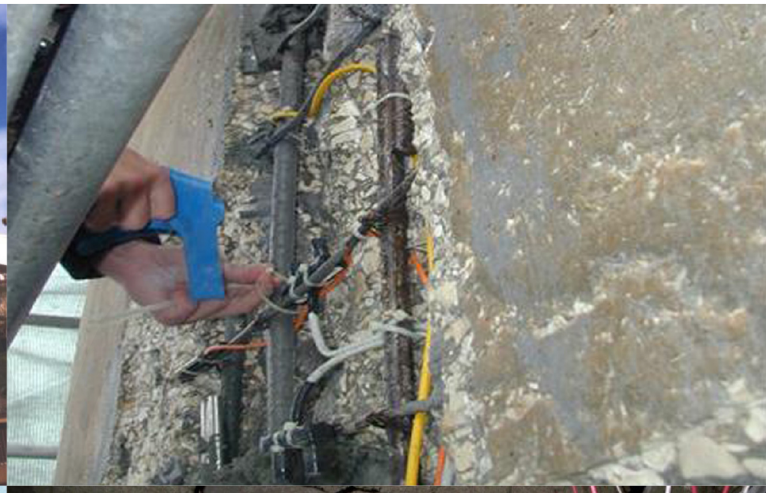
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Bulletin 102










# Guide for Protection and Repair of Concrete Structures

Guide to good practice

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fib CEB-FIP Bulletin

# 102

# Guide for Protection and Repair of Concrete Structures

Guide to good practice

Task Group 8.1

March 2022

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| **Category**: | **Approval by**: |
|---|---|
| Technical report | Task group and chairpersons of the commission |
| State-of-the-art report | Commission |
| Manual / Guide to good practice / Recommendation | Technical Council |
| Model code | General Assembly |

Any publication not having met the above requirements will be clearly identified as a preliminary draft.

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# Foreword

It is widely accepted that in parallel with the construction of new structures, great efforts must be made to preserve and rehabilitate existing structures with sustainable perspectives in mind. Indeed, one of the main changes under the *fib* MC2020 is to consider the updated design of existing structures. Therefore, background and supporting documents are needed for this purpose.

Many different concrete structures around the world require interventions either for "short" term strengthening or "long" term improvement of their overall durability performance.

The activities associated with the design, implementation, and control of the various possible interventions must be carried out in close collaboration between Asset Owners, Designers, and Specialized Contractors to select the most appropriate techniques.

To provide the international community with an up-to-date guide of available protection, repair, and strengthening methods, the *fib Durability* commission (COM8, formerly COM5) set up a task group some years ago to produce this guide to good practice Bulletin. The aim of this Task Group is to produce a comprehensive document with valuable information for all those involved in the process, including technical descriptions of the main available techniques, practical recommendations for implementation, but also case studies for better understanding by the readers.

I am convinced that this guide (bulletins 102 and 103) will provide users with a valuable tool for managing the service life of existing structures under different environmental conditions and will certainly prevent possible misuse of some techniques related to the protection, repair and strengthening of concrete structures.

Lionel Linger

Chairman of *fib* Commission 8 *Durability*

# Editorial note

The main text is in Optima 12 pts.

*Additional comments are in Palatino 10 pts, italics.*

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# Preface

The idea of preparing a technical document for the repairs and interventions upon concrete structures goes back to the former *fib* COM 5: Structural Service Life Aspects, being the goal of the then TG 5.9. After a long period of reduced activity, and taking into account the reorganization of *fib* commissions that meanwhile took place, on June 2017 a different approach was proposed to push forward the task of TG 8.1 (formerly TG 5.9).

The (new) goal of TG 8.1 was to deliver a 'how-to-do' guide, gathering together protection, repair, and strengthening techniques for concrete structures. Chapters are intended to provide both guidelines and case-studies, serving as support to the application of *fib* MC 2020 pre-normative specifications.

Each chapter was written by an editorial team comprising desirably at least a researcher, a designer and a contractor. Templates have been prepared in order to harmonize the contents and the presentation of the different methods. Following the writing process, chapters were reviewed by experts and, after amendments by the authors, they underwent a second review process by COM 8 and TG 3.4 members, as well as by different practitioners.

For each protection, repair and strengthening method addressed in this guide, readers have a description of when to adopt it, which materials and systems are required, which techniques are available, and what kind of equipment is needed. It then presents a summary of stakeholders' roles and qualifications, design guidelines referring to most relevant codes and references, the intervention procedure, quality control measures and monitoring and maintenance activities. Due to the extent of the guide, it was decided to publish it as bulletin 102, addressing protection and repair methods, and bulletin 103, addressing strengthening methods.

We would like to thank the authors, reviewers and members of COM 8 and TG 3.4 for their work in developing this *fib* Bulletin, which we hope will be useful for professionals working in the field of existing concrete structures, especially those concerned with life-cycle management and conservation activities.

As noted above, this Bulletin is also intended to act as a background and supporting document to the next edition of the *fib* Model Code for Concrete Structures, which is currently under development under the auspices of TG10.1 with the working title of '*fib* Model Code 2020'.

Eduardo Júlio
Convener of TG 8.1

Eduardo Cavaco
Secretary of TG 8.1

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# 1. Hydrophobic impregnation

## 1.1 Foreword

Surface impregnation has been used in various forms in the construction industry to help prevent the ingress of water and other substances, and its benefits are well documented.

This method can be divided into three principal categories (Fig. 1-1):

1. Coatings;
2. Pore blockers; and,
3. Pore liners.

The surface impregnation generally does not hinder the free movement of water vapour through the pores and, therefore, it allows concrete to breath. Evidence from numerous studies and field applications illustrate that the application of a hydrophobic impregnation significantly reduces water/chloride ingress and therefore the risk for structural deterioration. Hydrophobic impregnation of concrete surfaces has been used to prevent water/chloride ingress and allow the concrete to dry out. By promoting drying of the concrete cover, it may in theory reduce any existing risks of: i) rebar corrosion, after chloride contamination of the concrete cover has occurred, ii) concrete expansion, caused by alkali-silica reaction (ASR), and iii) concrete scaling by frost attack.

The most common hydrophobic treatment is the application of a silane (silane/siloxane/silicone) liner (coating), which belongs to the pore liner category.

Hydrophobic treatment of a concrete surface does not affect the outer appearance of a structure. Compared to concrete structures treated with a conventional surface coating, inspection of silane-coated concrete surface is easier, because the substrate concrete is visible.

*Comments:*

*Concrete is a naturally porous material. The size and distribution of pores in concrete varies and depends on the constituent materials, the quality of compaction, the materials used in the mix design, the water-to-cement ratio, the degree of hydration, and the curing.*

*The relevant transport mechanisms for the ingress of water, gases and ions are:*

- *diffusion of free molecules or ions due to a concentration gradient;*
- *permeation of liquids through water saturated specimens due to hydraulic pressure difference; and*
- *capillary suction of liquids due to surface tension acting in capillaries.*

This chapter was written by Christodoulou (main author), Kikuta, and Kobayashi (main author).

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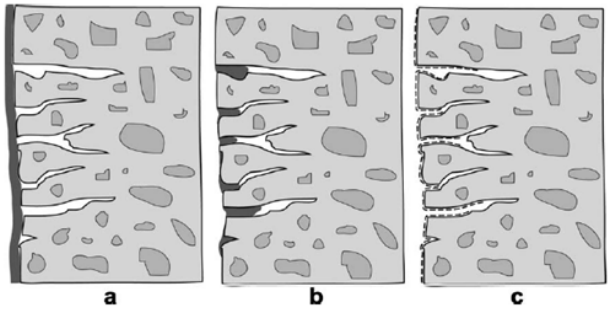



Fig. 1-1 Categories of surface treatments: (a) coatings, (b) pore blockers and (c) pore liners (Kropp and Hilsdorf 1995).

## 1.2 Basics

### 1.2.1 When to adopt this method?

Surface impregnation of reinforced concrete structures is used to prevent ingress of water and dissolved deleterious substances such as chlorides. A structure protected against water/chloride ingress will attain a longer life with a reduced maintenance regime. It is:

1. Water proof

   For preventing ingress of water itself, and for preventing deterioration such as frost attack, ASR, etc. caused by ingress of water.

2. Ion proof

   For preventing ingress of ions into the concrete carried by water as a medium. Therefore, this protection/repair method is effective in protecting the concrete from chloride attacks.

In particular, surface impregnation is well suited for surface areas which are not subjected to traffic and, as a consequence, there is no mechanical wear of the impregnation nor forced pumping transfer of deleterious materials into the substrate.

For new concrete structures, impregnation with a hydrophobic treatment can be regarded as an additional measure to protect young concrete. For existing concrete structures, not showing deterioration, frequent application of a hydrophobic treatment may be an economical preventive strategy. When the concrete is severely deteriorated, this method can be used in combination with local repair.

### 1.2.2 Materials

Hydrophobic impregnations are an attractive solution due to their effectiveness, low cost and ease of application. The solution requires the application of a hydrophobic material with the most common being that of a silane and more recently crystal blockers (Fig. 1-2).

Hydrolysed silane monomers and oligomers form a chemical bond with hydroxyl groups on the concrete micro-pore surfaces, resulting in the formation of a hydrophobic layer. The hydrophobic surface layer suppresses the ingress of liquid water and aggressive agents carried

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by water into the concrete. On the other hand, silane does not block the concrete micro pores and allows water vapour to pass unhindered through the concrete pores and to evaporate from the treated concrete surface.

Silane products are either solvent or water based with new systems, predominantly water based due to their reduced toxicity levels.

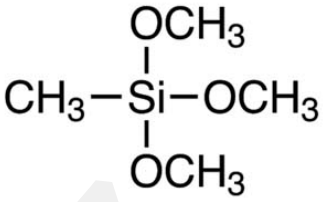



Fig. 1-2 Typical alkyl alkoxysilane molecular structure.

*Comments:*

*Silanes belong to the group of silicones and contain one silicon atom. Alkoxy and alkyl silanes are routinely used for hydrophobic surface treatments. A typical example of an alkyl alkoxy silane is illustrated below. The alkoxy groups ($CH_3O$) linked to the silicate (Si) atom contain silicon oxygen bonds to the silicates present in the concrete. The organic alkylic ($CH_3O$) group remaining will protrude from the pore structure and due to its fatty character lines up the pore wall and renders the surface hydrophobic.*

*Silane, mostly monomer, oligomer or a mixture thereof, is diluted in water or in an organic solvent, such as mineral spirit or isopropyl alcohol.*

### 1.2.3 Application techniques

Impregnations are an attractive solution because of the simplicity of the techniques available, regarding on site application.

Common methods of application include the following (Fig. 1-3):

- Low pressure spray;
- Airless spray methods;
- Brush or roller.

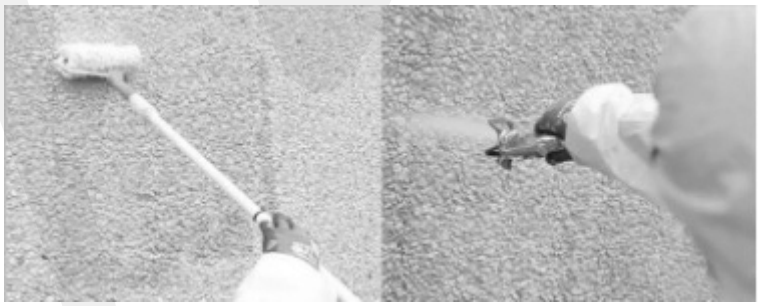


Fig. 1-3 Application techniques.

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## 1.3 Stakeholders' roles and qualifications

### 1.3.1 Owner

The Owner ought to consider the following, prior to the actual application of the technique:

- Overall structural consequences of corrosion of the structure under consideration;
- Applicability of impregnations depending on exposure conditions and environmental classification;
- Environmental constraints;
- Clear definition of design intent, such as intended service life of product, performance requirements and service life extension of the structure under consideration and requirement and/or constraints with regards to future maintenance and monitoring.

A number of the above items could be reasonably delegated or assigned to the Designer, with the exception of the intended service life requirements and identification of relevant information related to environmental constraints.

### 1.3.2 Designer

There are no particular Designer capabilities or technical expertise requirements in the same manner as for an electrochemical system for example.

However, as Designers will most probably undertake the condition and corrosion risk assessment prior to the design of a hydrophobic impregnation, they should seek assistance from durability consultants, experienced in condition testing and evaluation, to allow for the identification of the most suitable project conditions for the application of a hydrophobic impregnation.

It is also recommended that, at least, the Designer holds suitable experience in the specification requirements of coatings and surface preparation of concrete structures, as both coatings and impregnations have a number of design and application similarities.

Last, but not least, it is recommended that Designers consult manufacturers, to obtain their recommendations in the use of proprietary products.

### 1.3.3 Contractor

In general, depending on the adopted application methodology, hydrophobic impregnations only introduce few requirements covering the following issues:

- Surface preparation – experience in the surface preparation method to be used. Specialist skills and certification are needed when water blasting methods are employed.
- Application – general experience in the application of coatings and in controlling the rate of application. Specialist skills, certification and encapsulation methods are needed when airless spray or pressurised air application methods are used.

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### 1.3.4 User

One of the principal advantages of impregnations is the fact that they do not require routine inspection or maintenance.

However, the following are recommended as part of generally good asset management practice:

- Routine visual inspections of structures where an impregnation has been applied, to assess whether there are any indications of concrete deterioration;
- Material testing of the substrate concrete prior to the installation of the impregnation to establish a baseline for future reference;
- Periodic testing of the substrate after application of the impregnation to evaluate its effectiveness. This is recommended to be undertaken every 5 years and, at minimum, every 10 years.

## 1.4 Design procedure

### 1.4.1 Assessment of present conditions (existing concrete surface)

Prior to the design and installation of an impregnation, the structure should be assessed with respect to the overall deterioration risk and a condition baseline should be established. It is recommended that this includes, but is not limited to, the following:

- Visual inspections;
- Delamination survey;
- Half-cell potential survey;
- Carbonation testing;
- Chloride testing;
- Petrographic analysis;
- Sorptivity testing of the substrate concrete.

*Comments:*

*The damaged (and loose) concrete should be removed and replaced by repair mortar; after repair, the hydrophobic agent can be applied, as a preventive measure, on the completed concrete surface.*

### 1.4.2 Service life

The service life of impregnations has been noted to be highly variable, as it is a function of the selected proprietary product. These products show differences in their chemical composition, active content and also whether they are water or solvent based.

In addition, service life will vary significantly as a result of the chemical properties and characteristics of the substrate concrete.

Factors that affect service life include, but are not limited to, porosity, network of capillaries, cementitious content, degree of moisture at the time of application, etc.

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Furthermore, the environmental exposure conditions of the structure will also affect the long-term performance of the impregnation. One environmental condition that can be overlooked at times is UV exposure.

Generally, impregnations are estimated to have a useful service life of circa 10 to 15 years, depending on all of the above factors. However, studies show that measurable residual protective effects have been noted even after 20 years of service life.

### 1.4.3 Codes and standards and other relevant references

- ISO 16611-1: Maintenance and repair of concrete structures
- EN 13057:2002. Products and systems for the protection and repair of concrete structures – test methods – determination of resistance of capillary absorption. 2002.
- EN 1504-2:2004, Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity: Part 2. Surface protection systems for concrete, 2004.
- EN 1504-9:2008, Products and systems for the protection and repair of concrete structures – Definitions, requirements, quality control and evaluation of conformity: Part 9. General principles for the use of products and systems, 2008.
- ASTM C1585 – 13, Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes.
- ASTM C1757 – 13, Standard Test Method for Determination of One-Point, Bulk Water Sorption of Dried Concrete.
- ASTM C1585 – 04, Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes

*Comments:*

- *Kropp J., Hilsdorf H.K. 1995, RILEM Report 12, Performance criteria for concrete durability, Taylor and Francis, Oxford, UK.*
- *Concrete Society. Technical Report 31. Permeability testing of site concrete. Surrey (UK), 2008.*
- *Concrete Society. Technical Report 50. Guide to surface treatments for protection and enhancement of concrete. Surrey (UK), 1997.*
- *JSCE Concrete Library 119: Recommendation for Concrete Repair and Surface Protection of Concrete Structures, 2005.*
- *ACI 546R-14: Guide to Concrete Repair, 2014.*
- *British Standards Institution. BS 1881-122:1983. Method for determination of water absorption. BSI, London, 1983.*

### 1.4.4 Design assumptions

The molecular size of the interconnected pores and capillary system in the concrete structure is larger than the molecular size of the proprietary product to be used for impregnation purposes and the latter will chemically bind to the substrate.

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### 1.4.5 Design procedure

The design procedure for an impregnation system may be simplified into the following steps:

- Selection of the area of application based on the results of the condition and corrosion risk assessment.
- Identification of application constraints, such as access, geometrical, site and environmental ones.
- Selection of an appropriate proprietary product.
- Adoption of appropriate specifications. These must be determined for the selected materials and the surface conditions of the concrete to be impregnated. Manufacturers of hydrophobic agents usually provide recommendations and materials' specifications, which should be taken into consideration.
- Consultation with the manufacturer with regards to any special material handling, storage, mixing and application requirements.
- Selection of the execution method.
- Determination of the application rate of the impregnation product.
- Design of pre-treatment. Damaged concrete must be restored before the application of hydrophobic impregnation.

*Comments:*

*Important note: In some geographical locations, product selection may be subjected to an approved list of components and products that the Technical Authority may have in place.*

*Application rate of impregnants is critical and this ought to be determined based on consultation with the manufacturer of the chosen proprietary product and also on its interaction and impregnation capabilities for the specific substrate being designed for.*

### 1.4.6 Supporting documents

The following are considered, as minimum requirements:

- Material data sheet;
- Health & Safety Material Hazard Assessment;
- Design documentation including drawings and specification;
- Records of application rates;
- Records of performance evaluation following the application of the impregnation – in accordance with design and specification quality assurance and conformity requirements.

Along with records of design details, a maintenance plan should also be prepared.

*Comments:*

*The maintenance plan must be prepared beforehand. Periodic investigation of the water-repellent performance is recommended.*

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## 1.5 Execution

### 1.5.1 Preparatory works

Confirmation of the location and area to be treated on site. It should be checked if the temperature is higher than the freezing point of the product.

The following preparatory works need to be carried out:

- Removal of deteriorated concrete and any existing surface coatings;
- Crack sealing (as necessary);
- Surface preparation of concrete substrate by either mechanical or water blasting methods to achieve design specific surface key level;
- Cleaning of prepared surface by means of an air lance to remove any loose debris, oil or other laitance.

### 1.5.2 System trials

As noted previously, it is advisable to undertake material sampling and in particular chloride testing of the parent concrete prior to the application of the impregnation to establish residual levels of contamination for benchmarking purposes and long-term evaluation of performance.

Furthermore, it is recommended that the effectiveness of the proposed proprietary product in reducing the sorptivity rate of the substrate in question is assessed and evaluated by extracting cores from the structure and applying different application rates in order to determine the optimum one.

### 1.5.3 Execution procedure

- Preparation of the surface of the concrete substrate;
- Impregnation application.
- The dosage specified in the design procedure must be used.

*Comments:*

*Enriched materials should be diluted in the ratio specified by the manufacturer.*

### 1.5.4 Finishing

The applied impregnation product shall be treated as per the manufacturer's requirements and recommendations.

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## 1.6 Quality control

### 1.6.1 Quality control of materials

Products must be stored in a cool place and protected from direct sunlight. The container should be sealed off after use.

### 1.6.2 Quality control before intervention

As impregnations penetrate the concrete substrate, the surface preparation requirements are generally minimal and are focused primarily on ensuring that the substrate is free from loose debris, oil and laitance alike.

### 1.6.3 Quality control during intervention

Most modern proprietary products are supplied by the manufacturers premixed in single containers. When mixing of parts or dilution is required, records of the quantities used shall be kept, to ensure that the mixing conforms to the thresholds set by the manufacturer.

During application, the following shall be recorded as a minimum:

- Ambient weather conditions (i.e. air shade temperature and notes on rainfall);
- Moisture level of substrate;
- Dew point;
- Application rate of material.

In order to ensure whether the impregnation could penetrate sufficiently to form an effective hydrophobic layer, it is recommended to measure the penetration depth.

*Comments:*

*In dry climates, the treated area may be divided into small sections to improve quality and quantity control. If necessary, the impregnation may be applied several times in accordance with the procedure specified by the manufacturer.*

*The humidity of the concrete surface is a very important factor that affects the effectiveness of an impregnation. If concrete is too dry, there will be insufficient water for hydrolysis. On the other hand, if concrete is too wet, the treatment will not be able to penetrate sufficiently deep.*

### 1.6.4 Quality control after intervention

In most applications there is no requirement for quality controls after intervention.

Nevertheless, it is recommended that following application of the impregnation, core samples where the material was applied are extracted from the substrate concrete and tested to evaluated the resulting sorption.

This will assist in evaluating whether the application rate was sufficient to meet the original design target and whether the product had been applied at the design requirement rate.

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*Comments:*

*After penetration into the concrete, silane is hydrolysed and condensed to make the concrete hydrophobic. The treated area must be protected against direct contact with water (rain, snow) until the complete hydrolysis is achieved.*

### 1.6.5 Health and safety

Local laws regulating the work environment and waste disposal of organic solvents must be abided by.

## 1.7 Monitoring and maintenance

### 1.7.1 Monitoring

Impregnations require no regular in-service monitoring. It is only recommended that their levels of residual protection are assessed at least every 10 years, but preferably every 5 years, by means of sorptivity testing. This will assist in evaluating when protection may be practically lost or diminished which would necessitate reapplication.

### 1.7.2 Maintenance

There are no maintenance requirements with the application of impregnations.

However, the surface should be impregnated again if a significant decline in the water repellent performance is observed.

*Comments:*

*The old hydrophobic layer does not have to be removed before the new treatment is applied. However, the preparation work (See 1.5.1) has to be conducted.*

### 1.7.3 Post-intervention documentation

As a minimum, it is recommended that the following records are kept:

- Design details highlighting area over which the impregnation was applied, design application rate and specification requirements;
- Application records stating actual application rate, date of application and details of the proprietary product used;
- Records of substrate and material testing.

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## 1.8 Case study

| | |
|---|---|
| Project Title: | Application of silane surface impregnated material in cold region |
| Location: | Hokkaido, Japan |
| Client: | Hokkaido Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism |
| Designer: | not available |
| Contractor: | not available |

### 1.8.1 Description of the structure

The structure for application of the material is a national highway bridge built in the mountainous region with a snowy cold climate, where de-icing salts are applied during winter periods.

### 1.8.2 Description of the intervention project

Concrete structures in cold regions are exposed to severe environments that are susceptible to the combined action of freezing/thawing and chloride attack (de-icing agent, seawater, etc.). The application of a hydrophobic treatment of to the concrete surface is expected to suppress the penetration of water and chloride ions, which are deterioration factors, and to delay the occurrence and progress of frost damage and chloride attack.

However, the effect of the silane surface impregnated material is affected by various factors, such as the type of product, the quality of the concrete surface, the execution environment, and the environment to which the structure is exposed. In order to obtain the intended effect, it is important to pay careful attentions to the choice of the proper hydrophobic agent and execution method.

For this project, in order to prevent chloride attack and surface scaling caused by de-icing agents, it was decided to apply silane surface impregnated material to the concrete barrier of the newly constructed structure shown in Fig. 1-4.



Fig. 1-4 Concrete barrier of bridge to which a silane surface impregnated material is applied.

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## 1.8.3 Description of the intervention works

### 1.8.3.1 Applicable condition

The following findings have been obtained from the results of previous research shown in Fig. 1-5 regarding the application conditions of the silane surface impregnated material

1. Even though the silane surface impregnation could supress the scaling in experiments that simulated the immediate application after construction and replacement, it could not suppress the deterioration well in experiments that simulated existing structures.
2. When the concrete was immersed in water, the suppression effect was lost, and then, rapid scaling followed.

