other case dates back some 12 years and is a bridge in The Netherlands. For various reasons, the results presented should be taken with caution. They are by no means generic and may not be representative for cases that differ by some important factor. Furthermore, prices vary in space and over time. However, these examples are as real as possible and they could at least be viewed as typical for many bridges in Central European Countries. For both of these cases, maintenance cycle cost curves have been calculated following the principle explained above.

4.6.2 Slovenia, VA0128 overpass substructure

This is a recent example, on which CP trials are performed. It is described in more detail in Appendix B3. Most important here is that it concerns four cross beams, supporting a double bridge in the Ljubljana motorway system built in 1979. Each cross beam is supported by two massive columns. The cross beam cantilevers had developed serious corrosion and concrete damage after some 25 years, due to leakage of de-icing salt solution through the joint above, and more leakage from the (defective) bridge drainage system. The cantilever heads are c. 4.5 m long with a cross section of 1.5 m x 1.5 m. Cover depth to outer reinforcement was 25 mm on the average, with many values below 15 mm. The total concrete surface area that needed repair and/or protection was about 150 m^2.

Two options for repair and protection are compared:

a. full conventional repair including removal of concrete behind reinforcement and bringing concrete cover to required values over the complete surface (vertical and horizontal faces),

   a.1 without maintenance or need for later repairs

   a.2 with repairs again needed after some 13 years

b. superficial repair and installing cathodic protection on vertical faces; here three sub-options were considered:

   b.1. a conductive coating anode system (as applied in Test Area 2, see Appendix A1)

   b.2. a horizontal titanium strip system (as in Test Area 1)

   b.3. a titanium mesh and shotcrete overlay system (not tested).

The costs of various items were calculated on the basis of real costs for superficial repair and CP option b.1 as they were made in the CP trial; and they were estimated for full repair and CP options b.2 and b.3. Prices refer to 2008 and local costs for repair and European costs for CP, as shown in Table 3.

Table 3 Cost input for repair and protection options

Item (unit)	Cost (€)	Comment
Working platform (8 weeks) (per m^2)	22	All options
Concrete removal, mechanical, 30 – 40 mm (per m^2)	190	Full repair
Concrete removal, manual, 20 mm (per m^2)	125	CP
Thorough steel cleaning, mechanical (per m^2)	65	Full repair
Superficial steel cleaning, manual (per m^2)	44	CP
Application of anti-corrosion coating (per m^2)	14	Full repair
Application of repair mortar 40 – 60 mm (per m^2)	230	Full repair
Application of repair mortar 20 mm (per m^2)	115	CP
Construction site costs (-)	1150	All options
CP design (-)	1150	CP
Connections, RE's, cabling (-)	7000	CP
2 cabinets, 2 solar power units (-)	5500	CP
Start-up (-)	1000	CP
Conductive coating (per m^2)	100	CP
4 titanium strips, mortar dykes (per m^2)	95	CP
Titanium mesh (medium strength), shotcrete (per m^2)	205	CP

Table 4 Cost output for repair and protection options in net value in year 1; rate of interest 3%

Option	Net Cost in year 1 (€)	Comment
a.1 Full conventional repair	80,000	No maintenance assumed
a.2 Full conventional repair plus re-repair	50,000	Full repair again after 13 years
b.1 Conductive coating CP	63,000	Annual checks, working life 13 years, then partial recoating for € 25,000
b.2 Titanium strip CP	72,000	Annual checks, working life 25 year
b.3 Titanium mesh CP	76,000	Annual checks, working life 25 year
CP checking	1,000	Every year

Maintenance cycle costs were calculated over a period of 25 years using the output from Table 4, taking into account a net rate of interest of 3% and annual costs of € 1000 for CP monitoring. It was further assumed that the conductive coating CP system needs partial recoating after c. 12 years. In the base case (a.1), any maintenance cost for the full repair option was neglected. This is reasonable, as regular maintenance will probably be carried out only once every five years and will be cheap (visual inspection). However, there is a chance that the repairs will fail within 25 years, necessitating new repairs. Estimating the probability is difficult. Following the results of CONREPNET, 50% probability of failure after 10 years [Tilly and Jacobs 2007], seems pessimistic in view of the thorough character of the repairs. A short working life of the repairs would significantly increase the life cycle cost, obviously. Having to repeat the repairs after 13 years would again cost € 80,000 (gross). Such an expense in year 13 corresponds to € 50,000 net value in year 1. If this would happen with a probability of 50%, the risk would be considerable, namely 0.5 * € 50,000 = € 25,000 net. The maintenance cost curves for the four options are shown in Figure 13.

It can be clearly seen that over the first 12 years, CP with a conductive coating is the most economic option. From then on, things become more uncertain. If full repair would last 25 years, it is less expensive than CP. If there is a probability of 50% of having to repeat the repairs, it is the most expensive option. As a conductive coating CP system may require repainting after 13 years, it becomes slightly more expensive than CP with titanium mesh. If both full conventional repair and coating CP require serious maintenance as indicated, CP with titanium mesh becomes the most economic solution over 25 years.

The conclusion from this exercise is that predicting life cycle cost over 25 years is difficult, due to uncertainties in the working life of individual options. It remains, however, that CP offers a high degree of certainty with respect to reinforcement protection and thus, the absence of corrosion, damage to concrete and steel cross section loss.