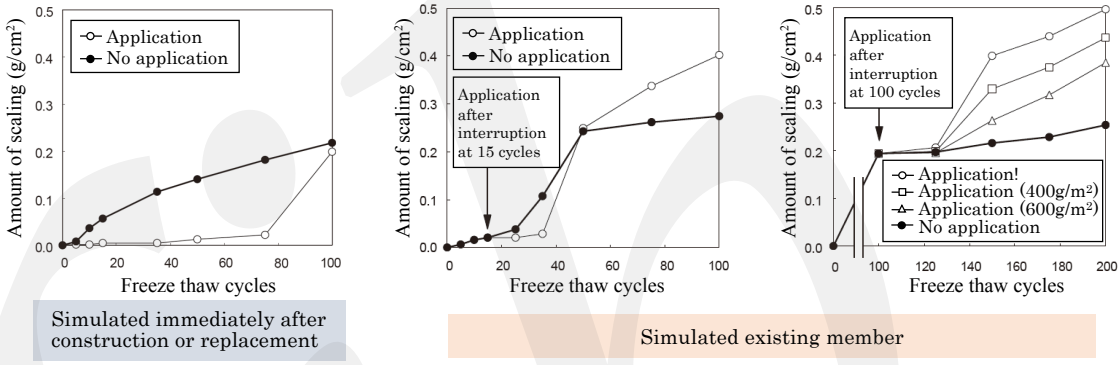



Fig. 1-5 Influence of applying silane surface impregnated material on surface scaling.

Therefore, the following two conditions should be taken into account when a hydrophobic treatment is considered:

1. The silane surface impregnation shall be applied to newly built parts or a member, immediately after a repair.
2. Application to parts that are always immersed in water should be avoided as much as possible.

The silane-based hydrophobic agent can be applied to the concrete surface if the chloride content at the depth of the reinforcing steel has not yet reached a level where corrosion will start and if harmful cracks are not present, the silane can be applied to the existing structure.

The structure addressed in this example is a newly constructed bridge, and the barrier is not always in contact with water. Regarding the above conditions, it was expected that the silane surface impregnation works well, therefore it was applied there.

### 1.8.3.2 Selection of materials

The effect of silane treatment on frost/salt damage largely depends on the quality of the water-repellent layer formed on the concrete surface. For this reason, a product which are is able to penetrate deep into the concrete to form a water-repellent layer capable to resist ingress of water including dissolved deleterious species was selected.

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### 1.8.3.3 Execution

Contact with water will cause silane's "flash setting reaction", resulting in insufficient penetration. For this reason, it is necessary to pay attention to avoid moist concrete surfaces. This time, silane- based hydrophobic agent was applied in favourable weather conditions.

The impregnation depth of the silane surface impregnated material depends not only on the material properties but also on the quality of the concrete, and the impregnation depth decreases when the water-cement ratio is low and the concrete is dense. For this reason, the achieved impregnation depth should be determined on site. This time, however, it was decided not to test the depth, as since a current concrete was very usual concrete with designed design compressive strength of 24 MPa was present.

### 1.8.3.4 Record

It is important to record the information about the material, as it is very valuable for maintenance and drawing up a re-application plan in future. This time, the rating plate shown in Fig. 1-6 was installed. Table 1-1 is an example of the general description items.




Fig. 1-6 An example of rating plate.

Table 1-1 An example of general description items on the plate

| |
|---|
| Name of structure |
| Applied parts |
| Applied date |
| Material used (product name) |
| Material supplier |
| Contractor |

Fig. 1-7 shows the scaling by the freeze thaw test. From the graph, it can be seen that scaling progressed until a depth of 6mm into the concrete whose W/C was 0.55. Thus it is expected that silane will penetrate much deeper to form a hydrophobic concrete layer.

It has also been found that concrete can have sufficient scaling resistance when a chloride ion penetration depth is 3 mm or less*.

By considering these findings, a material that satisfies these requirements was chosen.

* Chloride ion penetration depth in "resistance test against chloride ion penetration" specified by Japan Society of Civil Engineers (JSCE-K 571). The penetration depth is measured after the immersion of concrete specimen in NaCl 3% solution for 63 days.

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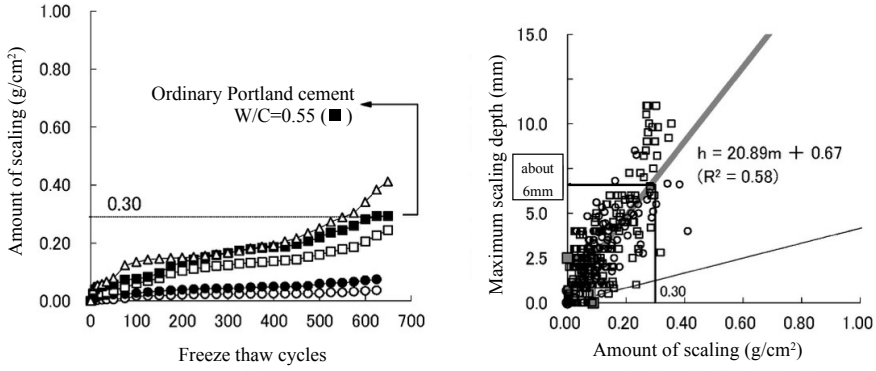



Fig. 1-7 Scaling amount and depth by freeze thaw test.

### 1.8.3.5 Re-application plan

As shown in Fig. 1-8, re-application of the silane surface impregnated material is more effective against the progress of the scaling. In order to maintain the preventive effect for a long term, periodic re-applications are required before any deterioration occurs. By considering this, a re-application plan was drawn up.

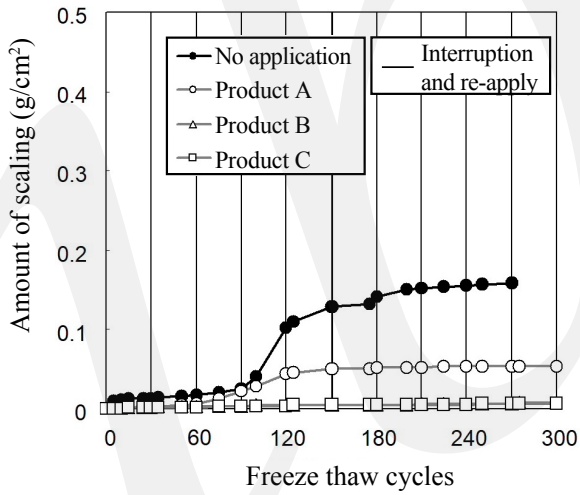



Fig. 1-8 Suppression of scaling by repeated application of silane surface impregnated material.

Based on the results of trial applications and a follow-up survey conducted on a concrete bridge in a similar environment, the life cycle cost was calculated when repeated re-applications were performed. The outline and results of this calculation are as follows.

1. As shown in Fig. 1-9, a water repellent layer that can prevent chloride penetration was persisting even after 10 years having passed since its first application, so the reapplication interval was set to 10 years.
2. In the case of concrete without silane application, it was assumed that the removal and replacement would be required at 50 years after its construction, according to the predicted results on chloride ion penetration.
3. As shown in Fig. 1-10, it is found that the life cycle cost with silane after 100 years is expected to be lower by about 35% compared to the case without silane application. Based on these examination and calculation, it was planned to re-apply silane surface impregnated material to the barrier every 10 years.

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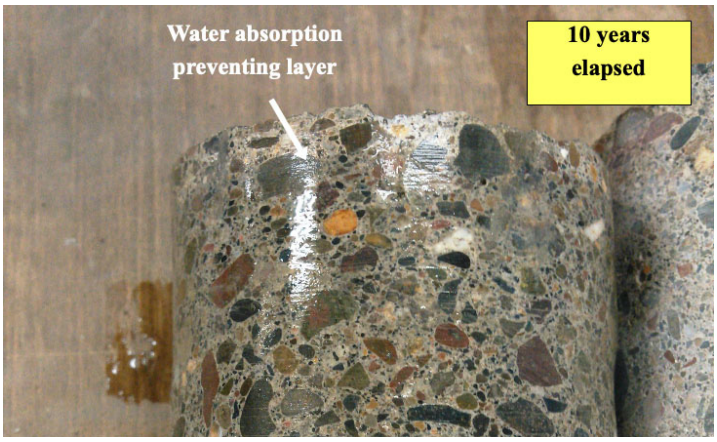



Fig. 1-9 Water absorption prevention layer observed 10 years after application.

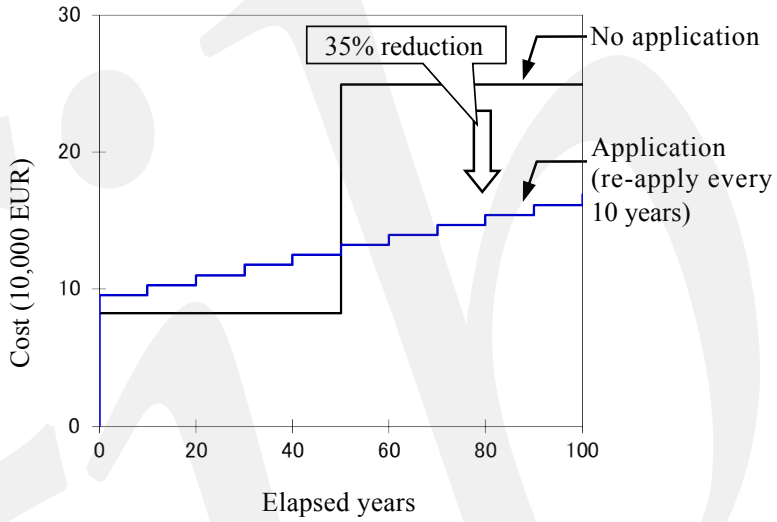



Fig. 1-10 Trial calculation result of life cycle cost.

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# 2. Coating of concrete surfaces

## 2.1 Foreword

Coating of concrete surfaces is a method of protection consisting of applying a product that forms a continuous layer on the concrete surface, after curing or drying. Layer thickness ranges typically from 100 μm to 5 mm. Examples of products are liquid paints, paste like products (based on polymers with cement as a filler) or polymer modified hydraulic cement mortars.

Coating can change the surface appearance of the concrete structure. Frequently coatings are used also for aesthetical reasons, especially after repair actions, in order to obtain a uniform aspect.

## 2.2 Basics

### 2.2.1 When to adopt this method

Coatings can be applied to new or repaired concrete structures in order to prevent or reduce deterioration processes either in the concrete itself or in the reinforcement bars (corrosion). The environmental loads defined in EN 206-1 or other physical and chemical actions are the causes of several concrete defects in result of those deterioration processes. Common causes of defects and deterioration of concrete structures are described in ISO 16311-3, EN 1504-9 and DURATINET (Technical Guide - Concrete Structures).

Coatings can be applied (i) as a preventive measure in the construction phase, before initiation of the deterioration processes, in order to achieve a required service life of the concrete structure or (ii) can be included in a repair system to achieve a prolonged service life.

*Comments:*

*According to EN 1504-1 and EN 1504-2, coating is a "Treatment to produce a continuous protective layer on the surface of concrete".*

*For particular applications like waterproofing, damproofing, flooring and foundations, layers can have several millimetres (much higher than 5 mm) and must have special requirements since they must sustain substantial mechanical loadings. Membranes, sheets and linings, preformed or not, are examples of materials for some of such applications.*

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This chapter was written by Cabral-Fonseca, Moreno, and Paula Rodrigues (main author).
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Coatings can:

- prevent or mitigate deterioration processes involving concrete carbonation and concrete chloride penetration, erosion, freeze and thaw, acid attack or biological attack, reinforcement corrosion, alkali aggregate reactions (AAR) and sulphate attack (SA);
- increase the length of the initiation period preceding the degradation by reducing the transport of water, carbon dioxide, chlorides, sulphates, acids or some other aggressive compounds;
- mitigate the reinforcement corrosion, by reducing corrosion rate due to the decrease of access of oxygen and water, mainly if concrete is not contaminated with mix chlorides.

The coating to be used must have performance characteristics adequate to each situation and identified principle of protection that are also related with the intended use. Those characteristics, test methods and minimum values of the characteristics are indicated in EN 1504-2 for each identified principle of protection.

*Comments:*

*Hypothetically, coatings can mitigate internal expansive reactions (AAR or SA) by controlling the humidity level inside concrete, since they act as a barrier against ingress of water. However, the effect of the coating depends on the residual reactivity of the aggregates and on the moisture level inside the concrete at the time of application.*

*Depending on their performance characteristics, coatings can:*

1. *protect concrete against the ingress of aggressive substances present in the surrounding environment, acting as a "barrier" against those substances;*
2. *reduce the access of water and allow the drying (if their water vapour permeability is adequate, which depends on the type of coating and thickness);*
3. *avoid or reduce the influence of physical actions over the concrete surface (such as abrasion and impact);*
4. *avoid or reduce the concrete attack by chemicals like those listed in EN 206-1, EN 13529 or ACI 515.2R-13);*
5. *contribute to increase the resistivity of concrete to a level that allows the corrosion rate of the rebars to be maintained at a low level (as long as they have adequate characteristics to reduce the access of oxygen and water and allow the drying of concrete).*

The Principles of Protection identified in EN 1504-9 are based on chemical, electrochemical or physical principles that can be used to prevent or stabilise the deterioration of concrete or the electrochemical corrosion of the steel bars. The Principles of Protection under the base of application of coatings as a method of protection are the following five (in a total of eleven covered by other kind of methods of repair and protection):

P1- Protection against ingress;

P2- Moisture control;

P5- Increase physical resistance;

P6- Resistance to chemicals;

P8- Increasing resistivity.

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Care needs to be taken to consider the possible adverse effects of the application of a coating system after a rehabilitation process due to eventual consequences of interactions between the repair methods chosen. For example, surface coating may entrap moisture, leading to a breakdown in adhesion or reduced frost resistance.

*Comments:*

1. *P1 - Protection against ingress*

   *Effect: to reduce the access of aggressive agents to the inner concrete - water and moisture, aqueous solutions (e.g. chlorides and sulphates) or gases like $CO_2$.*

   *Example of afforded protection: restrict or prevent corrosion of rebars caused by concrete carbonation or chloride penetration; sulphate action.*

2. *P2 - Moisture control*

   *Effect: adjust or maintain certain limits of humidity level inside concrete.*

   *Example of afforded protection: restrict or prevent degradation mechanisms of physical degradation related with freeze thaw cycles and salt crystallization inside concrete; reduce alkali- aggregate reactions.*

3. *P5 – Increase physical resistance*

   *Effect: increase of the resistance to physical and mechanical actions Example of afforded protection: restrict or prevent the surface degradation by erosion or impact.*

4. *P6 – Resistance to chemicals*

   *Effect: increase the resistance to chemical attack.*

   *Example of afforded protection: restrict or prevent adverse chemical reactions between compounds present in the environment and concrete constituents.*

5. *P8 – Increasing resistivity*

   *Effect: control of the humidity inside concrete.*

   *Example of afforded protection: restriction or prevention of rebars corrosion by reducing of ionic flux inside concrete (reducing corrosion velocity), or reduction the effect of stray electrical currents conducted or induced in the reinforcement from neighbouring electrical installations.*

### 2.2.2 Materials and systems

Coating is the result of the application of a product that forms a continuous layer on the concrete surface. The products can be liquid or in paste, water based, solvent based or solvent free and can be formulated with different binders like organic polymers (acrylics, epoxies, polyurethanes, vinyl, styrene butadiene, etc.), hydraulic cement + polymers, bitumen + polymers.

The coating protection layer can be a multi-layer system resulting from the application of layers of different types of products, following recommendations of the manufacturer. For instance, an organic coating system can consist of several layers where each single layer has specific functions (barrier against water, oxygen or $CO_2$ and the final coat offering protection against UV radiation or abrasion resistance). The performance characteristics of the system must be evaluated as a whole.

### 2.2.3 Techniques

Information given by the manufacturer on the technical data sheets must be followed, namely in what concerns surface preparation, application technique, dilution and drying times between each layer.

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### 2.2.4 Equipment

Coating products are applied on the properly prepared concrete surface with brush, roller, trowel or by spraying depending on the type of product, its viscosity, the area to be treated, its accessibility, localization and surrounding area. Information given by the manufacturer on the technical data sheets must be followed.

## 2.3 Stakeholders' roles and qualifications

### 2.3.1 Owner

The owner must:

- Designate an expert/designer for a formal assessment of the structure's condition prior to any decision on protection or repair to be made, in order to accurately determine the causes of deterioration (e.g. salt exposure; under-design; concrete defects, etc.);
- Provide documentation or reports about previous interventions on the structure (repair or maintaining actions and dates, type of used products and materials);
- Inform the designer about the future use, life expectancy and budget constraints for the structure, so that the designer can propose the type of protection method and the characteristics of the protection coating system;
- Designate responsibility for the supervision and quality control of the works to ensure consistency and quality of the protection and repair.

### 2.3.2 Designer

The designer must:

- Assess the structure and the defects and their causes in order to specify the protection and repair procedure and the requirements for the surface preparation and for the coating system;
- Consider the requirement for the substrate preparation that must be adequate for the coating system to be applied;
- Define the performance characteristics of the coating materials to be used;
- Develop instructions on inspection and maintenance for the total or residual life of the structure.

In this way the designer must know the available products on the market and their generic characteristics and must have knowledge about the specific properties required for the protection coating system. All these requirements must be available to the contractor as a specification for the project. The specification should be detailed and cover all aspects of the work including surface cleaning and preparatory works, the nature and sequence of the protection or repair operation. Information on weather precautions, material thicknesses and curing procedures must also be indicated.

Guidelines and information can be found in EN 1504 Parts 2, 9 and 10 and also in Concrete Society Technical Report No. 54, "Diagnosis of Deterioration in Concrete Structures".

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### 2.3.3 Contractor

All the works, from the surface preparation, until the coating application, shall be carried out by a contractor with experience in concrete protection and repair of a similar nature to those being addressed and must:

- Follow the requirements of the specification project;
- Use products with the specified characteristics and present a report from an independent entity attesting those characteristics;
- Have the means in terms of qualified staff and adequate equipment.

Inspection and supervision should ensure that surfaces are well prepared and suitable for application of the coating protection system.

### 2.3.4 User

The user must:

- Give to the structure the expected use;
- Follow the maintaining plan, if previewed.

## 2.4 Design

### 2.4.1 Assessment of existing conditions

Application of coatings is the final part of a repair strategy that was decided after assessment of the concrete structure in terms of defects and their causes. Guidelines can be found in EN 1504-9, containing comprehensive information on the assessment of the actual conditions of the concrete structure and on planning the work.

Further information on the inspection and testing procedures to assist the designer in forming an accurate diagnosis can be found in Concrete Society Technical Report No. 54, “Diagnosis of Deterioration in Concrete Structures”.

### 2.4.2 Service life

Coatings inevitably require maintenance and must be reapplied in cycles over the life of the concrete structure. The potential durability of the coating system can be evaluated by laboratory tests but in practice it depends on the exposure conditions and in-service conditions of the structure. Information about the experience of use or on the expected life span of the product can be taken into account for the cost analysis of the work.

### 2.4.3 Codes and standards and other relevant references

- EN 1504: Products and systems for the protection and repair of concrete structures. Definition, requirement, quality control and evaluation of conformity. Part 1: Definitions (2005); Part 2: Surface protection systems for concrete (2004); Part 9: General principles for the use of product and systems (2008); Part 10: Site application of products and systems and quality control of the works (2017).

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- ISO 6311-3, 2014, Maintenance and repair of concrete structures. Part 3: Design of repairs and prevention.
- DURATINET, 2012 – Maintenance and repair of transport infrastructures – Technical guide. Concrete Structures. LNEC Publisher, Collection Publisher, ISBN: 978-972-49-2237-9.
- ACI 515.2R-13, 2013. Guide to Selecting Protective Treatments for Concrete. ACI American Concrete Institute.
- Advanced Concrete Technology - Processes. Eds. John Newman, Department of Civil Engineering-Imperial College-London and Ban Seng Choo, School of the Built Environment-Napier University-Edinburgh, Elsevier, 2003.
- Concrete Society Technical Report No. 69. Repair of Concrete Structures with Reference to BS EN 1504. April, 2009.

### 2.4.4 Design assumptions

The coating system must be suitable for use on an alkaline concrete surface. The surface must be cleaned and prepared to be coated. All the components of the coating system should be obtained from the same supplier to ensure compatibility.

Recommendations should always be available from the supplier about the range of ambient temperature and humidity acceptable during application and drying/curing, since they can vary, according to both generic type and formulation.

### 2.4.5 Design procedure

The selection of the protective coating systems shall be based on results of laboratory performance tests and on practical experience.

This selection must be made after identification of the degradation actions and exposure environment to which the concrete structures are subjected in order to define the Principles of Protection (e.g. P1, P2, P5, P6 or P8 according to EN 1504-9) that can be used to prevent or stabilise the deterioration of concrete or the electrochemical corrosion of the steel bars.

Coating systems must have performance characteristics that allow them to protect concrete against the previously identified actions, related also with the intended use and environmental conditions.

On previously coated surfaces, the protective coating system shall be tested for compatibility with the existing coatings or the removal of the old coating should be considered.

Guidance about the specific characteristics and corresponding test methods and minimum values to be satisfied by the coating systems can be found in EN 1504-2. More information is given in the present document in Section 2.6.1.

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### 2.4.6 Supporting documents

All activities performed during the several steps of the work must be described in documents and registers:

1. Specification for the project;
2. Coating plan proposed by the manufacturer for the products and respective data sheets;
3. Test report with the characterisation results of the products;
4. Planning for the surface preparation and acceptance;
5. Planning for the final acceptance of the works;
6. Maintenance and inspection of the coating system, after the intervention.

## 2.5 Execution

### 2.5.1 Preparatory works

The application of a coating system on the concrete surface is the last phase of the process of repair and protection.

Since many coating failures are due to inadequate concrete surface preparation, a proper preparation of the surface must be performed in order to get a sound, rough-textured, and dried substrate. All the concrete defects must have been corrected and cracks, pores and other surface defects that cannot be covered by coatings must be treated. If necessary, smoothing coating materials shall be applied and cured before protection coating products are applied to fill uneven surfaces and surface pores and to allow the formation of a homogenous coating film. The type and nature of what is required depend on the material to be applied and manufacturers typically make specific recommendations for their products that must be followed.

The concrete surface shall be free from dust, loose material, salts, surface contamination and materials which reduce the bond of the coating. Water-jetting, water-pressure cleaning, clean compressed air or vacuum cleaning are effective methods.

Cleaning of concrete surfaces without removal of concrete is normally performed with water pressures up to 18 MPa. Another alternative is blasting with water and sand in order to obtain a clean and rough substrate to promote the coating adhesion.

If necessary, adjacent glass, metal, and painted surfaces must be protected from the coating system constituents. Overspray protection must be provided if spray application equipment is used since fine spray mists can travel long distances and damage nearby buildings, equipment and automobiles.

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The need to remove an existing coating depends on its condition, including adhesion and its chemical compatibility with the new coating system. Laboratory and in situ tests can be done in order to identify the old coating and its adhesion (e.g. performing the cross-cut test according to EN ISO 2409) and help to select the procedure to follow.

EN 1504-10 gives general advice on procedures to be followed for a correct surface preparation.

Guidance on Surface preparation can be obtained e.g. from the American Society for Testing and Materials (ASTM), American Concrete Institute (ACI), International Concrete Repair Institute (ICRI), and Society for Protective Coatings (SSPC).

### 2.5.2 System trials

It can be helpful to define in the concrete structure a reference area accepted by all interested parties to establish a minimum acceptable standard for the work, either in terms of surface preparation or the coating acceptance, and to enable the performance of the coating to be assessed at any time after completion of the work.

### 2.5.3 Execution procedure

Specified storage and application conditions regarding ambient temperature, humidity and dew point, wind force and precipitation and any temporary protection which is needed must be accomplished. Information on the technical data sheets must be followed.

The products must be applied in the correct way, within the specified maximum and minimum thickness, according to the technical data sheets. When several layers are to be applied in order to get a specified thickness, it can be useful that the intermediate layer has a different colour in order to facilitate the control of the number of layers.

Manufacturer's curing requirements should be followed. Different materials will have different curing requirements, and exposure conditions will impact on curing time.