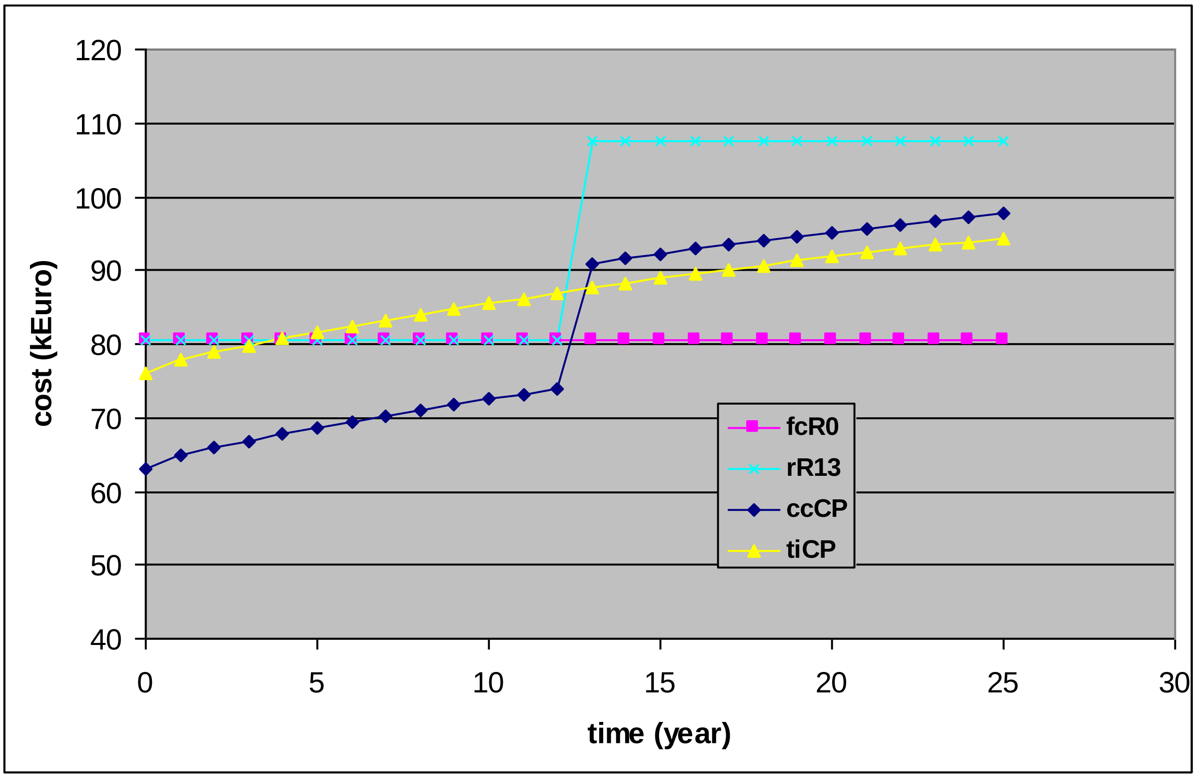

Figure 13 Maintenance cost curves for option **a.1** (full conventional repair without new repairs, legend *fcR0*) and **a.2** with repeated repair after 13 years (*rR13*); **b.1** (conductive coating CP, *ccCP*) and **b.3** (titanium mesh CP, *tiCP*)

4.6.3 The Netherlands, two bridges near Barneveld

General

This case concerns two bridges consisting of precast concrete beams supported by five piers. Each pier consists of two precast columns and a precast cross-beam. Both bridges (built in 1965) span a motorway and a railway track, with spans measuring 16 to 19 m. The cross-beams are post-tensioned with prestressing bars in grouted ducts. In 1993 corrosion of the skin reinforcement and spalling of concrete cover was found, apparently due to ingress of de-icing salts through leaking joints in the bridge deck, as illustrated in Figure 2 (see section 2.2 above). The cover depth was about 30 mm and near corrosion spots, the total chloride content varied from 0.7 to 2% by mass of cement. The complete substructures of the two bridges were replace with new reinforced precast concrete elements in 1996. The actual cost of this replacement was used as input for the study. The cost of CP as an alternative for replacement was estimated by designing and specifying a CP system, pretending it was still possible to install CP. Two concrete repair companies with field experience in CP were invited to give a detailed estimate of the cost, including regular control activities and a prediction of the service life of the system. Supervision by an experienced consultant was included. The total life cycle cost was estimated by calculating the net present value of installing the system and its maintenance for periods up to 100 years. In the original case study the costs were specified in Dutch guilders (NLG). When the Euro was introduced, one Euro equalled 2.20 NLG.

CP design

The concrete surface area of beams and columns were about 18 m^2 each, the steel surface 7 m^2 in the beams (excluding post-tensioning) and 23 m^2 in the columns. Each system had a total surface of about 260 m^2. Each bridge would have one rectifier with 5 A capacity. The anode would be a conductive coating with embedded silver ribbons for primary anodes, provided with a cosmetic topcoat (Polder 1997). Because both companies had ample experience with such a system, the cost information may be considered quite reliable. As an alternative anode system, activated titanium mesh with a shotcrete overlay was considered. The normal monitoring by depolarisation was included, also involving monitoring the safety of the prestressing steel using true reference electrodes embedded near the prestressing ducts.

Boundary conditions were safety for workers and vehicles and minimum disturbance of traffic. Due to the relatively short period of operation, traffic control was possible with mobile vehicle barriers. Special measures were foreseen to guarantee safety with regard to the railway line.

Cost estimate

Not only the initial cost (installing the CP system), but also the cost of annual checks and maintenance to the anode system were considered. Annual checks using a datalogger and telephone connection are more economic because the site must be visited only once a year; on the other hand, the cost of the datalogger increases the initial cost. The cost of maintenance was estimated, assuming a working life of 13 years. Subsequently, the conductive coating is removed and replaced. Other parts to be replaced then are: decay probes, rectifier and datalogger. Every 26 years, together with replacing the anode, the electrodes near the prestressing and the more permanent parts as cable ducts are replaced as well.

For a system based on titanium mesh and shotcrete, giving a firmly based life expectancy is difficult. It may be assumed that where locally current densities are highest, some degradation of the overlay may occur. For the calculations it was assumed that after 26 years, about 25% of the overlay has degraded and has to be replaced. Every 13 years, rectifiers and other small equipment are replaced as with the conductive coating system.

Maintenance cost (inspection, repair) of the replacement option were neglected (set to zero). This is obviously an underestimation of the real maintenance cost over 100 years.

Results

The cost of replacement of the two substructures was NLG (Dutch guilders) 810,000. The cost estimates for CP obtained from the repair companies were analysed and a synthesis made. The results are shown in Table 5. The initial cost of a titanium based system was estimated at NLG

642,000 and maintenance (25% replacement) at NLG 194,000. All costs over the life cycle were converted to net present values. Some of the input factors were varied:

- the net rate of interest was assumed to be 4% (7% gross interest - 3% inflation); the effect of 3% and 5% was calculated

- the service life of the anode was assumed to be 13 years; the effect of 11, 15 and 18 years service life was also calculated

- the effect of manual control versus remote control was studied.

For the various combinations, the initial cost of CP is lower than replacement of the substructures. For every year of service, a small amount is added (annual checks) and every 13 (11, 15, 18) years, a larger sum is added to the capital that has to be reserved. At a certain moment in time, the cost of CP is equal to the cost of replacement and beyond that point in time CP is more expensive than replacement. This "year of cost balance" is shown in Table 6. The development of the present values of the various options is shown in Figure 14.

Table 5 Best estimates of cost of installing, checking and maintenance of two CP systems (bridge substructures) with conductive coating including supervision, prices 1996

	Cost in Dutch guilders (NLG)	Frequency
Initial CP installation, 2 * 260 m^2, remote control	472,000	-
Protection checks	3,000	3 monthly
Maintenance: replace anode coating and some parts[1]	276,000	year 13, 39, ..
Maintenance: replace complete system[2]	308,000	year 26, 52, ..

Notes: [1] includes anode coating, top coat, decay probes, rectifier, datalogger

[2] includes above plus: true reference electrodes and all cabling

Table 6 Results of net present value calculations of the cost of CP compared to replacement of two bridge substructures

Input variables	CP more economic
1. conductive coating, 4% interest, maintenance interval 13 years, remote control	up to 38 years

2. as 1, manual control	up to 25 years
3. as 1, net interest 5%	up to 51 years
4. as 1, net interest 3%	up to 25 years
5. as 1, maintenance interval 11 years	up to 21 years
6. as 1, maintenance interval 15 years	up to 44 years
7. as 1, maintenance interval 18 years	up to 89 years
8. titanium mesh and shotcrete overlay, 4% interest, maintenance interval 26 years (25% of surface), remote control	up to 51 years

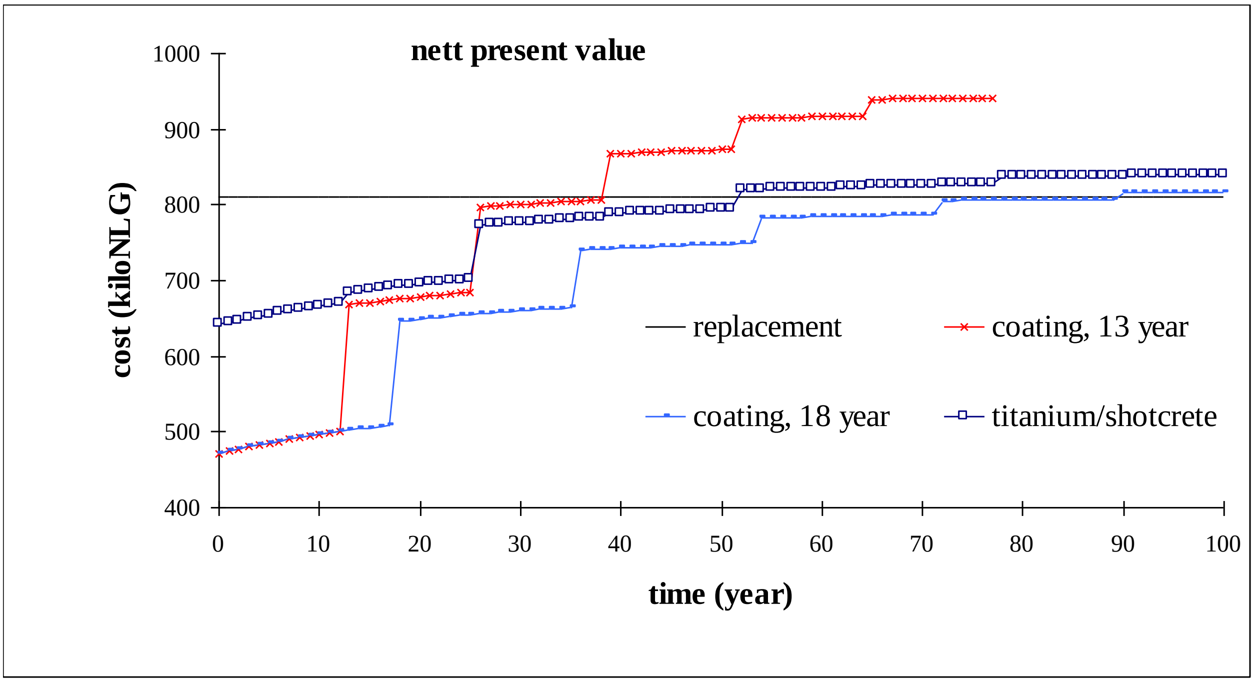

Figure 14 Development of net present value cost as a function of time for applying and maintaining CP for two bridge substructures (conductive coating and titanium mesh/shotcrete), compared to replacement

With respect to this example, it should be stated that in over ten years that have passed since it was analysed, prices for CP systems have decreased as a result of technical and market developments. Without further detail, it is strongly believed that in 2009, the case for CP would be economically even more favourable than outlined above.

For the example of two bridge substructures and depending on the assumptions, CP was shown to be more economic than replacement for periods between 21 and 89 years. The variation in this “moment of break-even” is caused by variation of the net rate of interest and the actual service life of the anode system (necessitating anode maintenance). The central variant (most realistic input for service life and rate of interest) shows that CP with a conductive coating anode is more economic than replacement of the two substructures up to 38 years of service life. CP with a titanium mesh anode and shotcrete overlay is more economic than replacement up to 51 years. A conductive coating is more economic than titanium mesh until 25 years service. For the required frequency of checks (four per year), and taking into account replacing the datalogger every 13 years, remote control is more economic than manual control.

5. CONCLUSIONS

Cathodic protection (CP) of reinforcing steel has been applied to concrete structures with corrosion damage for over 25 years. World wide experience shows that CP prevents further development of corrosion damage in a reliable and economical way for a long time, provided that the CP system is designed, executed and maintained properly.

Since the 1980's, CP has been applied to buildings, marine structures, tunnels and bridge decks and substructures in the US, Europe, the Middle East, Asia and elsewhere in the world. New anode materials have become available, of which in particular activated titanium and conductive coatings have proved their good performance over more than two decades.

CP of concrete structures has been standardised in the US for more than about 20 years and in Europe since the year 2000.

As a wide variety of anode systems is available with a good track record, tailor made solutions can be provided for every type of structure.

The principle of CP is active and permanent intervention in the corrosion process. Therefore, monitoring is an essential part of operating a CP system. The main advantage is that monitoring proves the absence of corrosion on a regular basis.

Designing a CP system for a particular structure requires that proper information is available on the structure, in particular about the extent of damage and the layout of the reinforcement.

CP can be applied to both reinforced and prestressed concrete structures. Application to post-tensioned structures can be done on a routine basis. It is recommended to perform a trial in case of slender (low cover) pre-tensioned elements. In all cases, very negative potentials of stressed high strength steel should be avoided, as they may cause loss of ductility. Dedicated monitoring of pre-tensioned steel or post-tensioning ducts in prestressed structures is needed to avoid overprotection. Numerical modelling of CP has been demonstrated to be a useful tool, both for predicting CP operation in general and the safety of potentials of prestressing steel in particular.

CP systems have been shown to have a long working life. As growing experience has shown, typical life of conductive coatings systems is well over 10 years. The life of typical activated titanium (mesh, strip) systems is at least 25 years and probably much more. Corrosion of reinforcement will be completely absent during this period and probably many more years after the end of the working life. The long life of CP is in strong contrast to the life of conventional repairs. Both European studies and typical examples have shown that conventional repairs may have short lives of as low as five to ten years. This is, among others, due to market pressure on repair prices, which results in poor execution quality. In any case, the working life of conventional repairs is highly uncertain and the absence of corrosion and potentially severe loss of reinforcement cross section cannot be guaranteed.