*Comments:*

*The substrate moisture content and temperature should be appropriate to the coating material (a normal temperature is typically 5 ºC to 30 ºC but the range 10 ºC to 35 ºC is in general acceptable).*

*Application of the products shall not be done during rain and normally not in moist conditions and an additional requirement for polymeric coatings is that they must be applied only if the surface temperature is at least 3 ºC above dew point to minimize risk of excess moisture at the surface.*

### 2.5.4 Finishing

For satisfactory appearance and durability, it is essential to monitor the application of the correct coating thickness by controlling the consumption rate over known areas – taking into account the effects of surface roughness and wastage.

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## 2.6 Quality control

### 2.6.1 Quality control of materials

The materials must be tested and e.g. in Europe they must have CE marking and suppliers should therefore maintain a continuous production control system and perform material characterisation tests.

During materials reception, marking and labelling should be checked. Visual inspection should also be performed.

Products should be handled, transported and stored according to manufacturers' specifications in order not to cause changes in their properties.

The characteristics described in technical data sheets must be compared with results from laboratory tests made to assess the performance characteristics and, whenever applicable, should fulfil the minimum required performance. Guidance can be obtained in EN 1504 2 that defines general performance characteristics (which all coatings must have and that are independent of the intended use), specific characteristics (additional to the general characteristics and appropriate to the actual intended use), and the corresponding test methods and minimum values to be satisfied by the coating systems. All these characteristics are function of the Principles of Protection previously defined (P1, P2, P5, P6 and P8).

*Comments:*

1. *Performance characteristics for "all intended uses" that must be evaluated on coating systems: Principles of Protection P1, P2, P5, P6 and P8*

   *P1 (Protection against ingress): permeability to CO2 (EN 1062-8), permeability to water vapour (EN ISO 7783-1 or 2), capillary absorption and permeability to water (EN 1062-3), adhesion strength by pull-off test (EN 1542).*

   *P2 (Moisture control): permeability to water vapour (EN ISO 7783-1 or 2), capillary absorption and permeability to water (EN 1062-3), adhesion strength by pull-off test (EN 1542).*

   *P5 (Increasing physical resistance): abrasion resistance (EN ISO 5470-1), capillary absorption and permeability to water (EN 1062-3), impact resistance (EN ISO 6272-1), adhesion strength by pull-off test (EN 1542).*

   *P6 (Resistance to chemicals): resistance to severe chemical attack (EN 13529), adhesion strength by pull-off test (EN 1542).*

   *P8 (Increasing resistivity): permeability to water vapour (EN ISO 7783), capillary absorption and permeability to water (EN 1062-3), adhesion strength by pull-off test (EN 1542).*

2. *Performance characteristics for "some intended uses" that can be evaluated additionally on coating systems in some situations: Principles of Protection P1, P2, P5, P6 and P8*

   *Linear shrinkage (EN 12617-1), coefficient of thermal expansion (EN 1770), adhesion (cross-cut) (EN ISO 2409), freeze-thaw cycling with de-icing salt immersion (EN 13687-1), thunder-shower cycling (thermal shock) (EN 13687-2), thermal cycling without de-icing salt impact (EN 13687-3), ageing: 7 days at 70 °C (EN 1062-11), behaviour after artificial weathering (EN 1062-11), crack bridging ability (EN 1062-7), slip/skid resistance (EN 13036-4), adhesion on wet concrete (EN 13578), fire classification of construction products and building elements - Part 1: Classification using test data from reaction to fire test (EN 13501-1).*

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3. *Performance characteristics for "some intended uses" that can be evaluated additionally on coating systems, specific to each one of the following Principles of Protection: P1, P5, P6 and P8*

   *For P1 and P8: resistance to thermal shock (EN 13687-5), diffusion of chloride ions (no international standard available).*

   *For P5: resistance to thermal shock (EN 13687-5).*

   *For P6: capillary absorption and permeability to water (EN 1062-3), resistance to thermal shock (EN 13687-5).*

### 2.6.2 Quality control before intervention

Immediately prior to the application of the coating system, the concrete substrate should be inspected, regarding the following aspects: removal and/or repair of degraded concrete (carbonation, chloride penetration, delamination, cracks); surface roughening, evenness, cleaning, temperature and moisture content (follow technical data sheet).

EN 1504-10 gives guidelines for the surface preparation, application and quality control procedures during intervention.

Some quality control parameters before application of coatings may be specified, for particular applications, such as roughness of the concrete surface or dew point temperature.

### 2.6.3 Quality control during intervention

Inspection of the work shall verify that the protection or repair works are completed in accordance with the project specification.

The following environmental conditions should be controlled prior to and during execution: temperature (within the range 10 ºC to 35 ºC), relative humidity, dew point, wind and rain.

During execution, quality control should cover the following aspects:

(i) handling and mixing of the products (dosage of components, mixing procedure and duration, homogeneity and colour uniformity, environmental conditions); (ii) proper execution procedure; (iii) thickness control.

For thickness control wet film gauges can be useful for carrying out rapid spot-checks during application, but the surface must be reasonably smooth in order that an acceptable accuracy is obtained. EN ISO 2808 provides three mechanical methods for measuring the wet thickness: comb gauge, wheel gauge and dial gauge; monitoring may be carried out via metallic plates attached to the surface prior to coating, thus allowing to use of the more common electronic thickness gauges.

If decided to use a reference area, the control is to be performed also in this area.

### 2.6.4 Quality control after intervention

To check the dry film thickness different destructive means are available such as the depth gauging method described in EN ISO 2808 - method 4B and the wedge cut method (using a special instrument which includes a microscope, an illumination device and a cutting tool) described in EN ISO 2808 - method 6B. Non-destructive direct reading instruments based on ultrasonic methods are also available – method 10 of EN ISO 2808 for instance. In the event of dispute, however, direct microscopic measurements on cored specimens may be necessary.

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Once the coating has cured, adhesion can be tested using the cross cut test as specified in EN ISO 2409 or by pull-off tests as specified in EN ISO 4624 or EN 1542, using one of the several instruments available commercially.

The cross-cut test can be used for layers less than 0,5 mm in thickness and the pull-off test for thicker layers. The procedures, the number of measurements and the performance criteria should be agreed beforehand.

In the case of organic coatings, the appearance of the film can be evaluated, by visual inspection, according to the standard series EN ISO 4628, which distinguishes the following defects: blistering (Part 2), rusting (Part 3), cracking (Part 4), flaking (Part 5), chalking (Parts 6 and 7), delamination or another artificial defect (Part 8).

## 2.7 Monitoring and maintenance

### 2.7.1 Monitoring

Periodic visual inspection must be done to detect deterioration and take measures to avoid its progression.

Tests on the reference area can be performed periodically, including by coring for lab testing.

### 2.7.2 Maintenance

Users should have a routine maintenance plan to keep the structure in good order. Clearance of drainage, repairing leaks, removal of detritus and cleaning of concrete surfaces are typical examples.

Some preventive measures that contribute to increase the durability of the coating material are also the following: removal of biological growth (fungi, algae, lichens); maintenance of construction details that protect the surface from runs and infiltrations in result of the rain action; repairing any damage to the coating not due to normal ageing.

Such operations may save time and money by avoiding major remedial works in future. In addition, they may facilitate future inspections carried out as part of the ongoing management of the structure.

### 2.7.3 Post-intervention documentation

The design and the repair project, details of execution, quality controls, and maintenance procedures shall be appropriately recorded and kept.

Those responsible for structures, particularly after they have been repaired, should have complete documentation of the work done and be aware of the structure's ongoing maintenance and monitoring needs. They should implement a program of at least regular visual inspection and where this shows up problems, they should be solved before they lead to health and safety issues and to further expensive works of repairing.

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## 2.8 Case study

| | |
|---|---|
| Project Title: | S. João bridge |
| Location: | Oporto, Portugal |
| Client: | Gabinete do Nó Ferroviário do Porto / Infraestruturas de Portugal, S.A. |
| Designer: | Project by Engineer Edgar Cardoso |
| Contractor: | Consórsio FERDOURO ACE formed by Sociedade de Construções Soares da Costa, Teixeira Duarte and OPCA, Obras Públicas e Cimento Armado, S.A. |

### 2.8.1 Description of the structure

Railway bridge crossing river Douro, linking the cities of Gaia and Oporto. The construction started in 1984 and the inauguration was in 24 June 1991. It is a prestressed concrete bridge with a main span of 250 m and a total length of 1028 m. The height above valley floor or water is 66 m (Fig. 2-1 and 2-2).



Fig. 2-1 S. João bridge over Douro river.



Fig. 2-2 South viaduct of S. João bridge.

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### 2.8.2 Inspection highlights

During the construction it was decided to use a surface coating with characteristics adequate to protect concrete against chloride penetration, besides carbonation. Justification: the structure, with a high steel density, is near the maritime coast and the atmospheric transport of chlorides could be an issue for the corrosion of rebars.

### 2.8.3 Description of the intervention project

All the concrete surfaces of the structure were coated with a white paint (Fig. 2-3) that was previously tested at the laboratory by using test methods available in order to guarantee that the coating had a performance adequate to protect the concrete against aggressive agents from the atmosphere. In that way, the following characteristics were determined: permeability to $CO_2$, permeability to chloride ion, water vapour permeability, permeability to liquid water, adhesion to a concrete substrate, resistance to alkaline substances and resistance to artificial weathering.



Fig. 2-3 S. João bridge during the painting phase in 1990.

In order to follow the performance of the coating during the life time of the structure, a reference surface close to the bridge was prepared by using a full-scale experimental structure that was constructed for the performing of tests during the progress of the construction project. This experimental structure had the same characteristics of the box girder of the bridge (Fig. 2-4 and 2-5).

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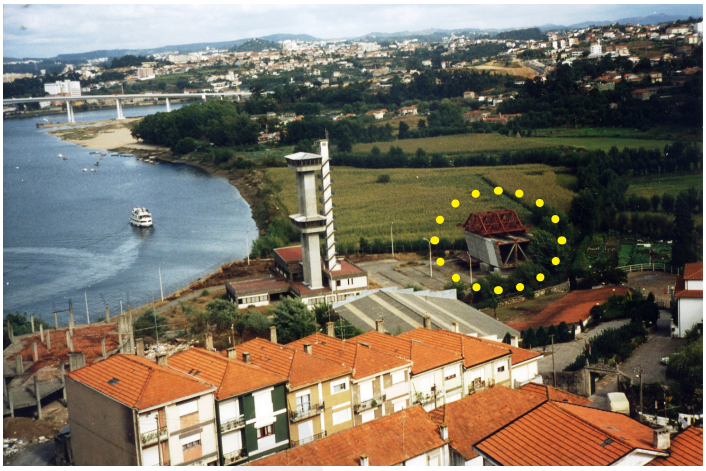


Fig. 2-4 View of the full scale experimental structure from the top of the bridge.



Fig. 2-5 Coated reference surface at the full scale experimental structure.

During the painting phase this experimental structure was used to make a coated trial that latter allowed the quality control and the acceptance of the works by the inspection.

### 2.8.4 Description of the intervention works

The painting works started in September 1990 and lasted for about 3 months. In order to define the surface preparation and the application method, a reference surface was prepared in a part of the structure approved by the relevant parties.

The cleaning and preparation of the concrete surface was made by blasting with water and sand in order to obtain a clean and rough substrate to promote the coating adhesion. However, it was observed that after this process the surface presented large pores and cavities that needed to be treated in order to guarantee an even and continuous coating film. A cement mortar was used for that purpose.

Since most of the structure was situated in an urban area with buildings and cars, it was decided to discard the use of the airless pistol method for the painting in order to avoid problems of contamination due to the transport of paint particles by the wind.

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After several experiments of using manual instruments to apply the paint in order to obtain at least 180 µm, it was decided that the better method was to apply three coats, two by brushing and the last one by roller.

These experiments (treatment and application method) were made first at the reference surface and, after approval by the parties, in all the concrete structure. Control of the thickness was made by the wedge cut method (ISO 2808 – method 6B) using a paint inspector gauge instrument. In order to facilitate the application control of the three approved coats, the second coat had a different colour – grey.

### 2.8.5 Description of maintenance operations

In order to predict maintenance operations concerning the need for repainting, from the end of the painting works in 1990 until 2008, the performance of the coating was followed by taking samples (cores) from the experimental structure.

The cores were obtained from painted and unpainted areas of the full-scale experimental structure (Fig. 2-4 and 2-5) in order to obtain the carbonation depth and the level of chlorides at two depths from the surface (0.5 cm and 1 cm). Fig. 2-6 shows the operation of taking some cores. After those operations, the holes were filled with a cement mortar. Several cores taken at different times are visible in the image. In the last analysed samples of the painted cores no significant carbonation or chlorides were obtained. Concerning the unpainted cores, also analysed for comparative purposes, the carbonation depth was a few millimetres higher but the level of chlorides was similar to the painted cores and at the level determined before, in 1990. The influence of the presence of the coating to protect concrete against carbonation would be more visible with a more porous concrete, as was confirmed by carbonation laboratoriy accelerated tests in which a concrete was used with a water cement ratio W/C = 0.70 painted with different types of coatings, including the paint used at the S. João bridge.



Fig. 2-6 Taking core samples for laboratory tests.

In 2008 some localised maintenance actions were carried out at the structural level in some of the piers that were repainted then. It is noteworthy that the remaining concrete elements that have not been repainted, maintain the initial coating for over 30 years.

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However due to the natural ageing of the film, mainly by chalking, after all these years, it is expected that the whole structure of the bridge needs to be repainted not only for aesthetical reasons but also to maintain the protection.

### 2.8.6 Concluding remarks

A coating system was applied more than 30 years ago and a recording of carbonation and chloride ion penetration was performed in order to follow the pathology state of the bridge. This is a necessary and very useful procedure for important civil engineering works, such as the one presented in this case study, in particular because the bridge is situated in an aggressive environment. From the results, it seems that the coating served its purpose as the long-term serviceability was achieved. The chalking effect observed was expected after all those years. The application of a new generation product and the recording of its behaviour were recommended to the owner.

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# 3. Application of membranes

## 3.1 Foreword

Membranes are used as a concrete surface protection method that leaves a layer which is impermeable to, for example, water, moisture, chloride, carbon dioxide and other chemical substances. It will be considered the following two types of membranes:

1. membranes that may be applied in liquid form - liquid applied membranes (Fig. 3-1a);
2. membranes in the form of sheets or rolls - prefabricated membranes (Fig. 3-1b).



a) b)

Fig. 3-1 Type of membranes: (a) liquid form; (b) prefabricated.

Membranes can be applied directly to the substrate or may be part of a build-up system including other layers (e.g. a vapour control layer).

The vast quantity of products or combination of products and techniques that exist, means it is not possible to cover all of them in the present text. In addition, new products and protection methods are constantly being developed and introduced into the market.

This chapter was written by Cabral-Fonseca (main author), J. Grandão Lopes, and M. Paula Rodrigues.

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*Surface protection of concrete is necessary to protect the structure from damage due to harmful substances that can attack and deteriorate concrete. In fact, beside the durability of concrete in most natural environments, it is sometimes used in areas and applications where this material is exposed to a harmful environment. In such situations the surface of concrete must be protected against the ingress of aggressive substances in order to improve its chemical resistance and consequently its durability. The protection reduces or prevents penetration of aggressive harmful substances (e.g. water and other liquids, vapor, gases or chemicals). There are several surface protection methods, namely the impregnation of concrete surfaces, the use of coatings and the use of barrier systems such as the application of membranes. The last method is the subject of the current section.*

## 3.2 Basics

### 3.2.1 When to adopt this method

Under most circumstances, the main reason for providing a surface protection system to a concrete structure is to extend its lifetime. Sometimes this protection is made as part of a more extensive repair strategy of the structure and its function is to reduce the rate of a deterioration mechanism already established.

The application of membranes, as a concrete surface protection method, could be understood as a means to create a barrier on the surface to meet one or a combination of the following requirements:

1. protection against ingress of moisture, water, chloride ions and other contaminants into the concrete;
2. improvement of the resistance to other deleterious influences, like chemical substances;
3. increase of the surface resistance against abrasion or impact.

Besides a protection against ingress, membranes may also be used to comply with the Principles 2 (moisture control), 6 (resistance to chemicals) and 8 (increasing resistivity), defined in EN 1504-9.

Since the application of membranes is not specifically regulated in the EN 1504 series, the required performance must be specified as the basis for the selection of products. In addition, materials and techniques selected should be consistent with the intended use.

Many concrete structures are designed and built to contain water within the structure (swimming-pools, water reservoirs, etc.) or maintain dry conditions within the structure when it is subjected to wet conditions on the outside (roofs, bridge decks, etc.). Since concrete is porous in nature and prone to develop cracks, it may be necessary to cover the surface with a barrier material, such as a membrane. In the present document, such membranes will be identified as waterproofing systems.

On the other hand, some concrete structures may be subjected to chemical attack and subsequent loss of integrity. In those cases, it is also necessary to cover the surface with a barrier material, such as a membrane. In the present document such membranes will be identified as chemical protective systems.

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*Comments:*

*According to EN 1504-9, there are eleven different principles of repair and protection from damages related to defects in concrete (Principles 1 to 6) and damages related to reinforcement corrosion (Principles 7 to 11):*

- *Principle 1: Protection against ingress*
- *Principle 2: Moisture control*
- *Principle 3: Concrete restoration*
- *Principle 4: Structural strengthening*
- *Principle 5: Physical resistance / surface improvement*
- *Principle 6: Resistance to chemicals*
- *Principle 7: Preserving or restoring passivity*
- *Principle 8: Increasing resistivity by limiting moisture content*
- *Principle 9: Cathodic control*
- *Principle 10: Cathodic protection*
- *Principle 11: Control of anodic areas*

### 3.2.2 Materials and systems

As mentioned in the previous section, a membrane is a barrier placed on the concrete surface. In practice, the membrane is only one component of a system composed of a number of materials having different functions, such as adhesive promotors, protective layers, etc.

Several new systems are available for which almost no service experience exists. Although it may be encouraging to incorporate such systems into a project, it is important to recognize that with unproven systems more risks and potential problems can happen.

Fig. 3-2 shows the main membrane systems used in concrete protection.

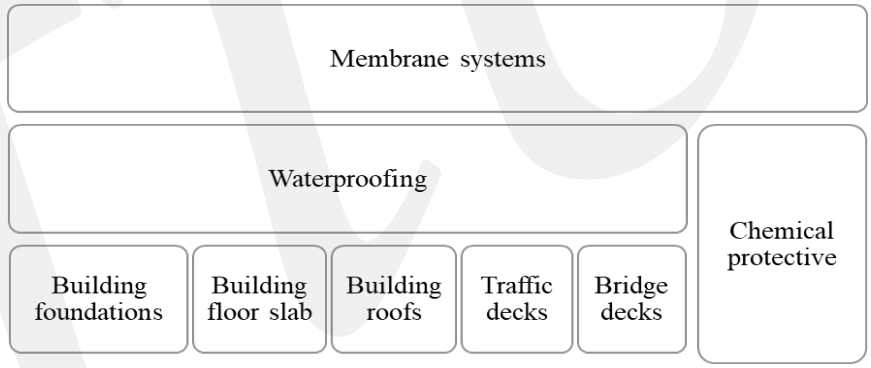



Fig. 3-2 Main membrane systems used in concrete protection.

#### 3.2.2.1 Waterproofing systems

Waterproofing systems are the most appropriate type of barrier to prevent the ingress of liquid water under hydrostatic pressure.

Usually, waterproofing membranes used in those systems usually can either surround the entire positive side (the same side of the applied pressure) or the negative side (the opposite side of the applied pressure) of the structure, forming a continuous barrier. In several applications, waterproofing systems are used to also prevent the ingress of liquid water in absence of hydrostatic pressure (sometimes this type of membranes is called “damp-proofing membranes”).

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#### *3.2.2.1.1 Building foundations*

Prefabricated membranes are used in building foundations and can be divided into three categories, according to the type of material: bituminous (rubberized asphalts), polymeric (thermoplastic) and elastomeric (vulcanized rubber). Liquid applied membranes, such as urethanes, rubbers, plastics and modified asphalts are also used.

For each type of foundation – footings, mat-slabs, pile heads and caps - different waterproofing systems can be used. Liquid applied membranes (e.g. liquid cold elastomeric modified polyurethane), self-adherent membranes (e.g. modified polyurethane roller and trowel) and multi-sheet systems may be used in backfilled and blindside wall applications.

The variety of solutions and the lack of universal technical information prove to be a great difficulty to the designers.

A few standards exist, but only for particular applications or types of products (e.g. ASTM D 7832, that lists test methods intended to establish a minimum level of acceptable performance attributes for waterproofing membranes applied to below-grade walls or ASTM D 6134 that covers unreinforced vulcanized rubber sheets made from ethylene propylene diene terpolymer (EPDM) or butyl (IIR), intended for use in preventing water under hydrostatic pressure from entering a structure).

BS 8102, from UK, is the "Code of practice for protection of below ground structures against water from the ground" and is a design document whose purpose is to inform the designer about the various methods of waterproofing available and to assist in the correct specification of those systems.

*Comments:*

*According to ASTM D 7832, the following attributes are required for a waterproofing membrane to be used in below-grade walls (vertical surfaces):*

- *perform under continuously moist conditions and alternate wetting and drying;*
- *perform under hydrostatic pressure;*
- *resist acids, alkalis and other chemicals including those commonly contained in fertilizers and soil poisoners;*
- *resist fungus and bacteria from soils;*
- *possess low water absorption;*
- *resist puncture;*
- *possess a moderate degree of elongation and elasticity;*
- *possess sufficient flexibility;*
- *possess crack bridging capabilities;*
- *adhere tenaciously to the substrate.*

#### *3.2.2.1.2 Building floor slab*

Because it is important to consider the waterproofing systems for all sides of a building, there are waterproofing systems for several functions, such as, below-slab-on-grade application that protect the slab from the intrusion of groundwater.

#### *3.2.2.1.3 Building roofs*

The waterproofing membrane is considered to be the most important component of the roofing system as it serves the main function of keeping water out of the building. Roof membranes can be liquid- applied membranes or prefabricated membranes.

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They are most commonly made from modified bitumen (with styrene-butadiene-styrene - SBS or atactic polypropylene – APP, e.g.), synthetic rubber (ethylene propylene diene methylene – EDPM, e.g.) or thermoplastic (polyvinyl chloride – PVC, e.g.). There is a large number of American Standards applied to roof systems that use membranes (a complete list can be found in the following address: https://www.astm.org/Standards/roofing- standards.html). However, only few design and application guides are available (e.g. Manuals from the American National Roofing Contractors Association - NRCA, and Application Guidelines from Single Ply Roofing Industry – SPRI).

In Europe, the European Organization for Technical Approvals (EOTA) has a Guideline for European Technical Approval (now used as European Assessment Document) of "Liquid Applied Roof Waterproofing Kits" (ETAG 005, 2004). This document gives guidance for the assessment of the fitness for use, including the requirements, methods of verification and the methodology for assessing and judging the fitness of products for the intended use.

*Comments:*

*ETAG 005 has the following parts:*

- *Part 1: General*
- *Part 2: Polymer modified bitumen emulsions and solutions*
- *Part 3: Glass reinforced resilient unsaturated polyester resin*
- *Part 4: Flexible unsaturated polyesters*
- *Part 5: Hot applied polymer modified bitumens*
- *Part 6: Polyurethanes*
- *Part 7: Bitumen emulsion and solutions*
- *Part 8: Water dispersible polymers.*

#### *3.2.2.1.4 Traffic decks*

An example of floor protection by membranes is their installation on parking garage decks. These concrete structures are sometimes exposed to harsh environmental conditions (exposure to de-icing salts and other chlorides, high humidity levels and carbonation that can lead to corrosion of the steel reinforcement).

A typical waterproofing membrane system consists of a series of coats or layers: (i) first, the primer or sealer, which promotes adhesion of the membrane to the concrete surface; (ii) next, the waterproofing membrane itself, which bridges cracks and prevents the ingress of chloride ions and water; (iii) then, comes the wear coat, which contains embedded aggregate to prevent wear of the membrane; (iv) and finally, the tie coat, which bonds the aggregate firmly to the wear coat.

The types of materials used in those membrane systems include polyurethane, polyurea and epoxy resins. The International Concrete Repair Institute (ICRI) has a Technical Guideline for horizontal waterproofing of traffic surfaces. Several companies have their own manuals for waterproofing solutions for traffic decks.