CP fits into a maintenance strategy in various ways. Several scenarios are possible. Both from technical and economical points of view, the recommended scenario is that corrosion and concrete damage are detected in a relatively early stage, and subsequently CP is applied. Waiting until damage has become extensive is an unfavourable scenario. Applying CP before any damage is detected is also less favoured. However, in individual cases following either one may be unavoidable or favourable, respectively.

Based on case studies, the cost of CP over the (remaining) life of a structure has been systematically investigated and compared to other options. The cost over e.g. 25 years appears to depend on the individual case and the expected working life of repairs. CP was found to be more economical than replacement of bridge substructures. Taking into account the uncertain working life of conventional repair, CP was more economical than conventional repair. From a technical point of view and with regard to structural safety, CP offers the most reliable solution to structures that are deteriorating due to reinforcement corrosion and in particular bridges under de-icing salt load.

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Appendix A Technical descriptions

A1 Cases

Two CP trials were foreseen in ARCHES, one in Slovenia and one in Poland. The trial in Slovenia was installed in 2008 and measurement results were collected until early 2009. The trial in Poland was realised early in 2009; no results were available within the period of reporting.

A1.1 Slovenia

CP was applied to part of the substructure of a bridge near Ljubljana, Slovenia. The case was used for cost calculations in chapter 4.6. Details on the bridge and the corrosion damage observed are presented in Appendix B3. From March 2008 until August 2008, the evaluation of the corrosion damage, the repair works and the application of the CP system were performed. Figure A.1.1 presents a view of the damage in one cantilever head and a detail of exposed reinforcement. Figure A.1.2 presents a map of corrosion potentials on the vertical surface of a cantilever head. Further investigations showed both chloride ingress and carbonation had caused corrosion, in combination with local low cover to the steel.

Figure A.1.1: The damaged bridge substructure in Slovenia

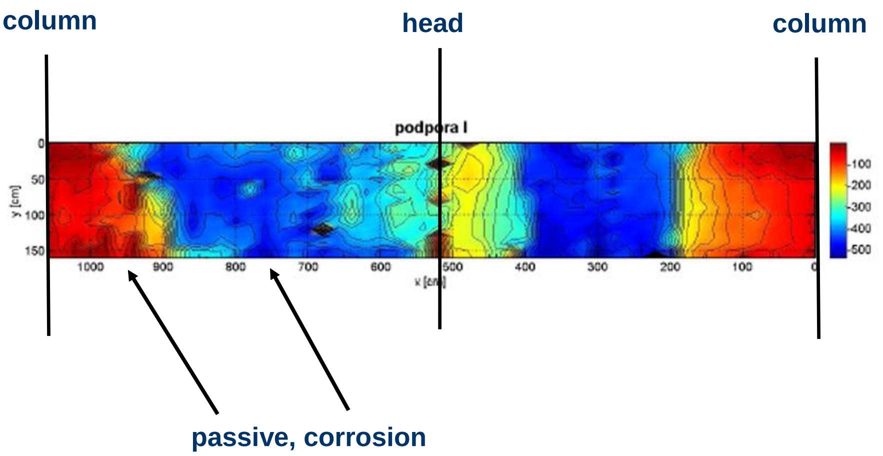

Figure A.1.2: Potential mapping of Slovenian bridge; blue indicates active corrosion, red passive conditions

Three different CP systems were applied, the “standard” anode (CP area 2), the “smart strip” anode (CP area 1) and the “minimal” anode (CP area 3), leaving one cantilever unprotected as a reference field (area 4), as illustrated in Figure A.1.3. A solar panel was installed for power supply.

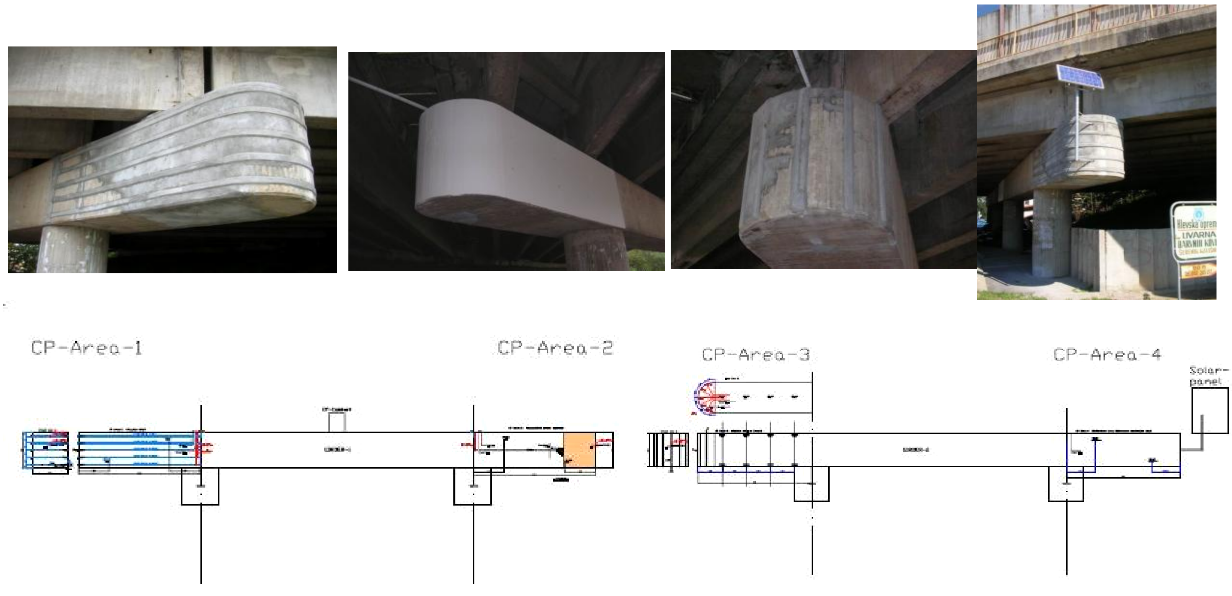

Figure A.1.3: The CP trial application in Slovenia

The results can be summarised as follows. In Test area 2 with conductive coating (AHEAD), CP works fine and can be modelled well (for a summary see Appendix A.2). In Test area 1 with horizontal titanium strips in mortar dykes over the whole beam surface, CP work well and modelling is possible. In Test area 3 with vertical strips only on the beam head, protection current does not reach the steel in other parts of the beam. Early 2009, in both Test areas 1 and 3, the mortar dykes containing the anode strips had become disbonded to a certain extent due to poor surface preparation.

Anticipating the ending of ARCHES, the following recommendations are made with regard to maintenance of the CP trial system:

- monitoring of currents, voltages and depolarisation should be continued, with a preferred interval of three months, including checking the solar power system
- depolarisation testing should be extended over one week (measuring potentials every day) at least once every year; switch-on current after seven days should be measured
- the system should be inspected with regard to leakage; major leakage should be stopped
- top and bottom surfaces should be checked for carbonation every year
- top and bottom surfaces should be provided with a normal concrete coating if carbonation becomes significant (say more than 5 mm) or if stopping leakage is not possible; both to prevent re-appearance of corrosion.

Measurements will be carried on by ZAG beyond the running period of ARCHES.

A1.2 Poland

The CP trial was installed on a motorway bridge near Warsaw, Poland, in late 2008 – early 2009. The case involves corrosion damage due to leaking drains as shown in Figure A.1.4. Preparing for CP trial application, the reinforcement was exposed; a repair mortar was applied and titanium strip anodes were installed as shown in Figure A.1.5 and subsequently overlaid with mortar. The system had not been activated yet at the time of closing this report.

Figure A.1.4 Corrosion damage due to leaking drain system in motorway bridge near Warsaw

Figure A.1.5 Titanium strip anodes installed before overlaying on side surface; three strips have already been overlaid in the bottom face

A2 Summary of Numerical calculations for CP trial in Slovenia

A2.1 Principle

For numerical analysis of a CP system, a finite element computer model is made of a typical part of the structure to be protected, which ideally includes the (concrete, steel) geometry, polarisation processes at the anode and at the steel and the ohmic drop in the concrete. Using such a model, current densities and potentials can be calculated at every point in the modelled structure. They can be used to verify the spread of protection for various anode configurations and to predict local polarisations that can be tested experimentally. The methodology is described elsewhere [Polder et al., 2008a,b; 2009].

A critical problem in modelling of CP is formed by the input variables. In particular the corrosion current density of the steel is an important variable. This parameter is strongly influenced by the flow of protection current. Once CP has become effective, all steel will be passivated and only passive steel should be modelled to predict the distribution of current and polarisation. However, at start up some steel will be actively corroding. Consequently, there is a period of transition from active to passive. A conceptual model was set up to predict the length of this transition period. The model results were compared to start up data from the test site in Slovenia, which showed satisfactory agreement [Polder& Peelen, 2009]. It appears that passivation took place in about four hours. Pending further confirmation, this would mean that steel in concrete under CP can be modelled as being passive.

Input from a structure can be obtained in the form of geometry (dimensions of elements and steel cover depth), steel potential mapping and resistivity measurements. Input for other electrochemical parameters is hard to obtain for individual structures. Consequently, educated guesses have to be made and/or literature values have to be used. The following text provides details on how the Slovenian CP trial was modelled.

A2.2 Summary of Modelling the Trial CP system in Slovenia

As full reporting on this work is outside the scope of this document, a summary of steps taken and results obtained is given here, including comparison the data measured on the trial CP system. This report also gives an impression of the input data that can be obtained on site.

The trial CP system in Slovenia comprises three test areas, TA1, 2 and 3, as shown in Appendix A1.1. The basic geometry consist of 5 m long (cantilever) beams, 1.5 m x 1.5 m cross section, with rounded head. Reinforcement is 16 mm diameter (vertical faces) and 16 and 32 mm (horizontal faces) bars near the surface, with a small amount of bars inside the cross section. Concrete cover to outer bars is 25 mm. Rebars are modelled as two adjacent layers, one at 25 mm and the second at 41 mm depth (front face of bars) and some bars inside, as shown in Figure A2.1.

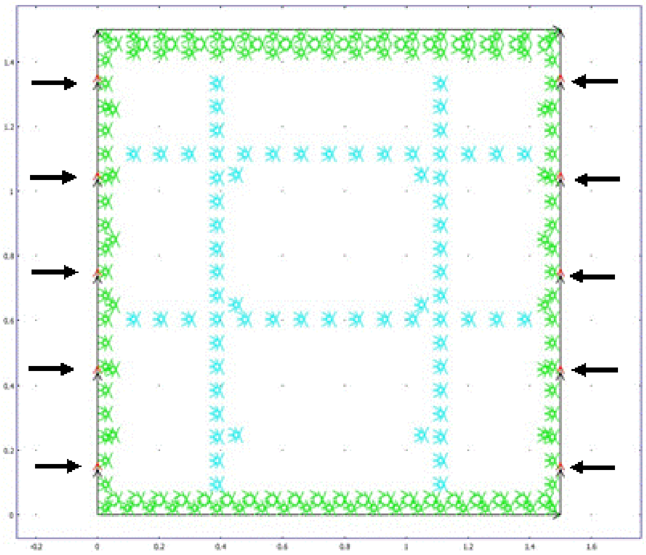

Figure A2.1 Typical model for vertical cross section with rebars indicated (here TA1 with 5 Ti* strips on each side face indicated by arrows)

The initial input used consisted of data measured on site (steel potentials, concrete resistivity), data from literature (cathodic behaviour) and educated guesses (anodic behaviour).

First, the electrical resistance of the Test Areas was calculated. It appeared that measured CP system resistances (both in August 2008 and March/April 2009) were much higher than those calculated. Apparently, the input had to be corrected. A value of 2400 Ωm was necessary, as opposed to values of about 600 Ωm measured on-site in November 2008. The difference is probably related to the applied wetting before measuring and/or rather strong seasonal changes. TA3 showed a high resistance in April 2009 due to disbonding.

Then, the total current in each TA was calculated, starting with TA2. Steel passivation was presumed, which is justified as CP obviously worked well, demonstrated by depolarisation values over 200 mV in 4 hours. Initially, the calculated current was higher than measured for the applied 1.2 V driving voltage. Using the higher resistivity value and a more negative steel potential reduced the current, but it remained higher than measured. The next step was considering the passive (cathodic) "corrosion" current density. Its value was reduced based on the effect of CP, which will presumably level out potential differences across the steel surface, reducing the current density. Another step was considering the anode polarisation (the anode/concrete potential drop). This polarisation contains contributions for hydroxyl ion oxidation (a standard value for the reaction thermodynamics plus a current dependent value for its kinetics) and a pH dependent term. With the reduced cathodic current density and an anode polarisation of 0.6 V a realistic current of 10 mA was obtained for TA2 (carbon based conductive coating anode), compared to a measured value of 8.5 mA.

Based on these modified input values, local polarisations were calculated. Results can be compared to measured depolarisation values for locations where reference electrodes were placed. For locations near the anode (vertical faces), 350 mV polarisation was calculated. In the field depolarisation values found 24 hour after current switch-off (March 2009) were between 180 and 240 mV. It is likely that depolarisation continues after 24 hours, so the measured values are pessimistic. Considering this, the agreement between calculated values and measured is good. Depolarisation testing over longer periods (say a week) would be useful to improve the comparison. Results are graphically represented in Figure A2.2.