There are some American Standards which often contain specifications and test methods for cold liquid-applied elastomeric waterproofing membranes for use with separate wearing course or with integral wearing surface, applied to building walls and decks with and without hydrostatic pressure (ASTM C 836 and ASTM C 957).

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*Comments:*

*The requirements defined in ASTM C 836 are: hardness; weight loss; non-volatile matter content; low temperature crack bridging; film thickness; adhesion-in-peel after water immersion; extensibility after heat aging; and storage stability.*

*The requirements defined in ASTM C 957 are: weight loss of base coat; low temperature crack bridging; adhesion-in-peel to cement mortar and plywood substrates after water immersion; chemical resistance after water, ethylene glycol, and mineral spirits exposure; weathering resistance, recovery from elongation, tensile retention, and elongation retention; abrasion resistance; and stability.*

#### *3.2.2.1.5 Bridge decks*

One of the most important causes of the deterioration of concrete bridge deck is the penetration of moisture and chlorides into the concrete with subsequent corrosion of the steel reinforcement. In concrete bridge decks, this phenomenon becomes more severe since bridge decks suffer from heavy traffic loading, which enlarges the cracks that allow the ingress of chlorides into the concrete.

The use of waterproof membranes is one strategy to prevent moisture and chloride penetration, providing a barrier on the top of the concrete deck. The waterproofing membrane is then protected from the traffic by an asphalt overlay.

Waterproofing membrane systems used in bridge decks generally consist of either prefabricated sheet systems or liquid systems:

1. Prefabricated sheet membranes are often rolled into place and bonded to the concrete deck using a pressure-sensitive adhesive on the sheet or through the use of heat (by torch, when bituminous membranes are used) to bond the membrane to the concrete deck; materials used to form the sheet membrane are described by the manufacturers e.g. rubberized asphalt, bituminous membrane, polymer-modified asphalt, modified bitumen and polymeric membrane.
2. Liquid membranes are applied as either hot or cold liquids (bituminous and resinous liquid-sprayed systems) and may include a layer of reinforcing fabric; manufacturers describe the material used for liquid systems as e.g. rubberized asphalt, two-components polymer, polyurethane, methyl methacrylate, rubber polymer, polymer-modified asphalt or rubberized bitumen.

The American National Cooperative Highway Research Program (NCHRP) has a report that presents a synthesis on Waterproofing Membranes for Concrete Bridge Decks (Synthesis 425, 2012). This document includes information on materials, specification requirements, design details, application methods, system performance, and costs of waterproofing membranes used on new and existing bridge decks since 1995.

ASTM D 6153 specification covers deck waterproofing membrane systems designed for use on bridge decks which will receive an asphaltic concrete overlay. It deals with cold liquid-applied chemically curing membrane materials, hot applied elastomeric membrane systems and prefabricated sheet membrane systems, applied to prevent the passage of water from the bituminous surface course to or through the bridge deck.

*Comments:*

*The physical requirements defined in ASTM D 6153 are: water permeability, pliability. Other performance requirements are: water vapour transmission, adhesion to concrete, tensile strength, minimum elongation at break, crack bridging, puncture resistance.*

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In Europe, EOTA (European Organization for Technical Assessment) has a Guideline for European Technical Assessment (now used as European Assessment Document) of "Liquid Applied Bridge Deck Waterproofing Kits" (ETAG 033, 2010). This document gives a guidance for the assessment of the fitness for use, including the requirements, methods of verification and the methodology for assessing and judging the fitness of products for the intended use.

*Comments:*

*Some of the requirements specified in the ETAG 033 are:*

- *bond strength (assembled system to support);*
- *capacity to bridge cracks;*
- *resistance to chloride ion penetration;*
- *resistance to dynamic actions;*
- *resistance to heat impact;*
- *resistance to perforation;*
- *resistance to shear (assembled system to support);*
- *water-tightness;*
- *compatibility of materials in contact.*

*Regarding durability, ETAG 033 states that the assembled system shall have sufficient resistance to the effects of the following deleterious actions:*

1. *contact with water, alkali, oil, petrol, diesel, de-icing salts and bitumen;*
2. *climatic conditions, including effects of heat ageing and freeze/thaw;*
3. *UV radiation (when applicable);*
4. *abrasion and wear.*

### 3.2.2.2 Chemical protective systems

Chemical protective systems are used to protect concrete from degradation by chemical and subsequent loss of cover concrete or structural integrity, to prevent staining of concrete, or to protect liquids from being contaminated by concrete (for example in water treatment facilities).

On one hand, the damage to concrete structures may vary in intensity from surface discoloration to catastrophic loss of structural integrity according the service conditions and the type of aggressive agent. On the other hand, a variety of protective systems is available. So the choice of the appropriate protective system can be difficult.

The American Concrete Institute published a guide to select protective treatment for concrete (ACI 515.2R-13). This document gives the recommended treatment methods for protection of concrete from various aggressive substances. To help in the selection process, the protective systems are divided into three general categories according to the severity of the chemical service conditions: (i) mild, (ii) intermediate and (iii) severe. The application of membranes is recommended for mild and in some cases to intermediate level of chemical severity. Performance requirements methods and test are not defined.

In Europe, EN 14879-3 and EN 14879-5 specify the requirements for organic coatings (polymeric membranes) and linings (prefabricated membranes) which are applied to concrete process engineering equipment that will come in contact with aggressive chemical substances. In particular, those standards specify requirements for the design of concrete

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parts used in process plant units to which a protective coating and/or a protective lining is to be applied.

Amongst other, the following materials/systems selection criteria must be considered:

1. exposing media, particularly concerning aggressive substances;
2. type and frequency of fluid loading (if applied);
3. thermal loading and changes in temperature;
4. mechanical loading;
5. weather factors (if applied);
6. additional specific requirements.

### 3.2.3 Techniques

#### 3.2.3.1 Liquid applied membranes

Liquid applied membranes are applied on site in a liquid form and can be water or solvent based. Liquid membranes can be applied in a single or two components system, with or without heating. Sometimes, for special applications, it is necessary to build a multilayer membrane, with the application of a sequence of different products. Liquid applied membranes are usually applied by brush, roller or spray. Careful supervision and control during application is needed, particularly in ensuring consistent thickness and uniform application and correct intervals between coats; to guarantee these conditions when liquid applied membranes are applied by spray, the air speed should be limited to certain values (1.5 m/s can be taken as a reference).

As with most coating applications, surface preparation of the substrate is one of the most important steps, to ensure a good long-term performance. Requirements for surface preparation can typically be found in manufacturer's application guides and specifications.

#### 3.2.3.2 Prefabricated membranes

These membranes arrive at the site in the form of rolls or sheets. According to their type, prefabricated membranes may be stuck to the concrete following the manufacturer's recommendations.

Bitumen-based membranes for waterproofing applications are generally of two types: self-adhesive (bitumen with a layer of polyethylene) and torch-on membranes (bitumen modified with SBS or APP).

Thermoplastic membranes are usually applied with hot air on lap joints, because the membrane becomes soft and pliable when subjected to high temperature, making it suitable for fusing the material on the joints. Usually, this type of membrane is applicable as a loose laid or a mechanically fixed system.

### 3.2.4 Equipment and tools

Different tools and specific equipment are required for the application of membranes. Details of the equipment and tools, as well as method of application of the system, must be included in the design project.

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The application equipment and skills for different applied membranes are quite different. For this reason, the contractor should ensure that his technical staff was trained in all the intended operations. In addition, regular equipment maintenance and cleanliness are vital to ensure the correct membrane application.

#### 3.2.4.1 Liquid applied membranes

Liquid systems generally consist of application of a primer followed by application of the liquid membrane. The membrane may be placed using either spray equipment or rollers and squeegees. The membranes are either hot or cold applied depending on the manufacturer's requirement.

#### 3.2.4.2 Prefabricated membranes

Prefabricated membrane systems using bituminous membranes fully adhered to the concrete substrate, usually involve the application of a primer to concrete to improve the adhesion of the membrane.

This is followed by the installation of the membrane. Some prefabricated systems include a self-adhesive backing on the membrane sheet; these sheets can be rolled into place and then bonded using a roller. In other systems, the membrane is bonded by heating using either a hand torch flame gun or a machine.

## 3.3 Stakeholders' roles and qualifications

### 3.3.1 Owner

The following factors must be considered, by the owner, in the definition of the protection and/or repairing strategy: (i) intended design life-time, following repair and protection; (ii) required durability and performance; (iii) need of future inspection and maintenance work during the defined service life of the structure.

In rehabilitation work, the previous structure survey and its assessment shall be carried out by a suitable qualified and experienced team, in order to identify defects and perform an in-depth condition analysis of defects to address the causes of the damage. Based on this assessment, the owner has a number of options to adopt while deciding the relevant actions to meet the future requirements of the structure and its remaining service life.

Owners should also evaluate the capabilities of the potential designers and contractors to use specialised concrete protection materials and techniques and to successfully execute the surface repair procedures.

### 3.3.2 Designer

Points that should be considered by the designer in selecting a particular solution using membrane systems include:

1. document review, namely technical information and guides provided by the membrane system's manufacturer;
2. test results already done with specific possible solutions;

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3. an almost full characterisation of the concrete structure – in the case of a repair, it is important to have information about the condition survey of the structure;
4. exposure conditions, such as temperature and humidity profiles and any special conditions as the presence of chemicals or freeze/thaw cycles, e.g.;
5. expected effectiveness of the protection;
6. estimated service life;
7. life-cycle cost analysis;
8. periodic inspection and maintenance requirements.

Protection systems with membranes should be designed by a specialist who can demonstrate that has a suitable level of knowledge on products, systems and on selection of the solution for each particular case.

Designers should always use normative references and existing Codes of Practice in their project specification. Within Codes of Practice there are suggestions, but it is important to recognize that every project is different and all details should be completed in accordance with the specified manufacturer's instructions.

When selecting a product or system, it is also important to consult the supplier and/or manufacturer to confirm what support they offer, including: design advice, site support, training for designers and applicators and post-application support.

### 3.3.3 Contractor

One of the most important aspect of a project involving application of membranes is the selection of a qualified contractor. It is also important to recognise that few contractors are proficient in all phases of all types of procedures. If possible, contractors who have shown evidence of expertise and successful completion in the specific type of surface protection technique, shall be selected.

### 3.3.4 User

Users should avoid any actions that could conflict with the performance of the protection membranes, as well as factors that could accelerate their degradation.

In addition, users should have a procedure for routine maintenance and which may facilitate future inspections to be carried out as part of the ongoing management of the structure. All documentation regarding periodic maintenance and inspection must be kept.

## 3.4 Design

### 3.4.1 Assessment of existing conditions

Prior to the development of a surface protection system, an assessment of the existing concrete structure must be done according to current codes or guidelines, e.g. ISO 13822 or ACI 364.1R-19. This aspect is particularly important when the protection is to be applied under a repairing strategy, since the concrete could have ongoing deterioration mechanisms.

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In a rehabilitation project, documents related to the structure's service history should be reviewed to learn as much as possible about the maintenance, damage, deteriorations and subsequent repairs that eventually took place.

### 3.4.2 Service life

The service life of a potential protection system should be evaluated before selecting a material and a technique. Among the factors to consider in the selection of a membrane to protect concrete are environmental concerns, such as exposure to wind-driven rain, ultraviolet radiation from the sun, temperature variations, presence of harmful chemical substances, abrasion or any other potentially destructive conditions.

### 3.4.3 Reliability requirements

Reliability requirements for each particular system should be included in the technical specifications.

### 3.4.4 Codes and standards and other relevant references

- ACI 364.1R-19: Guide for assessment of concrete structures before rehabilitation (2019).
- ACI 515.2R-13: Guide to selecting protective treatments for concrete (2013).
- ACI 546-R-14: Guide to Concrete Repair (2014).
- ACI 562-19: Code requirements for assessment, repair and rehabilitation of existing concrete structures and commentary (2019).
- ASTM D 7832: Standard guide for performance attributes of waterproofing membranes applies to below-grade walls/vertical surfaces (2014).
- ASTM C 836: Standard specification for high solids content, cold liquid-applied elastomeric waterproofing membrane for use with separate wearing course (2018).
- ASTM C 957: Standard specification for high-solids content, cold liquid-applied elastomeric waterproofing membrane with integral wearing surface (2017).
- ASTM D 6153: Standard specification for materials for bridge deck waterproofing membrane systems (2015).
- BS 8102: Code of practice for protection of below ground structures against water from the ground (2009).
- EN 1504: Products and systems for the protection and repair of concrete structures. Definition, requirement, quality control and evaluation of conformity. Part 1: Definitions (2005); Part 2: Surface protection systems for concrete (2004); Part 9: General principles for the use of product and systems (2008); Part 10: Site application of products and systems and quality control of the works (2017).
- EN 14879: Organic coating systems and linings for protection of industrial apparatus and plants against corrosion caused by aggressive media. Part 1: Terminology, design and preparation of substrate (2005); Part 3: Coatings on concrete components (2006); Part 5: Linings on concrete components (2007).

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- ETAG 005: Guideline for European Technical Approval of Liquid Applied Roof Waterproofing Kits (2004) – Superseded by EAD 030350-00-0402
- ETAG 033: Guideline for European Technical Approval of Liquid Applied Bridge Deck Waterproofing Kits (2010).
- ICRI – Technical Guideline 310.2R: Selecting and specifying concrete surface preparation for sealers, coatings, polymer overlays and concrete repair. International Concrete Repair Institute (2013).
- ICRI – Technical Guidelines 710: Horizontal waterproofing of traffic surfaces. International Concrete Repair Institute (2014).
- ISO 13822: Bases for design of structures - Assessment of existing structures (2010).
- ISO 16311-1: Maintenance and repair of concrete structures. Part 1: General principles (2014). Part 2: Design of repairs and prevention (2014). Part 4: Execution of repairs and prevention (2014).
- NCHRP Synthesis 452: Waterproofing Membranes for Concrete Bridge Decks. Transportation Research Board. Washington, D.C. (2012).
- NRCA: The NRCA Roofing and Waterproofing Manual, National Roofing Contractors Association (2006).
- SPRI: Application Guidelines (https://www.spri.org/resources- education/), Single Ply Roofing Institute.

### 3.4.5 Design assumptions

Design assumptions strongly depend on the type of products, protection method used and in-service conditions of the structure. The need to establish a cohesive team of professionals to assist with successful planning design and execution of the project is highlighted.

### 3.4.6 Design procedure

When selecting a protection method using membranes, all factors affecting the performance of that system on the concrete structure need to be considered collectively and not just in each individual components. Some of the factors that should be considered include:

- membrane material and system;
- interface between membrane and concrete surface;
- structural considerations;
- ambient environment;
- presence of water and pressures (if applicable);
- chemical exposures;
- exposure to abrasion or erosion (if applicable).

It is important to ensure compatibility between the intended protection method, the materials involved and the existing concrete substrate. Proper concrete surface preparation is a key element for the success of a protection or repair project using membranes. The

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guidelines from International Concrete Repair Institute (ICRI) provides the tools needed to select and specify the methods of preparing concrete surfaces prior to the application of a protective system or repair material (Guideline No. 310.2R-2013).

Each type of protection systems involving membranes has its own characteristics. The selection of the appropriate solution for a specific project, should take in account the following aspects:

1. Installation requirements:
   - clean, dry and sound concrete surface (levelled surface, if required);
   - sprayed, brushed, rolled or squeegeed (for liquid-applied membranes);
   - ambient conditions (air temperature and relative humidity, superficial humidity of concrete, etc.);
   - availability of skilled and experiences applicators.
2. Durability characteristics:
   - UV resistance (for outdoor applications);
   - chemical resistance (if applicable);
   - abrasion resistance (if applicable).
3. Performance behaviour:
   - water and vapour permeability;
   - crack-bridging properties;
   - skid and slip resistance (if necessary);
   - appearance (if relevant).

In conjunction with the selection of the membrane protection system, plans should be drawn up to establish a suitable inspection program to supervise the surface preparation and application work, ensuring that the membrane is applied according to the specifications.

### 3.4.7 Supporting documents

All documents used and produced in the design process must be carefully organised and stored, for consultation in the following steps.

## 3.5 Execution

### 3.5.1 Preparatory works

Before the application of a membrane for concrete protection, particularly when included in repair work, it is necessary to perform the following tasks:

1. repair surface defects;
2. stopping or rerouting of water;
3. surface preparation, according to the membrane manufacturer´s recommendations.

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Surface preparation requirements may vary with the protection system selected. The manufacturer of the system may have specific requirements for surface preparation, including the surface profile and moisture sensitivity.

*Comments:*

*Methods of concrete surface preparation:*

1. *chemical cleaning;*
2. *mechanical cleaning;*
3. *impact tools and scrabbles;*
4. *scarifiers;*
5. *blast cleaning;*
6. *sand blasting;*
7. *shot blasting;*
8. *water blasting;*
9. *acid etching;*
10. *flame cleaning.*

For liquid-applied membranes, the concrete surface should be relatively smooth, because most products are not intended to level off or hide imperfections of the substrate. The quality of the concrete surface should be evaluated before installing the liquid-applied membrane. It should have adequate strength and be free of contaminants that could affect the performance of the protection system.

For prefabricated membranes the concrete surfaces should be free of protrusions and rough edges.

### 3.5.2 System trials

For many projects it will be necessary to prove the membrane being applied in a site trial, in order to certify its compliance with the specifications and the requirements requested by the designer.

This is particularly true when the performance of a new membrane system has to be proven.

### 3.5.3 Execution procedure

The procedures used to apply membranes to protect concrete will vary with each system, and the applicator should be guided by recommendations of the manufacturer and information given in the design project.

#### 3.5.3.1 Liquid applied membranes

There are three types of problems that can affect the performance of a membrane system applied in liquid form:

1. application factors: poor mixing of the components; mixing of components at inappropriate temperatures; incorrect proportioning of the components and delayed application of the mixed components;
2. substrate conditions: moisture in the substrate and poor surface preparation;

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3. ambient weather conditions, particularly in outdoor application: speed of the air (for spray techniques), variations in air temperature, relative humidity and the presence of rain.

To avoid such problems, it is important to follow some practical recommendations:

1. manufacturer's specification on mixing and application must be followed;
2. avoid mixing liquid membrane components at low temperature (at low temperatures the increase in viscosity may cause improper mixing);
3. careful attention should be paid to the proportion of components;
4. poor mixing of the components must be prevented (the effect of poor mixing of some membranes can be drastic);
5. the membrane should be applied within the interval of time established by the manufacturer, after mixing;
6. proper concrete surface preparation (different methods of surface preparation produce different profiles that affect the adhesion of the membrane);
7. application in either high or low temperatures and high humidity conditions should be avoided (do not apply a membrane system if there is a prospect of rain).

#### 3.5.3.2 Prefabricated membranes

There are four types of problems that can affect the performance of a prefabricated membrane system:

1. insufficient width of lap joints;
2. substrate conditions: moisture in the substrate and poor surface preparation;
3. speed of movement of the torch equipment or the hot air gun;
4. application in either high or low temperatures and high humidity conditions should be avoided (do not apply a membrane system if there is a prospect of rain);
5. improper welding at lap joints.

### 3.5.4 Finishing

When necessary, protective coatings may be required. Compatibility with the membrane should be proved in advance. The chosen coating must be appropriate to the environment factors and in-service conditions of the structure.

## 3.6 Quality control

All the best efforts in designing and specifying the optimum protection or repair system are at risk if the implementation lacks the appropriate quality control to ensure successful installation. A quality assurance system should be proposed in the design specifications and implemented before, during and after the intervention.

The properties of the substrate, acceptance of products and the conditions for their application shall be subjected to quality control. The requirements are specific for each type of application but, in general, they should involve laboratory tests for material

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identification and assessment of its performance under special conditions (suggestions for test variations include different classes of exposure – such as such as aggressive environments, subject to high temperatures or temperature cycles, continuous immersion or alternate wet and dry cycles);

1. tests and checks during installation (e.g. adhesion test, such as pull-off);
2. control after intervention (e.g. visual inspection).

### 3.6.1 Quality control of materials

In principle, membrane systems should be certified and suppliers should therefore maintain a continuous production control system and perform characterisation tests for the material. Accordingly, all the relevant physical and mechanical characteristics should be provided and, whenever applicable, should fulfil the minimum required performance.

Membranes used to protect concrete for a specific chemical environment require an awareness of special items, such as: their resistance to deterioration by the chemicals to which they will be exposed at the operating temperatures or their resistance to thermal shocks caused by rapid changes in temperature. The type of membrane depends on the severity and aggressiveness of the environment, and the selection must be based on testing or past experience.

### 3.6.2 Quality control before intervention

A quality control for the preparatory work, such as repairing of surface defects and concrete surface preparation, must be implemented.

The air temperature, humidity and substrate temperature should be monitored before the membrane application. Depending on the membrane used, the application should be performed within the concrete moisture content limits indicated in the manufacturer's recommendation.

The weather conditions during application can determine the quality of the application, particularly for liquid membranes. In fact: (i) rain may wash away liquid membranes; (ii) sun and high temperature can result in shorter reaction times of liquid membranes and thus reduce the pot life and the time available for application; (iii) wind can increase water evaporation, especially in combination with high temperatures; strong winds can also cause problems with spray applications; (iv) during frost, material containing water must not be applied because it will freeze and thereby be destroyed.

### 3.6.3 Quality control during intervention

All construction details such as surface preparation of concrete, installation of the membrane system, ambient conditions, etc. should be in strict accordance with the requirements included in the project specification. For some systems, particularly in the case of liquid membranes, the air temperature, humidity and substrate temperature should be monitored before and during the membrane application.

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A good quality control during application of membranes will help detect failures that result from: (i) problems in application due to poor workmanship, lack of training and inadequate equipment and (ii) extended exposure to extreme environmental conditions, such rain, heat and UV radiation.

### 3.6.4 Quality control after intervention

Tests to be used as a basis for acceptance of the membrane system should be defined and described in detail in the specification. A repair procedure should also be included.

Quality control after intervention usually includes:

- frequent measuring of wet layer thickness during application of liquid membranes, as well as control of consumption;
- dry thickness measurement, in a test panel;
- visual examination of the surface during and after the application;
- testing of adhesion to the substrate (sometimes in a site trial panel);
- testing of welded seams for water-tightness, particularly in prefabricated waterproofing membranes;
- documentation of the work, based on established protocols, for example taking photos or filling check lists.

In some special cases, specific procedures are needed, such as:

- a visual inspection of the applied membrane (sometimes assisted by non-destructive testing);
- an integrity test in order to detect membrane's defects.

## 3.7 Monitoring and maintenance

Periodic visual inspections should be made (for example, annually) to provide an overview of the membrane's general condition and check for damage. A periodic inspection procedure should be implemented.

Since the preventive maintenance extends the service life of the membrane, and therefore the protection of the structure, a maintenance plan should also be implemented.

### 3.7.1 Monitoring

After execution, an "Inspection and Maintenance Plan" should be implemented, indicating the periodicity and type of actions to be performed. Periodical inspection should be reported, and any eventual deterioration should be evaluated.

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### 3.7.2 Maintenance

The concrete protected with the membranes, as well as unprotected concrete, requires regular maintenance to provide a satisfactory level of service and to meet service-life expectation without premature deterioration. The type and frequency of maintenance required are directly related to the types of deterioration problems involved and depends on the systems and conditions of use.

The actions recommended in the "Inspection and Maintenance Plan" should be executed at the suggested intervals. This plan can be reviewed at any time, in order to include further tasks and actions that have become necessary.

### 3.7.3 Post-intervention documentation

The owner should maintain accurate maintenance and inspection records to provide historical information that may assist in future deterioration and identification of potential problems.

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# 4. Corrosion inhibitors for steel rebars

## 4.1 Foreword

Corrosion inhibitors are a "chemical substance that, when present in the corrosion system at a suitable concentration, decreases the corrosion rate without significantly changing the concentration of any other corrosion agent" (ISO 8044).

Other definitions may be assumed. According to the mechanism of action, inhibitors may be adsorbed on the surface of steel, or chemically modify it, may cause a change in corrosion potential, or may induce passivity or may induce passivity (e.g. in Fig. 4-1).

This method addresses the protection and repair principle P11 - Control of anodic areas of the reinforcement (EN 1504-9).



Fig. 4-1 Corrosion inhibitor application in the rebars after concrete removal and cleaning.

## 4.2 Basics

### 4.2.1 When to adopt this method

Corrosion inhibitors are adopted when there is a risk of corrosion or when there is already damage caused by corrosion propagation, i.e., as a preventive or remedial corrosion measure.

Corrosion inhibitors are often used in conjunction with, or may provide an alternative to, other protection methods of reinforcement against corrosion. The low cost and easy application, with minimum intervention, of corrosion inhibitors is especially advantageous.

This method is applied to prevent or mitigate reinforcement corrosion caused by e.g. atmospheric pollution, chloride, carbon dioxide, and aggressive chemicals.

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This chapter was written by João Correia (main author), Paula, Paulo Costa, and Pereira.

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Where contamination of the concrete is extensive, but it is not possible to remove all concrete, incipient anode formation, which may be caused by differences in pH, chloride content, or defects, can be controlled by treating the surface of the reinforcement in patch repairs to prevent corrosion (ISO 16311-3).

Coatings can be applied directly to the reinforcement, where it is exposed as part of concrete restoration. These coatings can contain active pigments, which may function either as anodic inhibitors or by sacrificial galvanic action. Discrete galvanic anodes can also be installed at the perimeter of the patch to control the incipient anode (ISO 16311-3).

Other types of coatings can form barriers on the surface of the reinforcement (ISO 16311-3). This method can only be effective if the reinforcement is prepared to become free of corrosion and the coating is complete (i.e. the bar must be completely encapsulated and the coating defect-free). Alternatively, corrosion inhibitors can chemically change the surface of the steel or form a passive film over it.

The required concentration of anodic inhibitors is often determined by the amount of corrosive species (significant ratio of competing inhibitor/corrosive species). Concrete cracking has also an adverse effect on inhibitors' efficiency.

Performance of migrating corrosion inhibitors is often affected by obstacles to migration, either physical or chemical and thus it strongly depends on the quality of concrete. Therefore, the use of migrating inhibitors is not recommended in situations where its migration towards the reinforcement is prevented or inhibited (e.g. building components with previous repair with dense repair mortars, impermeable paint, etc.). Heterogeneous deterioration or varied concrete properties (e.g. cover depth and permeability) also affect the effectiveness of migrating inhibitors.

In the propagation phase, corrosion inhibitors may be limited to the degree to which active corrosion will be efficiently controlled. In severe conditions, e.g., extensive deterioration in marine environments, additional repair remedies may be required.

### 4.2.2 Materials and systems

The products and systems, which should be selected according to the project specification (ISO 16311-3), include corrosion inhibitors and mortars/coatings with inhibitors.

Corrosion inhibitors for concrete are classified according to their chemical nature (organic or inorganic), the procedure of application (mixed with concrete - admixed inhibitors; applied on hardened concrete - migrating inhibitors), and mechanisms of action. According to the electrochemical mechanism of action (anodic and cathodic principles illustrated in Fig. 4-2), inhibitors are:

- Anodic (passivating) – usually causing the formation of a film, they act on the dissolution reaction of steel, increasing its corrosion potential and, thus, reducing the corrosion rate;
- Cathodic (precipitation) – they act on the oxygen reduction reaction, by precipitating poor soluble salts on the steel surface which reduce the oxygen transport, reducing the steel rebar's corrosion rate by decreasing its corrosion potential;
- Mixed – they act on both anodic and cathodic sites, generally by surface adsorption to the steel surface forming a protecting layer, reducing the corrosion rate without a significant change in the corrosion potential.

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Products and systems, with properties documented by the specified test methods (ISO 16311-3), should be suitably stored before use, so that their properties shall not be impaired.

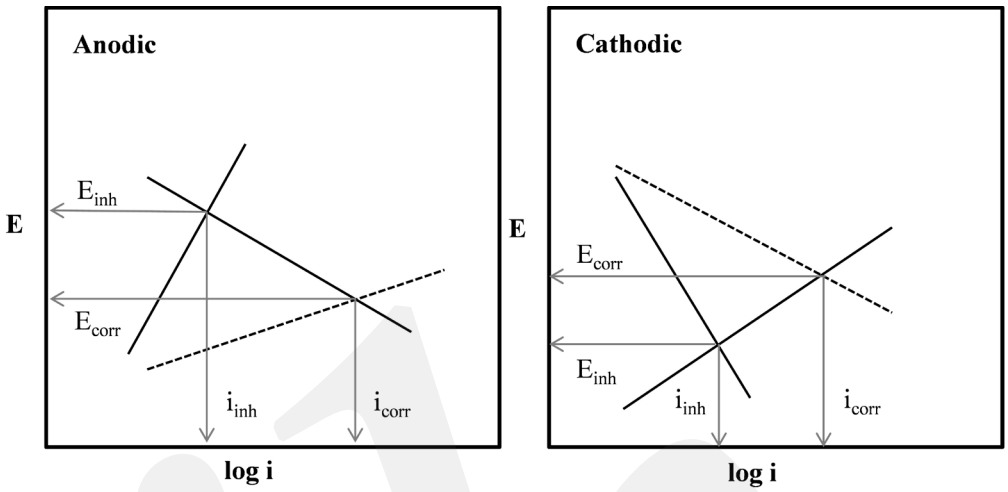



Fig. 4-2 Corrosion protection mechanism for anodic and cathodic inhibitors (adapted from Goyal A. el. al, AJSE (2018) 43:5035–5055).

### 4.2.3 Techniques

The application techniques of corrosion inhibitors are:

- admixed corrosion inhibitors - added to concrete or to repair product or system;
- penetrating corrosion inhibitors (also called migrating or surface-applied corrosion inhibitors) - applied on the hardened concrete surface followed by migration to the reinforcement by capillary action, vapour phase and ionic attraction;
- surface coating of rebars before concreting.

### 4.2.4 Equipment

No special equipment is generally required for admixed corrosion inhibitors. Penetrating corrosion inhibitors and coatings can be applied using conventional sprays, rollers or brushes. In all cases, the equipment should satisfy the manufacturer's recommendations.

## 4.3 Stakeholders' roles and qualifications

### 4.3.1 Owner

Owner requirements may include the remaining service life and performance of a structure, the likely service life of the repair works, the availability of the structure, the number and cost of repair cycles during the structure design life, the appearance of the repaired structure, and health and safety requirements (EN 1504-9).

The owner should deliver information on the existing structure, including the original project specifications and drawings, as well as documents referring to subsequent interventions (EN 1504-9).

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### 4.3.2 Designer

A qualified team with adequate knowledge on design, maintenance, and repair of concrete structures is required.

### 4.3.3 Contractor

The repair should be carried out by experienced and qualified repair contractors, having the required knowledge, skill, equipment and resources to execute the work, according to the requirements.

### 4.3.4 User

The user is recommended to implement a maintenance strategy (EN 1504-9, ISO 16311-1), with special attention to the corrosion control and monitoring plan.

## 4.4 Design

Design for the repair shall be formulated based on maintenance history, condition assessment, remaining service life, and projected structural performance (ISO 16311-1).

### 4.4.1 Assessment of existing conditions

An assessment shall be carried out to verify the structure's durability, performance, and reliability (ISO 16311-2). An appropriate assessment plan includes the planning of methods and procedures for investigation, assessment and prediction of deterioration progress, evaluation of the structure's condition and maintenance history, decision-making on remedial action, and recording (ISO 16311 parts 1 and 2).

Possible critical situations for the structure, identified by scenarios to represent a possible change in performance (ISO 16311-1), shall be specified (e.g. damage related to corrosion).

The defects, including their extent and expected rate of increase, and causes, as well as their consequences on the ability of a structure to perform its function, should be assessed (EN 1504-9).

A typical assessment would include visual inspection (general recording of visual defects like spalling, cracks, reinforcement corrosion, rust efflorescence, and delaminating concrete), reinforcement cover, carbonation depth, sampling to determine the chloride ion content and profile and the presence of other deleterious substances, cores for chemical analysis, and electrochemical testing (EN 1504-9, ISO 16311-2).

As deterioration and/or change in performance of the structure are affected by environmental conditions and/or physical actions, investigations shall also be conducted on actions and environment (ISO 16311-1, ISO 2394). Moreover, local changes in the structure, as well as the need or opportunity for further protection, should also be considered (EN 1504-9).

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As part of the structural appraisal, the properties of the concrete, the reinforcement detailing, and the effects on the remaining load capacity of deterioration and condition may be needed (EN 1504-9).

The assessment should also include the risks to health and safety, including the effect of deterioration upon the stability of the structure and potential impacts of materials and methods of repairs (EN 1504-9, ISO 16311-3).

The details and extent of data shall meet the requirements, so that the performance, durability, and remaining service life can be verified (ISO 16311-2).

### 4.4.2 Service life

Critical factors for the service life are (ISO 16311-1):

- performance criteria and areas of concern;
- assessing deterioration mechanisms and their combined effects, condition/ consequence level, deterioration rates and progress and its effects on performance.

Corrosion inhibitors can be applied for restoring the design service life of the element/ structure in a single operation, for repeated maintenance or reapplied repair (ISO 16311-3).

Corrosion inhibitor repairs have usually a reported service life from 10 to 15 years. This may not be verifiable, because the long term performance of inhibitors is influenced by their properties, mechanism of action, and other factors related with environmental exposure conditions and concrete properties. Currently, there is still a lack of commonly established results and recognized international testing procedures regardless of the latest developments. Thus, manufacturers' performance data is in most cases the single related available information on performance, especially due to proprietary issues of commercially available inhibitors and to the limited published work in the literature.

Deterioration and side effects of corrosion inhibitors use (e.g. inhibitors leaching, consumption or evaporation and adverse chemical or physical interactions with the substrate) may change either the inhibitors' effectiveness with time or the concrete performance (e.g. low concrete compressive strength, erratic setting times, efflorescence, and enhanced alkali aggregate reaction).

Prediction of the long term performance of inhibitors needs a clear understanding of the mechanism of corrosion inhibition, as well as the identification of all influencing factors. Thus, supporting expertise in corrosion assessment, control, and prevention is recommendable.

### 4.4.3 Reliability requirements

The structural performance during the remaining service life shall be verified with reference to the required thresholds, determined in accordance with the requirements from the government (e.g. standards, codes), owner, designer, user, etc. (ISO 16311-1).

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### 4.4.4 Codes and standards and other relevant references

- ASTM G109 - 07(2013). Standard Test Method for Determining Effects of Chemical Admixtures on Corrosion of Embedded Steel Reinforcement in Concrete Exposed to Chloride Environments.
- ASTM G180 - 13(2019). Standard Test Method for Corrosion Inhibiting Admixtures for Steel in Concrete by Polarization Resistance in Cementitious Slurries.
- ASTM C1582 / C1582M - 11(2017). Standard Specification for Admixtures to Inhibit Chloride-Induced Corrosion of Reinforcing Steel in Concrete.
- Elsener, B. Corrosion Inhibitors for Steel in Concrete: State of the Art Report, EFC 35, Maney Publishing, 2001.
- EN 1504-7:2006. Products and systems for the protection and repair of concrete structures. Definitions, requirements, quality control and evaluation of conformity. Reinforcement corrosion protection.
- EN 1504-9:2008. Products and systems for the protection and repair of concrete structures. Definitions, requirements, quality control and evaluation of conformity. General principles for use of products and systems.
- EN 1504-10:2017. Products and systems for the protection and repair of concrete structures. Definitions, requirements, quality control and evaluation of conformity. Site application of products and systems and quality control of the works.
- EN 1990:2002. Eurocode. Basis of structural design.
- EN 1992-1-1:2004. Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings.
- EN 1992-1-2:2004. Eurocode 2: Design of concrete structures - Part 1-2: General rules - Structural fire design.
- EN 1992-2:2005. Eurocode 2: Design of concrete structures - Part 2: Concrete bridges - Design and detailing rules.
- EN 1992-3:2006. Eurocode 2: Design of concrete structures - Part 3: Liquid retaining and containment structures.
- EN 13670:2009. Execution of concrete structures.
- EN ISO 8501-1:2007. Preparation of steel substrates before application of paints and related products. Visual assessment of surface cleanliness. Rust grades and preparation grades of uncoated steel substrates and of steel substrates after overall removal of previous coatings.
- ISO 2394:2015. General principles on reliability for structures.
- ISO 13822:2010. Bases for design of structures — Assessment of existing structures.
- ISO 16204:2012. Durability — Service life design of concrete structures.
- ISO 19338:2014. Performance and assessment requirements for design standards on structural concrete
- ISO 22966:2009. Execution of concrete structures.

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- *fib* Model Code for Service Life Design, N° 34, ISBN 978-2- 88394-074-1, February 2006.
- Söylev, T.A and Richardson, M.G. Corrosion inhibitors for steel in concrete: State-of-the-art report. 2008. Construction and Building Materials Volume 22, Issue 4, pp. 609–622. 10.1016/j.conbuildmat.2006.10.013

### 4.4.5 Design assumptions

The design assumptions are determined by the specificity and condition of each structure and by the established requirements and criteria. Assessment, suitable strategies and methods, properties of materials, products and systems, as well as other remedial actions (e.g. to prevent future contamination and deterioration) are the basis for decision (ISO 16311-1).

Usual assumptions in repair with corrosion inhibitors include corrosion performance and related influencing factors (e.g. chloride level at the rebars in the long term), and thus no deterioration and adverse reactions are expected over time.

### 4.4.6 Design procedure

A design of the repair/prevention should be executed after selecting the repair methods, the materials to be used, and the level of repair, addressing all the relevant factors for the efficiency and quality of the repair.

Manufacturers' recommendations and systems trials should be the basis for designing an efficient and durable inhibitor action. Generic requirements include:

- sufficient concentration of inhibitor - corrosion inhibitors act through different mechanisms, but generally their effectiveness is concentration dependent;
- no adverse effects on concrete properties - side-effects of products and systems reactions with each other and with mortar/concrete should be prevented (EN 1509-9, ISO 16311-3);
- stable inhibitor concentration for long term – taking into account that some inhibitors may be leached or evaporated from the concrete or be consumed by the protection reactions.

Manufacturers' guidance should also inform on hazardous products and environmental constraints, which may limit the amount of their use (e.g. toxic nitrites, irritating amines). This data should be included in Material Safety Data Sheets (MSDS) and Environmental Product Declarations (EPD).

### 4.4.7 Supporting documents

Supporting documents to the design include a brief description of the evaluation process during the assessment (ISO 16311-2), including: purpose and scope of investigation; existing construction and documentation; field observations and condition survey; sampling and material testing; evaluation; findings and recommendations.

Corrosion inhibitors products and systems certificates and technical related documents are essential.

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Upon completion of the design, the following documents shall be provided:

- documentation of the repair design, including any test results pertinent to the design;
- documentation of any quality control and assurance requirements for the execution of the repair design;
- instructions for inspection and maintenance to be undertaken during the remaining design service life of the repaired areas.

## 4.5 Execution

### 4.5.1 Preparatory works

Before the execution of repair, a detailed execution plan should be formulated based on the design procedure, and execution environment and timings. The following should be considered (EN1504-9, EN 1504-10, ISO 16311-3):

- general requirements (e.g. the chemical, electrochemical, and physical condition of the substrate, including the presence of any contaminants; the ability of the structure to accept loading, movement, and vibration during repair);
- ambient conditions; the characteristics of the materials contained in the structure and those of the repair products and systems and their compatibility; and the hardened properties of the repair materials and systems);
- structural safety and stability before, during and after repair;
- suitable preparation of concrete substrate and reinforcement;
- application of materials, products and systems;
- quality control of the work;
- maintenance following completion of the repair.

Products and systems manufacturers' recommendations, regarding their application, quality control of work and maintenance should be rigorously followed.

### 4.5.2 System trials

The use of corrosion inhibitors in concrete can delay the initiation of corrosion resulting from chloride attack and carbonation. However, after the initiation of corrosion, their effectiveness is reported to be less significant (Söylev and Richardson, 2008). Information on the long-term performance of inhibitors in real structures is very limited. Since the effectiveness of corrosion inhibitors is not well documented and there is not any standard,

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evidence of the effectiveness of such products and systems should be obtained before specifying its use. Therefore, systems trials may be conceived to compare their performance.

Systems, applied following the manufacturer's recommended procedures and quantities, should be tested, according to recommendations, and compared with the relevant performance requirements in the project specification (ISO 16311-1). Inhibitors performance should be demonstrated for long term use.

Repair products, that are part of a system for repair, shall not normally be tested individually, unless one or more of the repair products are intended to meet particular performance requirements in its own right (ISO 16311-3).

Several methods are suitable to study efficiency and performance of inhibitors, including immersion or exposure tests, followed by visual inspection and open circuit potential monitoring. Other useful tests include non-destructive electrochemical tests (e.g. polarization resistance, impedance) and destructive electrochemical tests (e.g. potentiodynamic, potentiostatic, and galvanostatic), as well as complementary microscopic observation or spectroscopic tests.

Examples of specimens and setups for testing of inhibitors performance are given in Fig. 4-3 and 4-4. Fig. 4-5 and 4-6 show typical information on corrosion obtained by microscopy and spectroscopy.

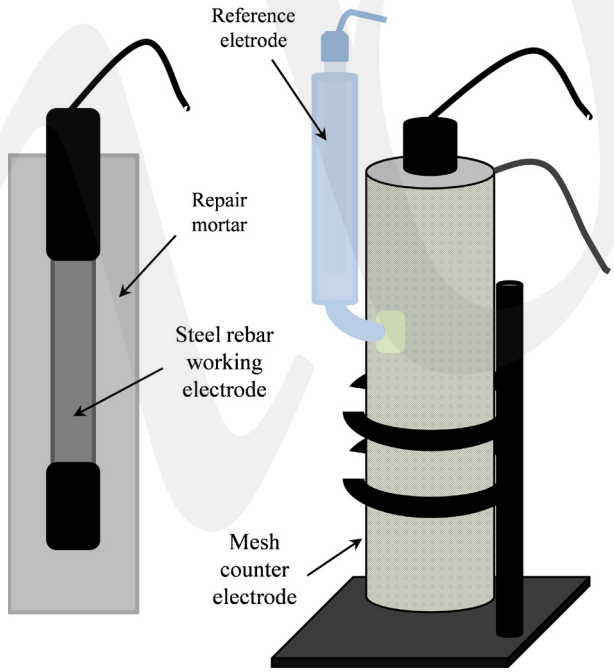



Fig. 4-3 Specimen and setup for electrochemical testing of corrosion inhibitors performance.

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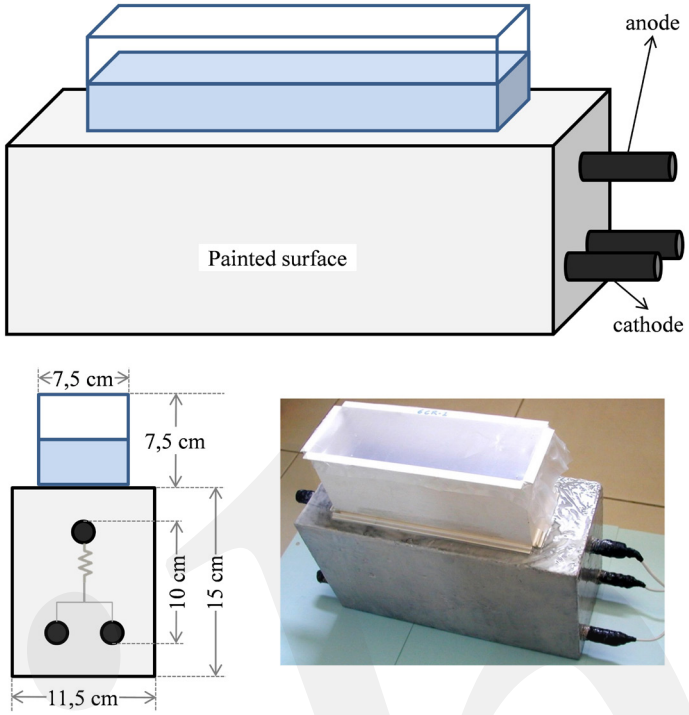



Fig. 4-4 Specimen and setup for testing the effect of chemical admixtures on corrosion of steel reinforcement in concrete exposed to chloride environments, according to ASTM G109.

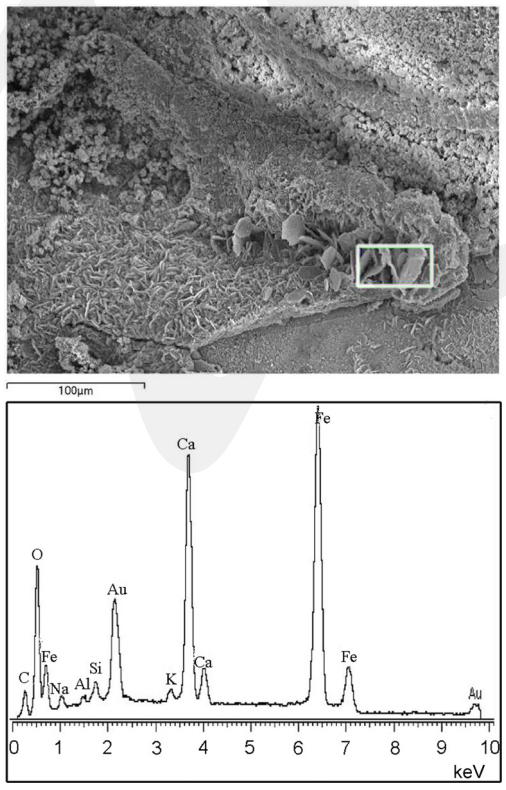



Fig. 4-5 Examples of results of observation and analysis of corrosion products by SEM/EDS.

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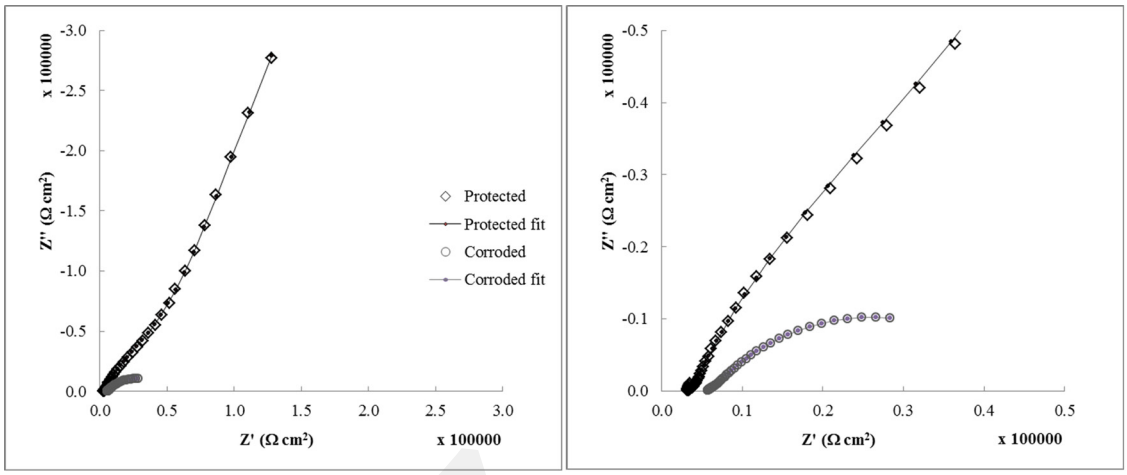



Fig. 4-6 Examples of electrochemical impedance spectroscopy testing results on protected and corroded steel rebar.

### 4.5.3 Execution

The application methodology depends on the particular circumstances of the repair and on the corrosion inhibitor being used. In all cases, the design specifications and the manufacturer's instructions must be strictly followed.

Usually, a typical repair procedure includes the suitable preparation of the substrate, with adequate access for the work, so that the products and systems can be correctly applied (ISO 16311-3). Protection may also be required for these procedures.

For applying corrosion inhibitors in or onto the concrete, weak, damaged, and deteriorated concrete and, where necessary, sound concrete, shall be removed. If required, cleaning shall be carried out after concrete removal. Possible resulting micro-cracked or delaminated concrete should be removed or remediated.