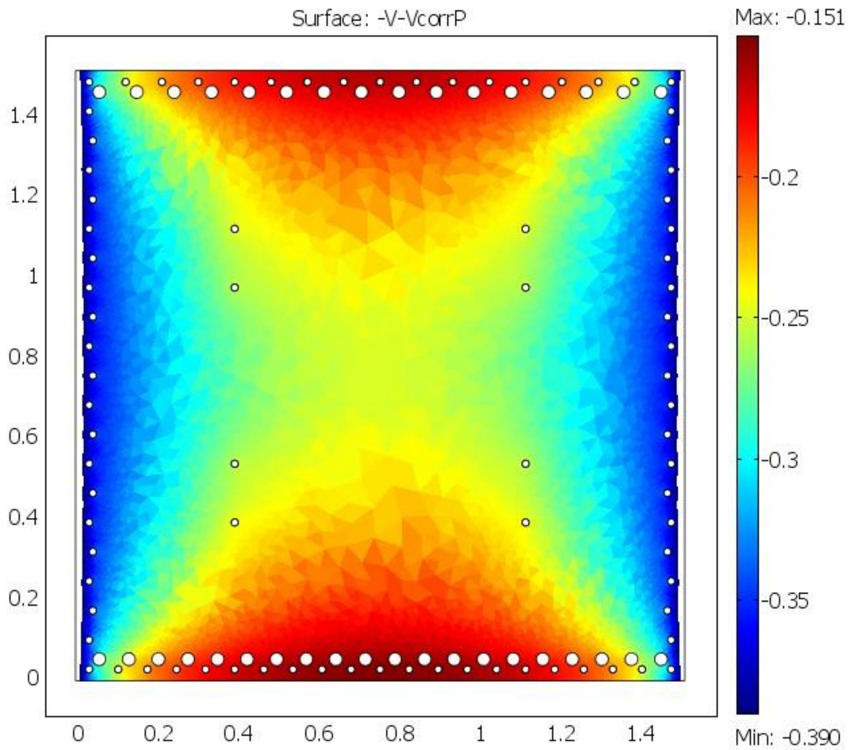

Figure A2.2 Polarisation distribution in TA2, input i0 0.01, rho 2400, Ecorr -.185, dEa 0.6

Local current densities were checked. It appears that steel in vertical faces in TA2 (just below the anode) received 0.6 mA/m^2 of steel surface area. Assuming all steel was passivated, bars in the middle of the top and bottom faces (with the highest distance to the anode) received 0.05 mA/m^2. Considering that all damage in these faces was repaired with alkaline mortar, the pH would be about 13 and CP-assisted passivation would be probable at such low current densities. If the pH would drop due to carbonation (say to 9) this current density would not be sufficient to achieve protection. It means that the concrete in the top and bottom faces should be protected from carbonation. Results are graphically represented in Figure A2.3.

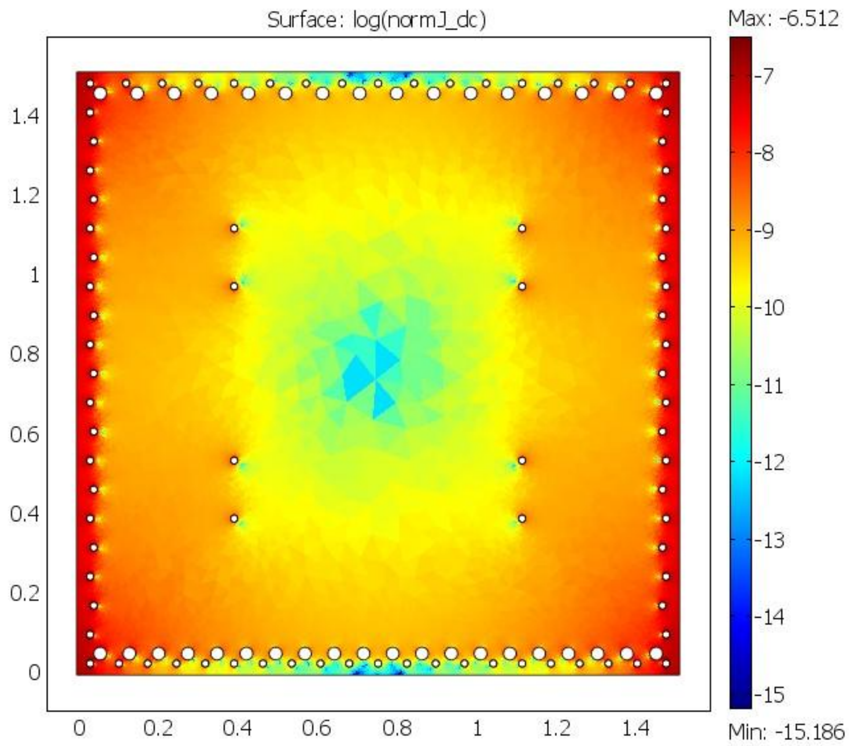

Figure A2.3 Current distribution, TA2

The adjusted input was also applied to modelling TA1. For a total voltage of 2.5 V, the measured current was slightly higher (46 mA) than the calculated current (35 mA). A factor here may be that the anode polarisation of activated titanium is lower than for carbon; adjusting would increase the current. Calculated polarisations were higher (190 to 540 mV) than measured depolarisations (up to 270 mV in March). Agreement is reasonable to good, also realising that longer depolarisation measurements may produce higher values. Interestingly, there does not seem to be an effect of the partial disbonding of anode strips.

TA3 was modelled using the adjusted input, for 4.0 V driving voltage, in a horizontal plane just below the top surface. Assuming CP-assisted passivity of all steel bars, a realistic current was found of 10 mA, the same value as measured. Polarisations of steel in the rectangular part of the beam (so away from the rounded head where the anodes are located) were between 30 and 190 mV, as may be seen in Figure A2.4. In the field, almost zero depolarisation was measured. Apparently the assumption of passivity is not correct. For a normal passive cathodic current density, polarisations in the beam were between 50 and zero, which corresponds better with measurements. Local steel current densities were very low. These results correspond with expectation, namely that it is impossible to protect steel with anodes more than one meter away. This shows that the modelling works for CP current densities greater than the normal cathodic current density.

To study the possibilities with TA3, the modelling voltage was increased to 10 V. Local current densities and polarisations still were below acceptable values. This shows that even with quite high voltages, it is impossible to protect steel at one meter or more from the anodes.