Mechanical percussion and abrasion, grit and sandblasting, and water with low pressure may be used for cleaning, while mechanical percussion and water with high to very high pressure can be used for concrete removal (ISO 16311-4). Structural considerations can limit the extent of the removal.

The reinforcement shall be prepared in accordance with the specifications and required performance. The extent of any cleaning, coating, removal, or replacement shall be specified taking into account the requirements regarding corrosion prevention and specified quality criteria (ISO 16311-3).

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As regards cleaning the requirements are (ISO 16311-4):

- remove rust, scale, mortar, concrete, dust, and other loose and deleterious material which reduce bond or contribute to corrosion;
- clean the whole circumference of the exposed reinforcement (normally extend 50 mm or more beyond the extent of the corrosion along the length of the bar);
- protect the cleaned substrates against further contamination;
- clean reinforcement without causing damage to it or damage to or contamination of the adjacent concrete or environment;
- where exposed reinforcement is contaminated with a corrosive agent, clean the whole of the circumference of the reinforcement by water washing to remove contaminants;
- specify a suitable method and degree of cleaning (ISO 8501-1) when reinforcement is to be coated, taking into account the coating to be applied, bar congestion, contact between bars, proximity to the concrete substrate and other factors which prevent access for cleaning (e.g. Sa2 standard of cleaning for using coatings with active pigment "thorough blast- cleaning").

Bonding requirements for coatings on reinforcement shall be specified. Coatings shall not be allowed to contaminate existing concrete, especially if detrimental to the bond between the existing concrete and the repair products and systems.

### 4.5.4 Finishing

Complementary repair methods and different corrosion inhibitors may be used for improving the efficiency of the repair, depending on the assessment, specified requirements, and repair products and systems used.

## 4.6 Quality control

A quality control system should be employed to ensure that the specified quality requirements are met with the arrangements made for acceptance (ISO 16311-3).

The properties of the substrate, acceptance of products and systems, the conditions for their application, and final properties of the hardened products and systems, shall be subjected to quality control (ISO 16311-4). Manufacturers' recommendations for quality control of corrosion inhibitors should be followed when applicable.

Parameters and frequency of observation or testing shall be in accordance with recommendations and project specifications.

### 4.6.1 Quality controls of materials

Identify all products and systems verifying written certification before use (ISO 16311-4).

Records of initial and performance requirements of products and systems should be provided by the manufacturer (EN 1504-8), according to the specified test methods and respective requirements.

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### 4.6.2 Quality control before intervention

Environmental conditions such as temperature, humidity, precipitation, and dew point should be controlled (ISO 16311-4, EN 1504-10).

### 4.6.3 Quality control during intervention

The specified environmental parameters should be controlled throughout the intervention (ISO 16311-4, EN 1504-10).

Substrate conditions, such as delamination and cleanliness, should be observed and tested (e.g. tapping with a hammer) before application of repair products (ISO 16311-4, EN 1504-10).

The temperature of substrates should be measured throughout the application (ISO 16311-4, EN 1504-10).

Specific characteristics of repair mortar/concrete (e.g. thickness or cover of repair material, and compressive strength) should be measured once after the repair (ISO 16311-4, EN 1504-10).

In the final hardened condition, characteristics such as delamination, thickness of the cover, adhesion, compressive strength, density, shrinkage cracking should be tested (ISO 16311-4).

The dry thickness and covering of reinforcement coating should be measured once to attest the efficiency of the repair (ISO 16311-4).

### 4.6.4 Quality control after intervention

The condition of the repaired areas should be investigated, with special attention to the reinforcement corrosion resistance. It is recommended that this evaluation be conducted by specialists, due to the specificity of the available methods and to the diversity of influencing factors on the results.

## 4.7 Monitoring and maintenance

### 4.7.1 Monitoring

Periodic inspection should be carried out to ensure the ongoing durability of the structure. Testing and monitoring techniques should be agreed, depending on the assessment evaluation and executed repair. A corrosion control plan is recommended. Periodic visual inspection, with non-destructive testing (e.g. electrochemical), and continuous remote sensing (e.g. in Fig. 4-7 and 4-8) are the most common strategies for corrosion monitoring.

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Fig. 4-7 Remote sensors instrumentation for monitoring of a repaired area with concrete removal (electrochemical based sensors).



Fig. 4-8 Remote sensors instrumentation for monitoring of a repaired area without concrete removal.

### 4.7.2 Maintenance

A maintenance plan should be developed (ISO 16311-1) and reported to the owner as guidance for achieving the design service life of the proposed repair work and of the structure (ISO 16311-3).

The essential information should include (ISO 16311-3):

- the design service life of the structure;
- identification of each area where the design service life is expected to be less than the remaining design service life of the concrete structure;
- plan for inspection and testing, including details on recording and support for decisions on future inspections;
- specification for continuous treatment and monitoring;
- a statement of precautions or restrictions.

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This information is case-specific and clearly depends on the adopted repair systems and methods, construction materials characteristics, environmental conditions, and actions.

Especially those parts of the repaired areas that may have a shorter design service life should be regularly inspected, tested, and renewed if necessary (ISO 16311-3).

### 4.7.3 Post-intervention documentation

Unless otherwise agreed, a record of the protection and repair works which have been carried out, documentation of any quality control and assurance requirements completed during the execution, and instructions on inspection and maintenance to be undertaken during the remaining design life should be provided (EN 1504-9:2008, ISO 16311-4).

## 4.8 Case study

| | |
|---|---|
| Project Title: | Structural Rehabilitation of a Chloride Contaminated Concrete Silo - ICRI 2009 Project Winner - Award of Excellence: Longevity Category. |
| Location: | Açores, Portugal |
| Client: | Sociedade Açoreana de Sabões, S.A. |
| Designer: | Eng. José Paulo Costa, STAP, S.A. |
| Contractor: | STAP, S.A. |

### 4.8.1 Description of the structure

The silo, a reinforced concrete structure completed by the slipform method in 1980, consists of a 62 m high equipment tower and a battery of 12 cylindrical bins, 6.5 m in diameter and 50 m high. The exposed external surface amounts to 6'000 $m^2$ and no protective coating or painting was originally considered. The structure stands circa 100 m away from the sea, in a very windy and moist site and almost permanently under sea spray condition. The silo was constructed using very porous volcanic aggregates and beach sand.

### 4.8.2 Inspection highlights

Signs of significant deterioration had been noticed, mainly widespread concrete spalling owing to reinforcement corrosion (Fig. 4-9 and 4-10).

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Fig. 4-9 Silo before and during the intervention in 1998.



Fig. 4-10 Area showing severe deterioration, marked for concrete removal.

Faced with the low durability of successive piecemeal repairs, which did not stop the deterioration process, the Client decided to order a thorough and accurate analysis of the anomalies of the structure. A complete assessment program was established, involving tests both on site and in the laboratory, to characterise the structure and its materials and diagnose the deterioration process.

A detailed inspection involving non-destructive testing was conducted. The inner and the outer surfaces of the concrete structure were assessed separately, since the exposure conditions were quite different. The following tests were carried out:

- Detection of reinforcement and measurement of cover thickness
- Determination of depth of carbonation
- Evaluation of risk of active corrosion
- Determination of chloride content at several depths
- Ultrasonic tests
- Rebound tests
- Simple compression tests on concrete cores
- Concrete petrographic analysis

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- Absorption and capillarity tests on concrete specimens
- Permeability tests on concrete specimens.

On the external surfaces, the thickness of cover was found to vary significantly. The mean thickness values ranged from 16 to 35 mm. The depth of carbonation ranged from 10 to 65 mm, showing a marked heterogeneity of the concrete. In 75% of the cases in which rebars were exposed, the carbonation depth was found to equal or exceed the thickness of the cover. Chloride content was high, 0.6% to 1% of cement mass at reinforcement depth (Fig. 4-11), and showed that the high chloride content obtained on the external surfaces was mainly due to the diffusion of the air-borne chlorides. The results obtained through measurement of corrosion rate, electric potentials and concrete resistance showed that the risk of active corrosion was widespread.

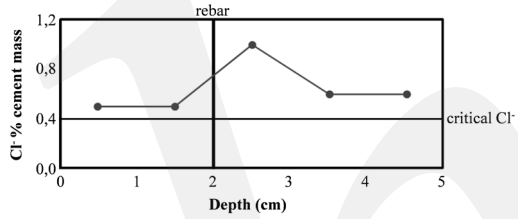



Fig. 4-11 Chloride content near the external surface of the silo.

On the internal concrete surfaces, the values of cover were in general close to 25 mm. However, the values were significantly lower at some areas of the zones that were surveyed. In general, carbonation had progressed until very close to the reinforcement and extended beyond, at some zones. The measured chloride content at the inside surface of the silos was 0.09% of cement mass at reinforcing depth (Fig. 4-12), lower than the outside value and lower than the critical limit value of 0.4% of the cement mass. The results obtained through measurement of corrosion rate, electric potentials and concrete resistance did not point to the existence of significant reinforcement corrosion in the inner surfaces, which, additionally, did not show the usually related anomalies.

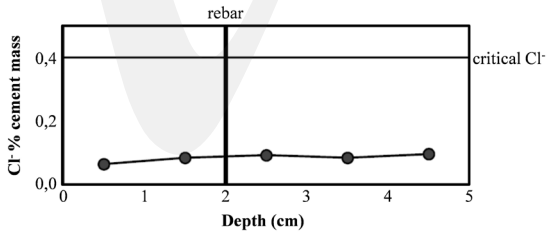



Fig. 4-12 Chloride content near the internal surface of the silo.

## 4.8.3 Description of the intervention project

Some repair options were evaluated. The more drastic solution, taken as a yardstick for the cost-benefit analysis of the other solutions, consisted of the construction of a new silo. Due to financial limitations and operational constraints, the selected rehabilitation strategy involved the repair of anodic zones (30% of total area), application of migrating

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corrosion inhibitors and total external protection through a coating against carbonation and chloride penetration. This intervention strategy had the shorter construction time, when compared with the other solutions, and was expected to ensure a 25-year lifetime.

In order to minimize negative effects for the non-repaired zones and mainly for boundary zones, it was decided to use migrating corrosion inhibitors, applied throughout the exposed surface of concrete, aiming at a general repassivation of steel reinforcement.

A basic requirement for a durable concrete rehabilitation is general surface coating with a paint previously tested and artificially aged, to confirm its appropriate resistance to chloride ion penetration and to the diffusion of carbon dioxide, as well as its imperviousness to liquid water and its permeability to water vapour. Such operation was deemed the most important factor to ensure the required 25-year durability of the non-repaired areas.

### 4.8.4 Description of the intervention works

The rehabilitation works were executed from December 1998 to February 2000. Total costs were 920.048 euros.

The rehabilitation intervention followed a sequence of required operations, namely:

1. Removal of deteriorated concrete by means of light pneumatic hammers (Fig. 4-13) in order to expose the corroded reinforcements and create a clean and sound surface to receive the new concrete material.
2. Sandblasting of the exposed reinforcement in order to remove all corrosion products (Fig. 4-14).
3. Application of dry-mix sprayed concrete, using double- chamber machines, assuring a concrete cover of 4cm minimum (Fig. 4-15).
4. Spraying of the migrating corrosion inhibitor.
5. Application, on all external concrete surfaces, of an acrylic paint with dry total thickness of 150 micron (Fig. 4-16), capable of resisting chloride penetration and carbon dioxide diffusion.



Fig. 4-13 Removal of decayed concrete.

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Fig. 4-14 Concrete surface after treatment, ready to receive the sprayed concrete.

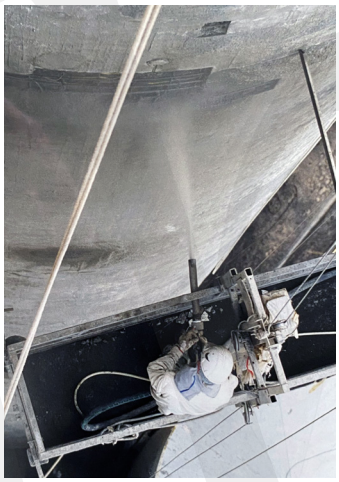


Fig. 4-15 Sprayed concrete.

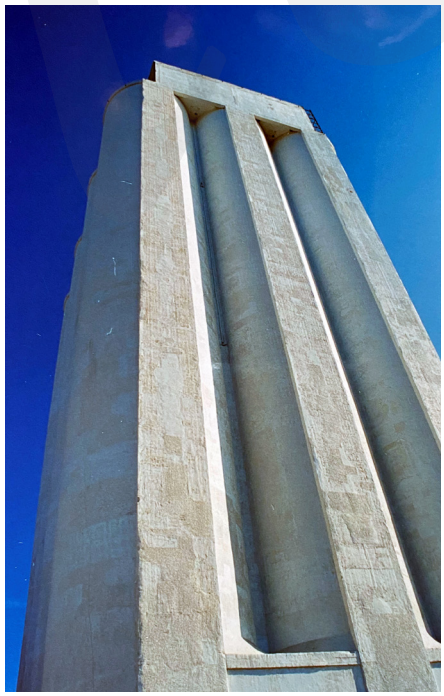


Fig. 4-16 Silo after the intervention in Feb. 2000.

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The composition of the sprayed concrete was selected taking into account the characterisation of the locally available materials. Fibres were used in order to minimize shrinkage cracking effects. Concrete was applied by operators certified according to ACI standards.

A compressive concrete strength of 45MPa was obtained as well as a low permeability and low porosity concrete. New concrete bond to the original concrete exceeded the tensile strength of the later (pull-off tests).

A quality plan was implemented during the silo repair work, including control of supplies and repair processes, inspection and testing in the course of the works and on the final product as well as accurate recording of all evidence on the controls carried out. To control the repair process, several inspections and tests were carried out (control of aggregate grain-size, control of cover thickness, determination of compressive strength, etc.)

### 4.8.5 Description of maintenance operations

A corrosion monitoring system was installed, involving a number of sensors embedded in the silo walls, both in new and old concrete (Fig. 4-17). The general objective was to assess, with sufficient accuracy, the real durability of the rehabilitation job and, in particular, to foresee the need for future renovation of the protection coating.

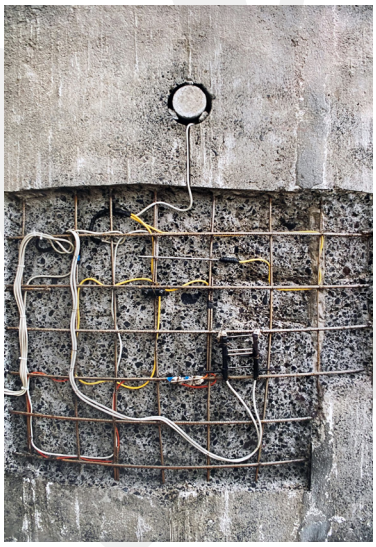


Fig. 4-17 Example of a monitoring area.

On the repaired surfaces, four areas of 1 $m^2$ each were chosen. Close to each monitored area, a sensor was placed in the original non repaired concrete. The monitoring system that was installed consists of embedded sensors capable of measuring:

- Macrocell current
- Electrochemical potential of reinforcing steel
- Concrete electrical resistance
- Temperature.

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Two different macrocell current sensors were built and installed in the original (non-repaired) concrete and in the sprayed concrete. Sensors for electrochemical resistance and temperature measurements were also embedded in the new concrete in each monitoring area. Data is acquired by means of a central data logger (Fig. 4-18).

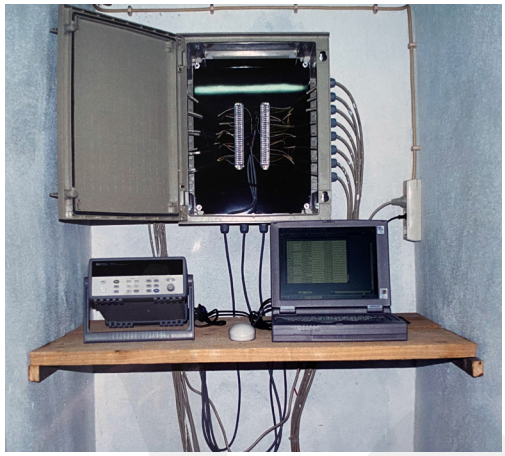


Fig. 4-18 Monitoring system.

The values of the potential obtained in the repaired areas were initially very negative. However, after two years, they were shifting to less negative values, a trend compatible with the repassivation of the rebars. The macrocell current values obtained during the following years until 2009 were very low, indicating low corrosion activity. The agreement between the values of the macrocell current with the values of the potential measurements, indicates that the reinforcing steel has been maintained in the passive state and, thus, free of corrosion. This demonstrates the effectiveness of the repair method for the monitoring period (Fig. 4-19).



Fig. 4-19 Silo in Oct. 2009.

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In the areas where the concrete was not replaced, the initially values of macrocell current were also high, indicating some corrosion activity. The values decreased with time to levels of the same order of magnitude of those measured in the repaired areas. This indicates a reduction of the corrosion activity and its absence since the rehabilitation works, in spite of concrete still being contaminated with chlorides.

The concrete electrical resistance values show generally a pattern compatible with the seasonal variation of temperature and rainfall. No trend towards lower values was observed over time.

According to data obtained up to the present, the rehabilitated silo is performing fairly well. This case study shows the importance of close cooperation between owner, designer, and contractor, and following the proper methodology when dealing with structural rehabilitation projects.

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# 5. Cathodic protection

## 5.1 Foreword

This section deals with Sacrificial Anode Cathodic Protection (SACP) and Impressed Current Cathodic Protection (ICCP) systems for concrete repair in atmospherically exposed structures (XCi, XSi exposure environment according to *fib* Model Code), both for cathodic protection and for cathodic prevention.

Cathodic protection (CP) of buried or immersed concrete is not considered herein as it involves different principles of spreading the protective current uniformly over the concrete surface.

Cathodic protection is the method for halting or limiting the corrosion rate of steel through the application of a permanent negative direct current. To be defined as cathodic protection the system should provide a high likelihood of meeting one of the depolarisation criteria given for cathodic protection in ISO 12696.

Cathodic prevention is the method of preventing corrosion activation of steel in a passive state at high chloride levels through the application of a permanent negative direct current. Typically applied to prevent the corrosion of steel in new structures in very severe exposures, particularly where low cover is to be applied.

Corrosion control consists of slowing the corrosion rate of active reinforcement through the application of a permanent negative direct current. This is a term commonly applied to sacrificial anode system where it is known that the ISO 12696 criteria for cathodic protection will not be achieved but significant reduction in the corrosion rate is expected.

*Comments:*

*Cathodic protection of reinforcement has been used in chloride contaminated or carbonated concrete for more than 35 years. Experience with the methods is very good – cathodic protection can lead to durable, long lasting and economical solutions to the problem of the reinforcement corrosion. Cathodic protection is especially suitable when chlorides are the cause of corrosion.*

The aim of a CP system is that a current is passed to the reinforcement from an external anode so that electrons flow to all the reinforcement inducing cathodic polarization and therefore very low corrosion rate.

Current distribution is achieved by two types of system:

- Type 1: Anodes are embedded inside the concrete and are used at relatively close spacing. The concrete is used to spread the current over the reinforcement.
- Type 2: Anodes are applied on the concrete surface. Current is spread through the cover to reach the steel reinforcement.

This chapter was written by Andrade, Bouteiller, Hallopeau (main author), Lesieutre, Meillier, Papworth. https://doi.org/10.35789/fib.BULL.0102.Ch05

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*Comments:*

*Cathodic protection of steel in concrete is based on the fact that the reinforcement's potential is shifted towards more negative regions as shown in Fig. 5-1 where the initiation of corrosion, or if corrosion has already started, the propagation of corrosion is so far suppressed that a corrosion failure is unlikely during the lifetime of the structure. Fig. 5-1 shows schematically the behaviour of steel at different potentials and chloride contents. At relatively noble (positive) potentials and high chloride contents as represented by domain A, the conditions are met for both initiation and stable propagation of pitting corrosion. In region B initiation is not possible, but still pitting can propagate ("imperfect passivity"); in region C both initiation and propagation are not possible ("perfect passivity"). At potentials lower than ca. -900 mV vs SCE (Saturated Calomel Electrode) problems might arise due to risk of hydrogen embrittlement of high strength steel or risk of bond loss at the steel concrete interface.*

*Point 1 in Fig. 5-1 represents the initial situation of steel in chloride-free concrete with a resting potential between -200 and 0 mV vs SCE. As the chloride content increases over time the situation is shifted to point 4 and pitting corrosion might initiate. By impressing a current (cathodic protection) the potential can be shifted to the negative domains to point 5 where the steel can repassivate. It might be insufficient to lower the potential only to point 6, since propagation could still take place in domain B.*

*However, such a potential shift is likely to slow the corrosion rate significantly. In case of "cathodic prevention" the steel potential is initially lowered just a little (point 2) and chloride ingress over time then leads to point 3, i.e. no corrosion initiation at high chloride levels.*

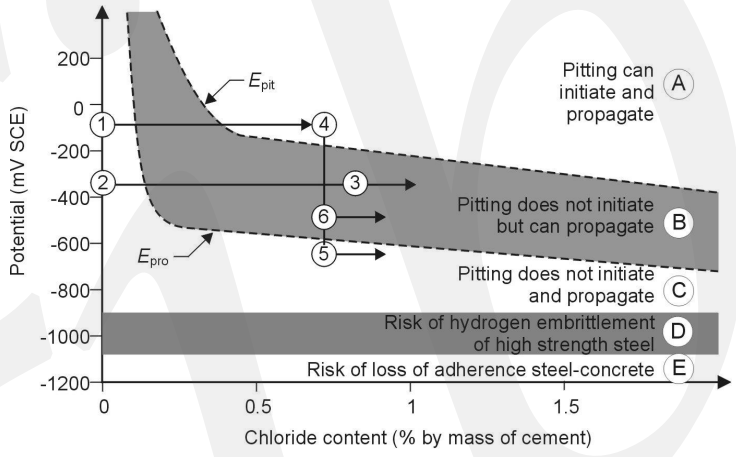



Fig. 5-1 Schematic presentation of the behaviour of the reinforcement for different potentials and chloride contents. (P. Pedeferri, 1996).

Fig. 5-2a is a schematic illustration of the set-up used when applying ICCP. The reinforcement that needs to be protected acts as a cathode and is connected to the negative pole of a rectifier (DC current) and the anode to the positive pole. The current delivery is therefore controlled.

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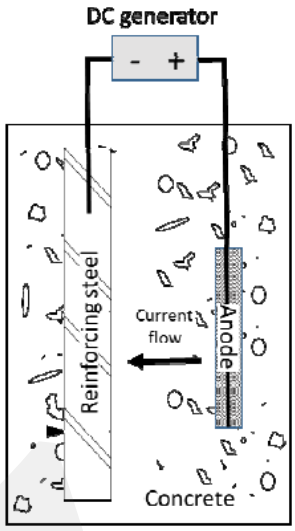



Fig. 5-2a Schematic illustration of an ICCP system.

For SACP system, the current between the reinforcement and the galvanic anode is not constant nor controlled as it depends on temperature and concrete resistance (relative humidity). The anode is linked directly to the reinforcement or through an external junction box as illustrated in Fig. 5-2b.

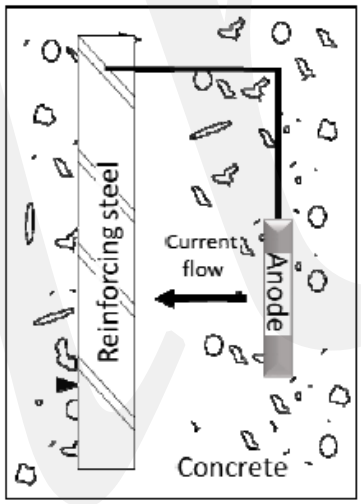



Fig. 5-2b Schematic illustration of an SACP system.

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## 5.2 Basics

### 5.2.1 When to adopt this method

When repairing a reinforced concrete structure suffering from corrosion, the owner has various options available. SACP and ICCP systems have different advantages and disadvantages. However, relative to other repair methods based on patches they are unique because they can halt corrosion without removing the cause of corrosion initiation, i.e. carbonated or chloride contaminated concrete as long as the contaminated concrete remains adhesive to the reinforcement.