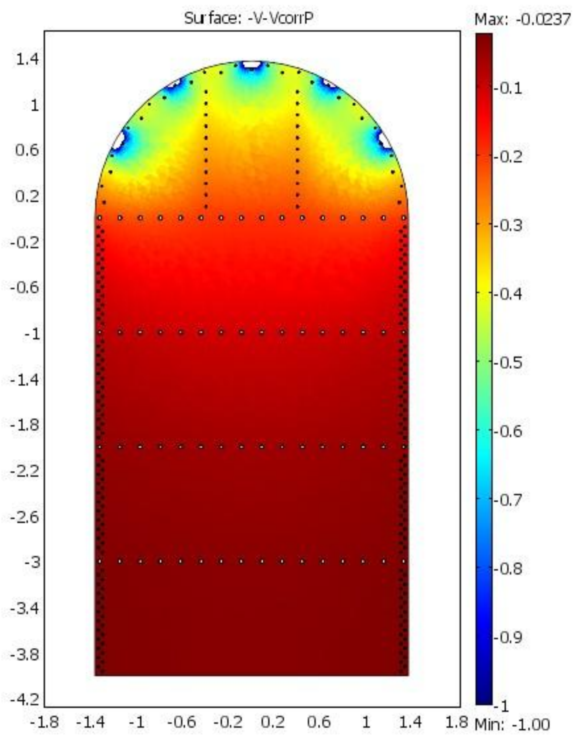

Figure A2.4 TA3, distribution of polarisation at 4.0 V

Appendix B Typical problem areas in bridges

This Appendix is intended to show some typical examples of bridges or parts of bridges that suffer reinforcement corrosion.

B1 Bridge over River Dommel, Den Bosch, The Netherlands

Structure description and age

Bridge over River Dommel, Den Bosch, The Netherlands, two parallel post-tensioned bridges, each 14 m wide, built mid 1970's

Type and size of area affected

Underside of edge beams below expansion joints between main deck and abutment. Heavy corrosion and concrete damage (spalling) in a zone of about 0.5 m wide. Total affected area 4 x 0.5 x 14 m is c. 28 m^2.

Cause of deterioration

Leakage of expansion joints caused chloride (de-icing salt) contaminated water to run along vertical face and underside of edge beam. Strong chloride penetration and subsequent corrosion of rebars occurred in underside.

Age when discovered as a problem and further relevant information

First corrosion damage was discovered about 1990. Conventional repairs were not successful: heavy corrosion and concrete spalling were again present in 1996. For sketch see Figure B.1.

Options considered for rehabilitation

First option: breaking out all contaminated concrete (behind rebars) and conventional repair, which was technically possible. This option required temporary lifting and support of structure, which would seriously impair traffic and thus was unacceptable. It was also considered that the working space was very low (c. 0.5 m), thus making proper cleaning of steel and application of shotcrete difficult. In stead, it was preferred to cast new concrete from the top side of the deck (through drilled holes), which would also disturb traffic. As a side effect, traffic safety measures (both for ensuring safety of traffic and workers) would be costly. In addition, leaking joints should be replaced, also requiring work on the top side of the deck.

Second option: cathodic protection of damaged areas plus an additional zone of 0.5 m wide, removing only spalled concrete and repair open spots. Anode would be a conductive coating over a 1 m wide zone along the edge beams. Monitoring would include decay probes near reinforcing steel and true reference electrodes near post-tensioning steel/anchors. In addition, leaking joints should be replaced, requiring work on the top side of the deck. Lifting and supporting the deck would not be necessary for the CP work.

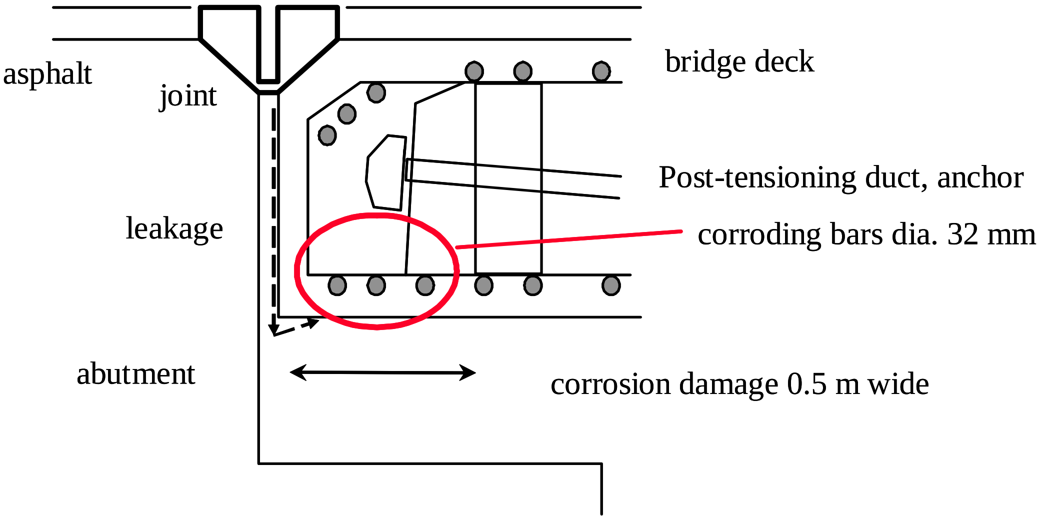

Figure B.1 Sketch of cross section Dommelbruggen, Den Bosch, NL

Measures taken

The owner decided to apply CP (second option) in 1996. In view of the cost of traffic safety measures, CP was more economic than conventional repair. Figure 7 of the main text of this document shows part of the structure after CP application.

Concluding remarks

Over one year of operation, the CP system showed to work well in wetter parts of the year. In drier parts of the year, the current was very low (mainly due to a restricted maximum voltage applied). One of four new joints was leaking for some time; CP current in that part was considerably higher than in other parts.

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B2 Lucko Overpass, Zagreb, Croatia

Type and brief description of structure, year of construction

Two largest bridges in junction Lucko, Overpasses 8 (138 m) and 9 (550 m) close to Zagreb, Croatia, Year of construction 1977. Type of construction system: monolithically connected columns and column heads (cantilevers) with box-cross section post-tensioned in longitudinal and transversal directions, supporting bridge spans made of four prefabricated prestressed ‘I’ girders (30 m span), cross girders and a deck made of precast slabs and cast in situ top layer (total thickness 100 – 120 mm)

Type and size of area affected

Area affected:

- Overpass 9: 11 columns → concrete damage over 10% of main girders, 30% of cross girders and 25% of cantilever head beams: 1600 m^2.
- Overpass 8: 4 columns → 450 m^2.

Cause of deterioration

The specific problem lies in reinforcement corrosion and concrete damage of the main load bearing system at its most delicate location, the support areas of the main girders, where they rest on elastomeric bearing pads as part of the cantilever head.

The unforeseen access of de-icing salts to the concrete has caused reinforcement corrosion and cracking and spalling of concrete. The deeper cause lies in the applied structural system, which is a set of Gerber girders as primary main load bearing system with a continuous deck slab made out of precast slabs as secondary system. The concept was based on one assumption that proved to be faulty. This assumption was: it is appropriate to lower costs by not installing rail expansion joints on the discontinuities in the main bearing structure. Thus, all elements above the main girders (deck slab, pedestrian ways, cornices, asphalt pavement, curbs and all fences) were carried out in continuity over all spans with only two additional rail expansion joints within the overpass (two were installed above the abutments), [1].