### 5.2.2 Materials

The material and systems used depends on the type of cathodic protection system. The primary difference depends on whether the system is impressed current or galvanic anode.

General requirements for an anode system for protection of steel reinforcement are:

- Good adhesion with concrete;
- Good electrical ability by spreading the current homogenously and continuously during the service life;
- Correct suitability depending on the anodes that should:
  - Show an acceptable current discharge for the proposed repair;
  - Offer an acceptable aesthetic condition;
  - Resist vandalism;
- Consistent mechanical properties suitable for installation and maintenance;
- Capable of durability throughout the service life.

#### 5.2.2.1 Impressed current system

Type 1 anodes are generally made from Mixed Metal Oxide (MMO) coated titanium mesh. These anodes provide an acceptable operating current density for the current required and design life of CP systems at a reasonable cost.

Type 2 anodes are typically surface applied coatings based on either a binder filled with graphite or a metallic spray.

*5.2.2.1.1 Type 1 – Mixed Metal Oxide coated Titanium anodes (MMO/Ti).*

Titanium sheet is cut and stretched to make an expanded metal sheet. The expansion can be controlled to give almost any degree of mesh openness. High surface areas of titanium per square meter can be obtained with a small expansion with reducing titanium area per square metre as the expansion increases.

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*Comments:*

*The titanium is typically supplied to meet the requirements of ASTM B265 Grade 1. Because ductility and strength are key requirements the titanium should pass requirements for tensile strength to ASTM E8 and bend tests to ASTM E290.*

*The titanium surface is made electrochemical active by covering the surface with a microscopically thin layer of various noble metals. These mixed metal oxides (MMO) of iridium, ruthenium, cobalt, etc are stabilised with oxides of titanium, tantalum niobium and tin. The coating is laid down in multiple layers (10-20).*

*Because the coating details is proprietary, the anodes are specified based on performance tests. The performance of each batch of anodes is verified by accelerated tests in accordance with NACE TM0108-2016 and TM294-2016 by external and independent laboratories. This type of anode has good mechanical properties and may be fixed to all surfaces; it may also be formed and cut to distribute the necessary current. Used on existing structures the metal must be covered with an electrolyte, e.g. spray concrete (when used as mesh anode). Activated titanium is used as meshes anode, ribbon or wire anode.*

#### *5.2.2.1.1.1 Ribbon mesh anodes*

Ribbon mesh is supplied in roles with the ribbon itself of various widths. The greater the width, the higher the rated current output per linear meter. Ribbon can be installed at 200-500 mm centre or less with the following configurations:

*Ribbon installed in slots*

A common approach is to use ribbon anodes in slots which are then grouted, see Fig. 5-3. The system is designed to have uniform current along the length of the ribbon. The current is then distributed out from the ribbon to the reinforcement on either side and under the ribbon. The grout is designed to give complete filling through the ribbon and in the slot.

Acidification is a higher risk where there are voids that get filled with water. It is likely to be the combined effect of voids and current dumping in low resistance areas that causes the problem. A cement mortar or grout is required aiming at filling the holes in the mesh. The method of application also needs to ensure that the grout will fully encompass the anode without leaving any voids.

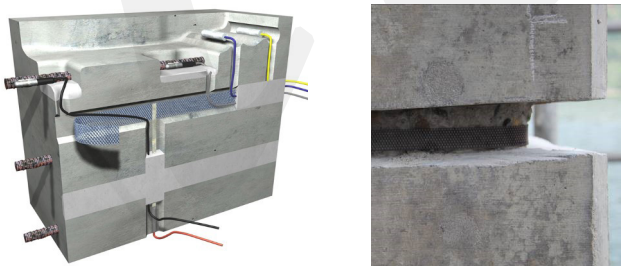


Fig. 5-3 Ribbon mesh grouted in slot.

*Ribbons pinned to the concrete surface*

Instead of embedding the ribbons on slots they can be pinned to the concrete surface and cement mortar or grout applied over them, see Fig. 5-4. This helps to reduce the issue of not having adequate cover to cut slots but is generally only used on floor slabs

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where the fineness and workability of a self-levelling topping leads to good penetration of the grout through and under the ribbon. Adequate cover over the ribbon will help preventing local current dumping in wet areas.



Fig. 5-4 Ribbon mesh pinned to the concrete.

*Ribbons embedded in the concrete repair*

Where large parts of an element have spalled, and extensive breakout is required to clean up the concrete surface prior to application of cathodic protection, it may be possible to incorporate the ribbon anode in the pour, see Fig. 5-5. The method of fixing the anodes is similar to the approach used for cathodic prevention in new structures. Particular attention has to be given to avoid shorting between the ribbon and rebar which will be in close proximity.



Fig. 5-5 Ribbon mesh fixed on plastic spacer

*5.2.2.1.1.2 Anode meshes*

Anode meshes typically have a much more open structure than ribbon. Because this means a relatively low current output per square metre of mesh. The mesh is typically applied over the whole concrete surface, see Fig. 5-6. This leads to a uniform current density over the concrete surface, a key design principle for cathodic protection systems. Because of the large concrete volume per unit area of mesh there is a large buffering capacity and acidification issues are a low risk. These systems have been successfully applied in splash and tidal zones.

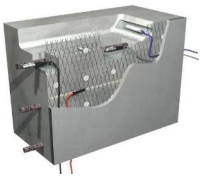




Fig. 5-6 Mesh applied on concrete surface before dry sprayed concrete.

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When concrete, shotcrete or mortar is used as a topcoat, the general requirements are:

- Good bonding to the concrete beneath;
- Electrochemical properties that will optimise the spread of current;
- Physical properties as close as possible to the concrete beneath to minimise the failure potential;
- A minimum thickness of 10 mm, consistent with protecting the anode mesh, in order to reduce weight impact on the structure.

*5.2.2.1.1.3 Discrete anodes*

- Discrete anodes are typically defined by their diameter and length, Ti/MMO cylinder or ribbon anodes spot welded on a titanium rod or plate. Fig. 5-7 presents a distribution of discrete anode location on a beam and the electrical wiring.
- Different configurations of anode mesh used to form the discrete anode lead to different current ratings per linear metre of anode based on the NACE, ISO 12696 or equivalent standard limit, to prevent acidification. This limit is then often used to give a current rating for the anode.



Fig. 5-7 Ti/MMO discrete anodes in concrete beam.

*5.2.2.1.2 Type 2 Coatings*

*5.2.2.1.2.1 Electrical Conductive Coating (ECC)*

The conductive coating is based on the principle of current delivery by a primary anode wire of strip laid at relatively wide centre, typically 1-2 m, with a conductive coating that is in electrical contact with the primary anode and then spreads the current over the concrete surface, see Fig. 5-8. Conductive coatings are carbon particle filled polymers (organic or inorganic). Film thickness is typically about 250 µm.



Fig. 5-8 Application of conductive coating.

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#### *5.2.2.1.2.2 Metalised coating*

A flame sprayed or sprayed titanium or titanium alloy coating is applied to the concrete surface.

Typically, the thickness of the sprayed layer is around 100µm or more.

### 5.2.2.2 Galvanic anode systems

Anodes protect the steel reinforcement as they have a lower natural potential than the steel and hence a potential difference is set up between the anode and steel such that the anode corrodes rather than the steel. Typically, Zinc is used as the anode as it has a small manageable corrosion expansion and an ideal potential that is sufficient to polarize the steel but not so high that it can cause hydrogen evolution. However, zinc anodes passivate at the normal pH of concrete and it is necessary to maintain the zinc activity by using lithium hydroxide based mortars to keep the pH above 13.5 and chloride and sulphate low pH based pastes, or any other way of activating zinc, such as using complexing substances.

#### *5.2.2.2.1 Type 1 - Lithium hydroxide activation*

One of the earliest sacrificial anodes was zinc with a lithium hydroxide activator. By maintaining the zinc at a high pH it was kept active. The reaction products at the zinc surface are lithium hydroxide which is expansive and hence the anode is surrounded by a porous mortar that the reaction products fill. Over time the anodes output drops but then the current demand also drops. The early anodes were only recommended for cathodic prevention or corrosion control situations where only a low current output was required. These low output anodes are still commonly used to protect against incipient anode failure.

A more recent development consists of using larger anode, in order to give higher zinc surface area and far better reinforcement polarization, see Fig. 5-9.



Fig. 5-9 Large embedded zinc anode.

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#### *5.2.2.2.2 Type 2 - Others*

##### *5.2.2.2.2.1 Hydrogel activation*

In the late 90's, a zinc sheet cathodic protection system that could be bonded to the concrete surface and remain active due to the hydrogel was introduced, see Fig. 5-10. The galvanic protection worked very well due to the high surface area and maintenance of zinc activation. The systems activator/adhesive is still susceptible to moisture and the installation design must take care to prevent moisture ingress.



Fig. 5-10 Zinc layer anode on a concrete brace.

##### *5.2.2.2.2.2 Thermal/Arc sprayed anode*

Another system developed in the mid 90's is thermally sprayed zinc. In these systems, zinc is sprayed onto the concrete surface to act as a sacrificial anode, see Fig. 5-11. Typically, the layer is around 200µm thick or more.





Fig. 5-11 Sprayed zinc on concrete.

### 5.2.2.3 Equipment

#### *5.2.2.3.1 Wiring and connections*

All wiring and connections from rectifiers to anodes shall be made of materials that can withstand the physical and chemical conditions to which they will be exposed in the course of the installation's lifetime.

Connections between primary anodes and copper cables are sensitive to accelerated corrosion where (even small) pathways for water penetration exist. Isolation of anode-copper connections needs to be executed with great care to avoid leakage pathways.

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*Comments:*

*Wiring shall be insulated and preferably without junction. The insulation shall be waterproof, airtight and alkali-resistant. If wiring must be interconnected, the same demands in respect of water resistance apply.*

*Colour-coded wire insulation shall be used, clearly delineating the different wires. Alternatively, numbered wires may be used.*

*The cross-section of the electrical wiring shall be such that no appreciable loss of voltage occurs from the rectifier to the point at which the reinforcement or the anode is contacted. The anodic current density in the zone should be designed to be uniform to within 10% of nominal current density in all areas of the zone (ISO 12696 or equivalent standard). If not otherwise stated, the wiring shall have a cross-section of 2.5 mm² as a minimum.*

#### *5.2.2.3.2 Voltage source*

The range AC input of the transformer-rectifier is generally 60 to 230V AC (50/60 Hz).

Power sources can also be based on solar or wind generators, usually with a battery backup for periods of low yield.

*Comments:*

*The voltage source shall have a power output adapted to the cathodic system, limited to 48V (Electrical human risk).*

*There are two main types of transformer-rectifier for concrete cathodic protection, phased angle (or thyristor controlled) and switch mode.*

#### *5.2.2.3.3 Monitoring units*

It shall be possible to monitor the effect of the cathodic protection, in the structure or in each defined zone.

Each zone should be monitored:

- Output DC Voltage;
- Output DC Current;
- Instant-off potential and depolarization of steel reinforcement versus minimum of two permanent reference electrodes integrated into the concrete at representative locations nearest to the reinforcement.

*Comments:*

*ISO 12696 plans to add depolarization probes to prevent potential shift of reference electrodes*

#### *5.2.2.3.4 Control cabinets*

The cabinet shall have space for the following primary elements:

- Voltage source;
- Monitoring unit;
- User's manual and drawings.

The cabinet shall also have a panel where it is easy to check each separate electrode and reinforcing steel.

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*Comments:*

*The cabinet shall be lockable. If the cabinet is placed outdoors, a steel cabinet will require corrosion protection. Alternatively, a cabinet of polycarbonate or stainless steel may be used, see Fig. 5-12. The control cabinet shall be located near the area which has received cathodic protection. Cabinet placement shall be secured and easily accessible to the user. A user's manual shall be placed in the control cabinet along with drawings of the structure, which show the location of reference electrodes.*



Fig. 5-12 control cabinet and junction boxes under wharf slab.

#### *5.2.2.3.5 Junction boxes*

Junction boxes shall be rated to render appropriate protection against the environment, taking into account the type of connections made within the box and the worst case external environmental and mechanical exposure to which the box is to be subjected to.

#### *5.2.2.3.6 Reinforcement connections*

The cable connections to the reinforcement and steel shall be made by methods providing a long-term cable/reinforcement or steel resistance of less than 0,01 Ω. Arc welding of a bar connected to a single core copper cable XLPE/XLPE sheathed is the best practice to provide a strong and reliable electrical connection.

*Comments:*

*For securing this, the potential difference between the cable and reinforcement/steel should be checked whether it is less than 1 mV or not.*

## 5.3 Stakesholders' roles and qualifications

### 5.3.1 Owner

The owner must determine the following, prior to the actual repair:

- Remaining service life of the structure;
- Performance requirements of the structure.

The owner must share the design plans and construction records of the original structure with the designer and the contractor.

### 5.3.2 Designer

The designer of the cathodic protection system should be involved through the project including the commissioning stage.

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### 5.3.3 Contractor

The contractor must have knowledge and competence in cathodic protection techniques and should have qualified and certified personnel for installation, testing of the installation, energizing, commissioning and long-term operational control.

*Comments:*

*The following are examples of the certification of the cathodic protection specialists: ISO 15257, NACE standard or equivalent.*

## 5.4 Design

Cathodic protection systems must be designed taking into account the preparatory works and the repairs (structural or not) to be undertaken, the whole in a coherent repair protocol and in accordance with the objectives of life expectancy of the project owner.

The cathodic prevention system must be designed taking into account the future environment of the structure, the methods of realisation and the qualities of concrete readily available, the whole integrated in a coherent construction protocol and in conformity with the life expectancy objectives of the project owner.

### 5.4.1 Assessment of existing conditions

The repair strategy will have to be robust and justified by evidence from a diagnosis report, written by qualified personnel.

The diagnosis report should create a vision of the situation of the structure in its environment:

- Information of target structure (type of structure, shapes/dimensions of members, kinds of reinforcement, location, construction year/season, quality of concrete [water to cement ratio, type of cement, the property of the other materials...], history of maintenance already carried out, …);
- Service life;
- The influence of third party;
- Environmental condition (exposure to water and oxygen, humidity, ambient temperature, wind, sunlight, …);
- Site conditions (the existence of power supply, time possible to conduct the cathodic protection work...);
- Visual inspection (bond failure, spalling, delamination, the existence and area of repair parts and the used repair materials...);
- State of concrete (chloride content profiles, carbonation depth, the possibility of ASR occurrence, concrete resistivity, mechanical characteristics of concrete ...);
- State of rebar (cover thickness, arrangement, amount of rebar, corrosion grade, corrosion potential mapping / corrosion rate, continuity/connectedness of rebar, linear polarization to determine corrosion rate...);
- Stray current corrosion risk.

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### 5.4.2 Service life

When cathodic protection is applied for repair, the remaining service life of the structure needs to be assessed. In some cases, owners may be satisfied with a relatively short residual life (e.g. 15-20 years) while for more monumental structures, or structures that have only achieved a short part of the expected design service life, a longer life will be sought.

#### 5.4.2.1 Galvanic anodes

Galvanic anodes are consumed at a high rate compared to impressed current anodes. This limits their life and design life of 15-20 years is often quoted. The consumption rate of zinc can be assessed with reasonable accuracy and hence the number of anodes can be designed to extend the design life beyond 20 years, for example by increasing anode mass and by installing them close together.

#### 5.4.2.2 Impressed current anodes

Impressed current anodes are designed for a long life (typically 50 years) under normal operating conditions.

By definition, impressed current systems make it possible to adjust current and voltage setpoints and thus respond to larger corrosion situations than that of galvanic anode.

*Comments:*

*Conductive coating anodes will not achieve 50 years of working life; if such a long working life is required, anode replacement at some point in time, typically after 15 years, must be taken into account.*

### 5.4.3 Codes and standards and other relevant references

- ISO 12696:2016, Cathodic Protection of Steel in concrete
- ISO 15257:2017, Cathodic protection — Competence levels of cathodic protection persons - Basis for certification scheme
- NACE SP0290-2007 formerly RP0290, Impressed Current Cathodic Protection of Reinforcing Steel in Atmospherically Exposed Concrete Structures
- NACE SP0216-2016, Sacrificial Cathodic Protection of reinforcing steel in atmospherically exposed concrete structures
- NF EN 206/CN:2014, Concrete. Specification, performance, production and conformity
- JSCE Concrete library 157:2020, Guideline for electrochemical corrosion control method

### 5.4.4 Design assumptions

#### 5.4.4.1 Anode Spacing (ribbon or discrete anode)

The maximum anode spacing is determined by the current throw. Typically, anodes are installed at 200-450 mm centres. The close spacing may be required particularly to give uniform current density over prestressing tendons while the higher spacing is most likely to be feasible in damp concrete. Spacings over 450 mm lead to a risk of uneven current distribution.

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*Comments:*

*In wet concrete the concrete resistivity is likely to be less than 30 ohm.m while drier concrete may still require cathodic protection with a resistivity more than 300 ohm.m.*

#### 5.4.4.2 Anode Current Density Limits

Generally, the anode current density is limited by design to a long-term maximum of 110 mA/m² but current densities of up to 220 mA/m² may be permitted.

Conductive coatings generally operate at a current density of up to 20 mA/m².

*Comments:*

*MMO/Ti activated titanium anodes may be operated at current densities of up to 400 mA/m² of anode surface area for short periods; the limiting factor is acid attack of the surrounding concrete.*

*For short periods (i.e. weeks) current densities of up to 30 mA/m² may be sustained on conductive coating without significant damage to the coating or its interface with the concrete.*

#### 5.4.4.3 Potential Associated Issues

##### *5.4.4.3.1 Hydrogen generation*

The generation of hydrogen at the cathode which can potentially cause hydrogen embrittlement to the reinforcement, and possibly interfere with the bond between the reinforcement and the concrete. These effects may reduce the load-bearing and spreading capacity of elements. Although these concerns are theoretically sound, there is no definitive confirmation of the occurrence in practice.

For prestressed structures, a most negative potential of -900 mV versus *Ag/AgCl*/0.5 M KCl applies in order to prevent hydrogen evolution at pre-tensioning steel or post-tensioning ducts and anchors.

##### *5.4.4.3.2 Alkali silica reaction*

The design of cathodic protection applied to ASR concrete structures can be conducted according to "JSMS:2007 Guideline for the Application of Electrochemical Corrosion Control Method considering ASR".

The increase in the risk of alkali-silica reaction from hydroxyl-ion generation around the steel and paste softening in the same region from alkalization electro-migration were found to be insignificant at the current densities normally associated with cathodic protection.

*Comments:*

*The accumulation of Na+ or K+ around the reinforcement due to the electrochemical migration possibly accelerates ASR which will cause the concrete cracking along the reinforcement.*

### 5.4.5 Design procedure

The structure is divided into zones of homogeneous anodic behaviour (which can be independently controlled), both in the structure to be protected and in its exposure to its immediate environment:

- Structure type (reinforced concrete or prestressed concrete);
- Corrosion state of rebar;

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- Exposure classifications (buried, immersed, tidal, splash, atmospheric, sheltered, exposed to brine…);
- Concrete chloride content;
- Concrete resistivity (original concrete / new concrete);
- Concrete moisture content;
- Reinforcement /steel surface area;
- Application range;
- Distribution and predictable current demand;
- Lifetime requirements;
- Service environment;
- Added weight (in the case of surface anodic systems requiring additional concrete cover);
- Installation sequence and constraints;
- Maintenance.

Attention must be paid to the compatibility of the cathodic protection system with the repair solution, considering:

- choice of type of cement, no epoxy resin mortar;
- no passivating agent on steels;
- choice of the type of concrete protection coating.

Protection current densities need to be provided in the following ranges:

- 0.2 to 2 mA/m² of steel for cathodic prevention (passive steel);
- 2 to 20 mA/m² steel for cathodic protection (active steel).

With multiple layers of reinforcement, the designer must take care about shielding effect (the nearest rebar layer will consume 70% of protection current, and impact the protection current spreading process).

*Comments:*

*The design principle of a cathodic protection follows the diagram in Fig. 5-13.*

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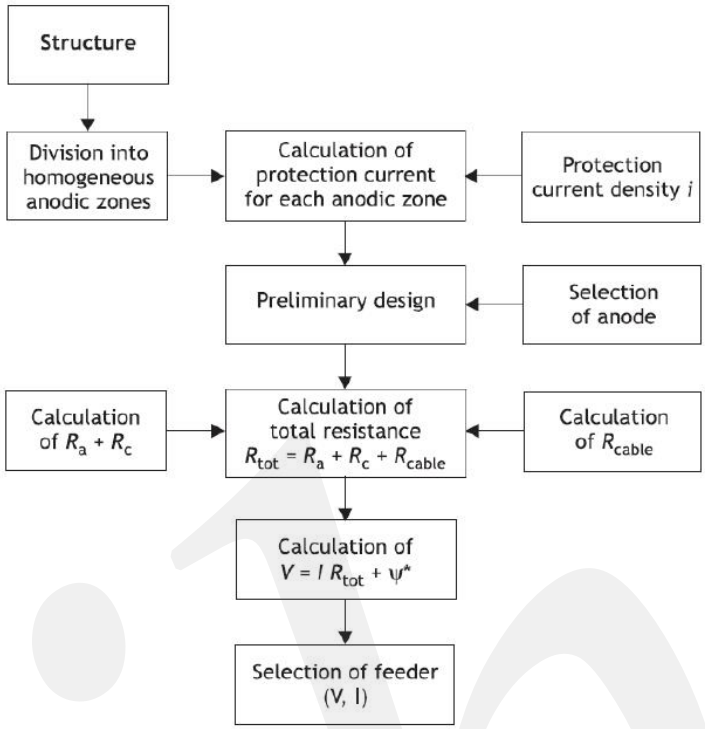



Fig. 5-13 Diagram from L. Lazzari and P. Pedeferri (Cathodic Protection, Polipress, 2006).

*Comments:*

*Particular attention must be paid to the variability of the concrete resistivity and the exposure to oxygen of the parts of the structure concerned by the cathodic protection.*

*These are design values, so the capacity should be available. However, in practice after some time, the current density needed for protection will be lower by a factor 2 to 4.*

*Wet/dry external conditions will affect resistivity; a fixed voltage will provide more current in wet times (when needed). Wet/dry variation does not have much effect on current distribution.*

### 5.4.5.1 Preliminary design for anode selection assessment

The design will have to take into account the evolution of the current demand and its foreseeable reduction after the initial energization period, so an anode current density higher than 110 mA/m² can be considered over a limited period of time (typically the first year).

When concrete is exposed to a wet and dry environment, the circuit resistance between anode and reinforcement steel changes constantly, this will affect concrete resistivity and, as a result, mean unbalanced current distribution may occur.

The designer must anticipate this type of behaviour in order to be able to choose the right type of anodes and the location of the measurement equipment accordingly.

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*Comments:*

*The designer must analyse the construction documents and anticipate the risks of insufficient embedding of steels: if concrete cover between the anode and reinforcing steel is less than 5 mm, an electrolytic or near short may develop, and it may result in the discharge of a highly concentrated current density at the point of near contact and in the case of impressed current systems, leading to acidification of the cementitious material around the anode.*

*In some cases, the addition of an additional concrete overlay (i.e shotcrete) prior to the anodic system may be necessary.*

### 5.4.5.2 Calculation of total resistance

The calculation of the total resistance circuit must take into account the driving voltage Ψ*. In the absence of factual elements, a value of 1.5 volts will be chosen and validated by field trial's results.

*Comments:*

*The voltage drop in an electrical conductor is easy to calculate by the formula, $R_{cable} = \rho \cdot x\ (L/s)$, and quickly indicates to the designer the type of metal (resistivity ρ), the length (L) and cross-section (s) of electrical conductor to be selected.*

*However, the anodic resistance Ranode (90% of total resistance) is complex to calculate: the anodic resistance calculation rules, usually used for earth and marine sectors, are based on the assumption of an infinite medium, which by definition cannot be the case in concrete, therefore the assumptions for the estimation of the anodic resistance in reinforced concrete sector should as far as possible be validated by a field trial.*

### 5.4.5.3 Field trial design and assessment

The field trial test should confirm:

- the choice of anodic systems;
- the protective current per anode zone;
- the anode current density;
- the polarization towards more electronegative potential values;
- the achievement of the performance criteria.