Figure B.2: Support areas on the cantilever head beams and main girders

Age when discovered as a problem and further relevant information

An intensive condition assessment was carried out in 2007-2008, at 30 years age.

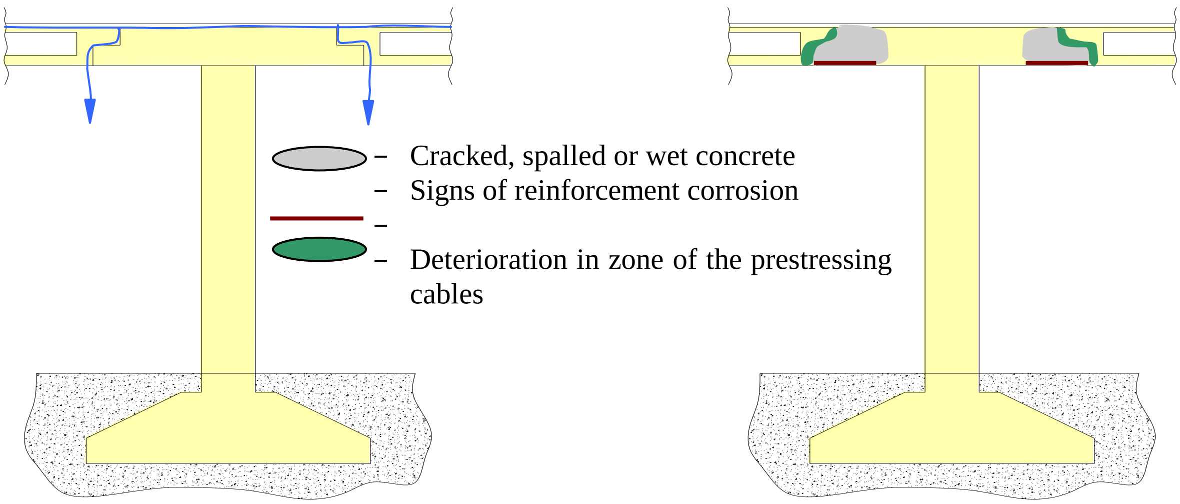

Figure B.3 Routes of salt water penetration to support areas and observed deterioration on these locations

Options considered for rehabilitation

Two alternative options were proposed to the Owner during 2008 [2, 3] and another alternative was discussed within ARCHES in 2009:

(1) Alternative 1 (10 year durability)
- Replacement of pavement elements (pedestrian ways, cornices, deck asphalt surfacing together with waterproofing, curbs and all equipment),
- Repair of levelling course concrete on precast deck slabs, repair of lateral and longitudinal joints between precast slabs that are leaking,
- Embedding of rail expansion joints on all locations of discontinuity of main bearing structure,
- Local repairs on main girders, head beams, columns and abutments.

(2) Alternative 2 (50 year durability)
- Construction of a new overpass which will fit in existing road routes and within the junction or out of it (integration of the new structure with structures of other overpasses),
- Removal of the entire existing overpass 8 and 9.

(3) Alternative 3 (at least 25 years of durability)
- local repair of damage
- cathodic protection of support areas and parts of cantilever head beams

Measures taken

None until February 2009

Concluding remarks

After 30 years of service and multiple increase in traffic loading of Junction 'Lucko', the need for serious rehabilitation works occurred on all structures (11 overpasses). The worst situation is on Overpasses 8 (138 m) and 9 (550 m) where serious problems lie in the particular structural systems and deterioration that occurred on critical load bearing parts of structural elements [4]. The owner still has not decided about the repair method.

Reference(s)

[1] Civil Engineering Institute of Croatia (IGH, 1977), Design projects of 11 overpasses in Junction “Lucko”, (in Croatian).

[2] Civil Engineering Institute of Croatia (IGH, 2007), Results reports about the investigation works on Junction Lucko structures, (in Croatian).

[3] Civil Engineering Institute of Croatia (IGH, 2008), Repair alternatives on Junction Lucko structures, (in Croatian).

[4] I. Stipanovic Oslakovic, S. Skaric Palic, A Balagija & K Mavar, Analysis of Durability & Life Cycle Costing of Different Repair Methods Applied on 11 City Overpasses, Structuralk Faults and Repair 2008, 12th International Conference, Edinburgh, UK, 10-12 June 2008.

[5] K. Mavar, S. Skaric Palic & A. Balagija, Condition assessment on Junction “Lucko” overpasses of Zagreb bypass, 2nd International Conference on Concrete Repair, Rehabilitation and Retrofitting (ICCRRR 2008), Cape Town, South Africa, 24 - 26 November 2008.

B3 Underpass in motorway Ljubljana – Koper, Slovenia

Type and brief description of structure, year of construction

The VA0128 overpass on the Ljubljana - Koper highway is located in the suburban area of Ljubljana; it was built in 1979. The substructure of this bridge was chosen for the trial application of a smart CP system (see A1.1).

The overpass consists of two parallel structures, one for each driving direction. Each has a total span of 89.34 m with two supports (spans 25.67 + 38.00 + 25.67 m) and is about 20 m wide (total width 40 m). The bearing system of each structure (substructure) consists of 4 piles, two 2-pier frame intermediate supports per bridge. The superstructure consists of 7 main pre-tensioned precast simply supported girders and an on-site cast concrete deck. The main girders are interrupted at the intermediate supports, but the concrete deck is cast continuously, without a construction joint over the two intermediate supports. The support beams have a length of 20 m, with a mid-span of about 10 m and two cantilever heads of 3.8 and 4.1 m.

Figure B.4 Overpass view from NE

Type and size of area affected

Cantilever heads of the support beams showed heavy damage due to reinforcement corrosion in the form of spalling of large parts of the concrete cover, exposing reinforcing steel, as observed about 2006.

Cause of deterioration

Although a waterproofing membrane was applied on the concrete deck, the superstructure is leaking. In winter time de-icing salts are applied on the highway. Melted snow and ice contaminated with de-icing salts leaks across the cross beams causing deterioration of concrete and reinforcement. In addition, the drainage system of the bridge is leaking. Investigations on the beams showed that the concrete cover depth varies strongly, with average 25 mm but locally even lower than the designed 15 mm. Carbonation depths were typically 20 mm and more; chloride contents were raised but relatively low. The combination of various faults (improper design, shallow construction and superficial maintenance) resulted in severe, but locally restricted corrosion damage.

Figure B.5 Reinforcement corrosion and concrete spalling