*Comments:*

*In cases where new anode types are used or where protection should be achieved through large concrete cover, it is recommended that a field trial test be conducted to validate the design assumptions. The test must be representative of the structure's foreseeable operating conditions and carried out over a period that takes into account the variability of these conditions according to seasonality.*

*The following test shall be conducted: cathode polarization test (preparation of test: temporary anodes, reference electrode, voltmeter with high resistance, direct current supply device...)*

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### 5.4.6 Supporting documents

The documents that may be required to support the cathodic protection installation are discussed in this section. The extent to which these documents are required will depend on the size and complexity of the designed system.

*Comments:*

*Particular attention will have to be paid to the level of knowledge and experience of the personnel involved in the drafting of the documents relating to cathodic protection*

*According to the recommendations of ISO 15257, the persons in charge of the design must be certified at level 4 in the reinforced concrete sector.*

#### 5.4.6.1 Design documentation

The design documentation should contain:

- calculation notes;
- detailed design;
- material & equipment list / specification (anode, reference electrodes, cables, transformer-rectifier unit, …);
- installation procedure and drawings (electrical architecture, anode layout, …);
- control equipment specification.

#### 5.4.6.2 Field trial

Field trial should include:

- specific assumption notes;
- test report including results from monitoring, analysis and interpretation of results;
- recommendations and modifying specifications of the design assumption notes if necessary.

#### 5.4.6.3 Commissioning report

Commissioning should include:

- visual examination and identification of materials, equipment and components;
- pre-commissioning checks of all circuits;
- survey of native cathodic and anodic potentials before polarization;
- initial energization, check polarization;
- check and verification of remote monitoring system;
- verification of system functioning between 7 and 30 days;
- achievement assessment of efficiency criteria.

*Comments:*

*In practice, several months up to one year may be needed to fully develop protection and establish operational parameters, especially with seasonal variation of temperature and moisture.*

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#### 5.4.6.4 Operation

It should describe checks and reporting during the working life of a system. In short: electrical checks every six months, visual checks every year; replacement of defective components

## 5.5 Intervention

### 5.5.1 Preparatory works

#### 5.5.1.1 Surface preparation

Before the installation of cathodic protection takes place, all spalls, cracks and delaminations, which can create discontinuity between reinforcement and anode, must be repaired. Cracks with an opening over a given size must be sealed according to specifications for that particular cathodic protection repair solution.

When conductive coatings are used as anodes, the substrate shall be free of potential short-circuiting points, such as nails, iron wire, etc. The surface shall not be treated such that the electrolytic contact between the anode and cathode is broken.

*Comments:*

*Injections of cracks must obligatorily be carried out with cement-based products (super fine binders) in order to maintain electrolytic continuity.*

*The use of injection products based on polymer binder (epoxy sealant, …) should be prohibited depending on its electrical resistance.*

#### 5.5.1.2 Systems trials

A reference zone shall be made at the beginning of the work. This zone shall act as a reference for the work to be executed, and for the final criteria to be applied.

*Comments:*

*Before further work, it will be necessary for the reference zone to be accepted as satisfactory by the contractor and the client. The size and location of the reference section shall be stated in the specification.*

### 5.5.2 Execution procedure

#### 5.5.2.1 Establishing reinforcement continuity

If a new reinforcement connection shall be established, this shall be achieved by exposing the reinforcing steel at points of intersection and then attaching them one to another.

After work is completed, new measurements shall be taken in order to document that the continuity level is satisfactory.

*Comments:*

*When welding is used, each weld shall be placed in areas of low strain and it must be ensured that the welds do not reduce the strength of the reinforcing bars.*

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### 5.5.2.2 Establishing electrical contact with the reinforcement

A minimum of two negative electrical reinforcement connections shall be established within each zone. This shall be accomplished by exposing and cleaning the reinforcing steel and then fastening the connections by screwing or welding, if the steel is weldable. Connections shall be durable.

Where screw connections are used, all connection points between wiring and reinforcing steel shall be covered and insulated with a non-conductive material.

*Comments:*

*Quality of contact between rebars and negative contact shall be verified by checking whether the potential difference between them is less than 1 mV, or not, and have a long term electrical resistance less than 0,01 ohm.*

*A terminal with a welding connection shall be of a commercially available material having a potential equivalent to that of reinforcing bars.*

### 5.5.2.3 Installation of impressed current anodes

The installation of anodes shall be executed according to the supplier's instructions. There must be no short-circuiting between the anode and cathode. The quality of these contacts shall be verified by checking that the potential difference value is more than 10 mV.

#### *5.5.2.3.1 Mesh anodes*

The mesh anodes shall be placed on and fastened to the concrete surface with plastic clips, see Fig. 5-14. The anode system comprises mortar or shotcrete that has electrolytic characteristics similar to those of the concrete substrate.



Fig. 5-14 mesh applied under a slab before covering it with dry shotcrete.

Care must be taken when cutting and installing the anodes. Under no circumstances must the anode come into contact with the reinforcing steel. The distance between the mesh and reinforcing steel shall be a minimum of 15 mm or what is stated in the supplier’s instructions.

#### *5.5.2.3.2 Ribbon anodes*

The ribbon mesh anodes shall be placed on the concrete surface and fastened to the concrete surface with plastic clips, see Fig. 5-15. The anode system comprises mortar or shotcrete that has similar electrolytic characteristics to the concrete substrate.

Otherwise, ribbon anodes shall be installed in specially made slots or cuts in the concrete surface. Cuts shall be filled with a low resistive mortar.

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Fig. 5-15 ribbon mesh applied before covering with dry shotcrete.

#### *5.5.2.3.3 Discrete anodes*

Discrete anodes shall be installed in holes made to accommodate the individual anode sizes. The holes shall be filled with a low resistive mortar.

Anodes shall be installed in such a manner that a break in an individual anode will not affect the performance of the other anodes.

Fig. 5-16 describes discrete anodes and typical implementation on a concrete support.

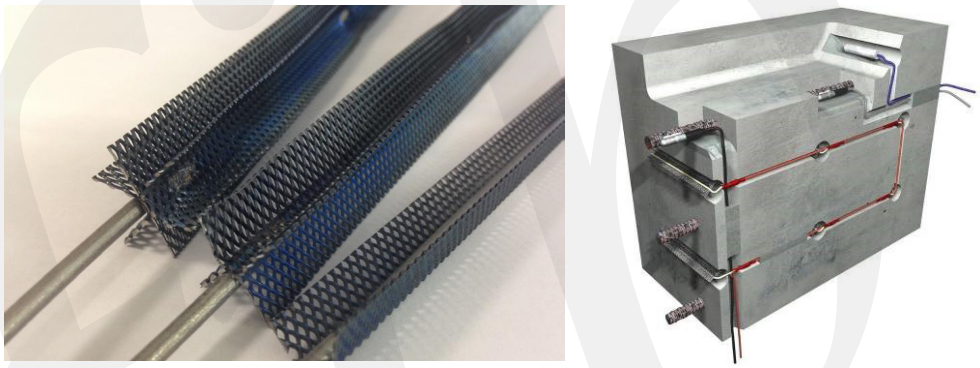


Fig. 5-16 typical discrete anode and implementation.

#### *5.5.2.3.4 Conductive coatings*

The conductive coating shall be applied directly to the concrete surface in accordance with the directions for the surface treatment. The conductive coating thickness and electrical distribution bands or threads shall be executed according to the supplier's instructions.

### 5.5.2.4 Installation of mains supply voltage

All installations that connect the main electrical supply to the control switchboard shall be executed by a certified electrician.

*Comments:*

*The system shall be grounded and have a ground neutral circuit breaker. All wiring shall be grouped together in a cable tray, conduit or similar, which has been correctly attached to the construction.*

### 5.5.2.5 Checks prior to application of electrical current

Before the voltage is applied, all the system components shall be checked separately to ensure that they are installed correctly and function as intended.

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*Comments:*

*Measurements shall document that the electrical current is distributed to each zone as intended, and that no short-circuiting or current leakage is occurring.*

#### 5.5.2.6 Adjustment and control of current and voltage

After electricity has been switched on, the current and voltage levels shall either be registered continuously with a logger/printer or measured manually once a week during the first month (if not otherwise stated) and later, as needed. Current/voltage shall be such that the predetermined protection criteria are met, and adjusted when they are not met. Systems which use a trial period longer than one month shall take that into consideration when adjusting and monitoring current and voltage.

*Comments:*

*In order to determine the amount of impressed current, the depolarization test is generally conducted.*

#### 5.5.2.7 Installation of galvanic anodes

When sacrificial anodes are used, care shall be taken to demonstrate that the anode is properly electrically connected to the reinforcement and well embedded.

## 5.6. Quality control

The contractor is required to have an experienced site manager stationed at the construction site who is able to survey the work done on a continual basis. The site manager and the rest of the construction crew shall have experience / knowledge of the concrete rehabilitation process using cathodic protection.

### 5.6.1 Quality control of materials

The materials used in the process such as patching material, crack fillers, anodes, cables, transformer-rectifier, reference electrode, etc. must meet quality standards. Including calibration certificate and control process of measuring equipment (voltmeter, reference electrode, …)

### 5.6.2 Quality control before intervention

Check if all the steel bars are electrically connected to each other in each electric circuit. Electric conduction must be secured by taking an appropriate measure if any defect is detected. Reinforcement continuity shall be monitored within each section or zone where cathodic protection is to be applied.

The requirement to demonstrate that the reinforcement continuity is satisfactory is that the electrical resistance measured between two points within the zone shall be less than 1 ohm.

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*Comments:*

*The electrical resistance or potential differences measured between two points (one being the drainage terminal) within the zone shall be less than 1 ohm or less than 1 mV respectively.*

### 5.6.3 Quality control during and after intervention

The contractor is responsible for executing the work according to the tender documents and work specifications. The materials used shall satisfy the specified demands.

The inspection log shall include data on the inspector used, time, place, inspection area and/or zone, inspection results and comments on the inspection. The daily journal shall contain data on temperature, humidity levels, time, place, crew members and work completed.

Random sample inspections made by the client do not release the contractor from the entire responsibility for the work completed. The contractor is required to repair or replace non acceptable work.

*Comments:*

*The contractor shall keep an inspection log and daily journal of work. A copy of the inspection log and daily journal shall be sent to the client when requested.*

### 5.6.4 Half cells

The reference electrodes commonly used to monitor cathodic protection systems are Silver/Silver Chloride and Manganese Dioxide.

Silver/Silver Chloride should not be used in areas of high moisture due to leaching leading to instability. Another issue with Silver/Silver Chloride is the consumption of chloride every time a measurement is taken.

A potential issue with half cells is that the porous tip comes out of contact with the concrete due to small movements. Some control systems record the potential, depolarisation and impedance. If the impedance becomes high due to the tip losing contact this is shown on the test results.

Decay probes are electrodes that may be used to check depolarization during up to 24 hours. Most decay probes consist of pieces of MMO coated titanium (same as anode material), embedded in mortar and placed in boreholes; or placed in repair spots and embedded in repair mortar. Decay probes are inexpensive and have long working lives (30 years experience).

*Comments:*

*When adopting manual monitoring every 3 months, this is not an issue, but with modern automated monitoring measurements, it can be taken as frequently as 30-minute intervals. Calculations show that at this rate the chloride might be consumed in less than 10 years.*

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## 5.7 Monitoring and maintenance

### 5.7.1 Monitoring for ICCP

Voltmeter / multimeter and a data logger with an internal resistance of 10 MOhm or more shall be used when establishing and monitoring a cathodic system.

For any structure, any representative steel in concrete location shall meet one of the following criteria:

a) an "Instantaneous OFF" potential more negative than –720 mV with respect to *Ag/AgCl*/0.5 M KCl;

b) a potential decay over a maximum of 24 h of at least 100 mV from "Instantaneous OFF";

c) a potential decay over an extended period (typically 24 h or longer) of at least 150 mV from the instant off subject to a continuing decay and the use of reference electrodes (not potential decay probes) for the measurement extended beyond 24 h.

It should be noted that it is not necessary to meet more than one item of a), b) or c).

No instant off steel/concrete potential more negative than –1'100 mV with respect to *Ag/AgCl*/0.5 M KCl shall be permitted for plain reinforcing steel or –900 mV for prestressing steel.

When determining protection criteria, the number of reference electrodes, measuring frequencies and the requirements for acceptance criteria must all be stated. Authorization requirements shall be stated, and may be based on a percentage of the reference electrodes, which satisfy the requirement, a percentage of the individual measurements for each electrode, which satisfies the requirement, or a combination of these two alternatives.

### 5.7.2 Maintenance

The operation and maintenance inspection and testing intervals and procedures shall be as recommended in the operation and maintenance manual or, as subsequently modified, based upon the performance of the system.

The intervals and procedures for routine inspection and testing vary from one cathodic protection system to another, depending upon the structure type, the cathodic protection system type, the reliability of power supplies (impressed current systems), the reliability of the galvanic anode in that environment and the vulnerability to accidental or deliberate mechanical or electrical damage.

### 5.7.3 Post-intervention documentation

It shall be documented that the cathodic protection system fulfils the requirements of the final system.

The contractor shall document that the chosen materials and products satisfy the given requirements. If these demands are not satisfied, the contractor is required to use other products, which satisfy these demands. The contractor shall not receive any compensation for the other materials and products used.

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A copy of the material specifications/product brochures shall always be present at the building site.

Prior to acceptance of the completed work, the contractor shall deliver a report upon his work, which shall be included in a final report for the project.

The final report shall act as the basis for later monitoring and checking, and shall therefore contain as much basic data as possible. It shall be easily understood and well arranged such that those who are unfamiliar with the cathodic system may be able to take control measurements at a later date without difficulty.

The summary report shall contain, as a minimum, the following:

- Description of the construction;
- An overview of suppliers/contractors/consultants involved;
- Product and material specifications;
- Quality documentation of products and materials;
- Discrepancies in products/materials/workmanship;
- Drawings of the system;
- Testing undertaken upon the system;
- Maintenance and follow-up instructions.

### 5.7.4 Long term maintenance

Long term operation and maintenance of CP systems requires regular checking of protection quality, usually every half year, and visual inspection every year; and reporting the observations, suggestions for adjustment and/or need for replacement of components. This shall be agreed by contract for a period of at least ten years.

## 5.8 Case study 1

| | |
|---|---|
| Project Title: | Saint Cloud Viaduct, Paris, France |
| Location: | Paris, France. |
| Client: | Direction Régionale et Interdépartementale de l'Equipement et de l'Aménagement Île de France (DRIEA) |
| Designer: | FREYSSINET |
| Contractor: | FREYSSINET France |

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### 5.8.1 Project Details

The St Cloud viaduct (Fig.5-17) allows the A.13 motorway to cross the Seine between the communes of Boulogne and Saint-Cloud in the Hauts de Seine department. Its total length is 1'103 m measured between the bearing axes of the outer abutments. The deck (Fig. 5-18) is composed of three cell box girders 3.60 m high, comprising 2 vertical walls and 2 inclined lateral walls. The wall thickness is 25 cm. The width of the upper slab varies from 20.40 m to 17.00 m.

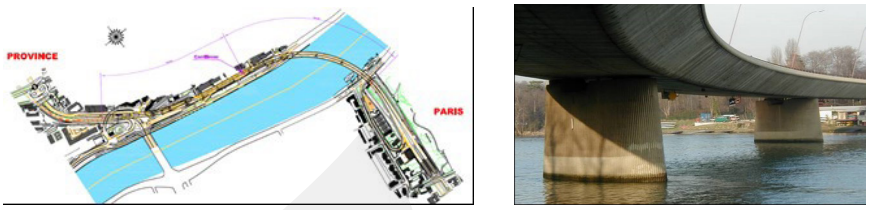



Fig. 5-17 Plan view and photograph of the viaduct.

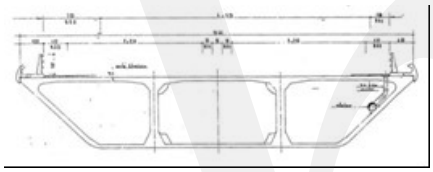


Fig. 5-18 Geometry of the segment: three-cell box girders.

A cantilever support, located at the mid-span of span 9, separates the structure into two parts called the access viaduct and the bridge over the Seine, which measure 574 m and 529 m respectively.

The assembly consists of 518 segments assembled by prestressed tendons, 16 over piers (i.e. 16 bays) and 2 abutments. Starting from the C1 abutment on the Saint-Cloud side, the geometry in plan develops as a curve with a radius of 700 m, continues in straight alignment for about 250 m and then crosses the Seine as a curve with a radius of 360 m. The longitudinal slope, which is more or less constant, is of the order of 0.8%.

Since the beginning, the subject of maintenance is an important one: the main dates to be retained are the following ones:

- 1972 to 1975: Construction of the viaduct
- 1979 to 1982: Implementation of additional prestressing inside the boxes,
- 1986 to 1987: Implementation of active studs in the upper lateral upper hopper between segments to recompress the deck,
- March 1998: Rupture of a 12T15 cable laid between 1979 and 1982,
- 2005 to 2009: Re-tensioning of additional prestressed tendons,
- 2012 to 2015: Implementation of acoustic protection (side screens on the viaduct),
- 2015 to 2017: Repair of the external facings of the brackets and implementation of cathodic protection by impressed current of reinforcing steel.

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### 5.8.2 Diagnosis

Throughout its history, the Saint Cloud viaduct has been the subject of particular monitoring because the traffic on the structure is important with 120'000 vehicles/day and totally cutting off traffic is unlikely.

The thickness of the concrete cover (in accordance with the standards at the time of construction, between 20 and 30 mm) proved to be insufficient, leading to active corrosion of the reinforcing steel and spalling of the concrete on very large areas Due to insufficient external cover and exposure to de-icing salts, corrosion of the external steel of the braces developed significantly, and the bridge owner had installed protective nets to prevent concrete from falling on the users.

The repair of the corrosion damage was first evaluated through a field trial and followed by the complete repair of the bridge. The field trial studied the feasibility of an Impressed Current Cathodic Protection (ICCP) with discrete MMO/Ti anodes embedded in the concrete (mainly from the interior of the segments). Then, Freyssinet France was commissioned by DREIA to carry out the repair, the implementation work and the monitoring of ICCP.

### 5.8.3 Field Trial

A field trial was carried out in 2012 over a 12 months period (DRIEA, Cerema, Gustave Eiffel University (Ifsttar)), on 3 segments, including one segment on pier (VSP2) (see 5.8.10 for reference).

The results allowed:

- Selection of the anodic system as discrete drilled activated titanium anodes (no overload, repair work from the interior of the viaduct);
- Current density ranges from 2 to 9 mA/m² steel;
- Achievement of performance criteria after an energising period of 2 months;
- No risk of hydrogen embrittlement;
- For the segments over pier, an extra protection was necessary in the area of prestressed tendons.

### 5.8.4 Design

The ICCP system was designed taking in account:

- The surface area of the steel reinforcement which was calculated for each segment;
- The electrical continuity between prestressed tendon and reinforcing rebar, (verified on site for each segment);
- The anodic resistance of each sub-zone was calculated and verified during the commissioning process, which revealed significant variations in the resistivity of the concrete and forced the designer to adapt the design of the ICCP system.

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### 5.8.5 Key aspects of the work carried out

- Determination of the precise location of steels and prestressed tendons by radar;
- Carrying out the concrete repairs and implementation of the reinforcement connections;
- Hydro stripping of 11'000 m² of exterior surfaces of the braces;
- Installation of 800 measuring points on the entire viaduct (reference electrode and depolarization probes).

### 5.8.6 Implementation of the ICCP system

Installation of 37'000 discrete anodes by gravity grouting inside the viaduct and 8'000 discrete anodes by injection grouting on the outside of the viaduct (upper zone inaccessible by the inside),

- Installation of ribbon mesh on cantilever support;
- Mesh on external faces;
- by an externally activated titanium surface anode (MMO/Ti mesh) on battery-powered segments;
- Per surface anode made of activated titanium (MMO/Ti ribbon mesh) on the inside on the cantilever support elements;
- Electrical continuity validation of anode and cathode circuits;
- Wiring of anodic and cathodic circuits;
- Installation and commissioning of control and monitoring equipment: 90 slave cabinets and 2 main cabinets;
- Application of 500 m² of dry sprayed mortar to repair the external facings;
- Polymer modified cementitious coating: 330 m² on external surface;
- Requires specialized personnel certified in CP.

### 5.8.7 Results

ICCP system is studied in order to estimate the long-term durability of such a treatment and its maintenance.

### 5.8.8 Special constraints

- Difficulties of accessing the work areas inside the cabinets;
- Difficulty of accessing the exterior cladding;

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- Complex zoning of the cathodic protection according to variations in steel densities, the presence of prestressed tendons, and a high density of electrical networks inside the viaduct;
- Environmental constraints: traffic on and under the structure;
- Significant variation in concrete resistivity (x 2,5 compared to market assumption);
- Variation in the length and number of anodes affecting the overall anode circuit resistance,
- Use of optical fibre for the transmission of information (length of the structure),
- Specific quality control plan.

These points are illustrated in Figs 5-19 to 5-21.

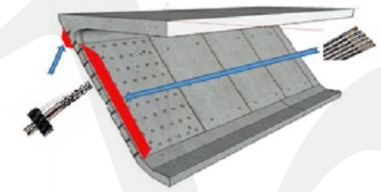


Fig. 5-19 Positioning of drilled discrete anodes.

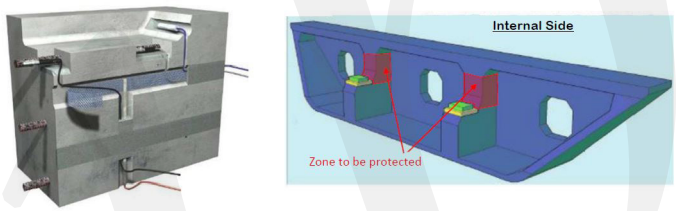



Fig. 5-20 Ribbons mesh applied on cantilever.



Fig. 5-21 Titanium Mesh under dry sprayed concrete.

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### 5.8.9 Monitoring of the ICCP system

- Sufficient protection quality was demonstrated by 98% of the anode zones achieving more than 100 mV depolarisation over 24 hours
- 798/800 measuring equipment are working properly,
- In addition to depolarization, the $E_{Off}$ cathodic potential is monitored in order to verify that there is no risk of hydrogen embrittlement, $E_{Off}$ > -900 mV vs *Ag/AgCl*/0.5 M KCl.

Freyssinet carries out installation monitoring under a multi-year maintenance contract awarded to it by the project owner. Monitoring is carried out remotely via a GSM link, which provides direct access to the control unit, to record the measurement data and, if necessary, to modify the set-up.

### 5.8.10 Reference

- Bouteiller, V. et al., "Impressed current cathodic protection with embedded anodes – field trial on the Viaduc de Saint-Cloud in France", *Concrete Solutions 2019*, MATEC Web of Conferences 289, 03002 (2019).

## 5.9 Case study 2

| | |
|---|---|
| Project Title: | Nouvelle Route du Littoral, Reunion Island, France |
| Location: | Reunion Island, France. |
| Client: | Conseil Regional de la Réunion |
| Designer: | Freyssinet |
| Contractor: | Groupement Vinci |

### 5.9.1 Project Details

In 2013, The Conseil Régional de la Réunion (The public authority of Reunion Island) let a contract for the construction of a 12.5 km long coastal road connecting the two main cities of Reunion Island, Saint-Denis and La Possession (Fig. 5-22). Known as the MT3 project, part of this roadway consists of a 5.4 km long marine viaduct along the coastal perimeter of the island. Standing between 20 m and 30 m above sea level, and located 70 m away from the coast, the elevated highway serves to accommodate buses, cars, pedestrians and bicycles across a dual three-lane configuration.

The bridge itself stands atop 48 massive reinforced precast concrete piers standing from seabed to road deck. During the design planning of these works, it was decided that due to the exposure of these piles to aggressive marine conditions, a cathodic prevention system would be installed to maximize the durability of the structure over its 100 years design life. As such the principal contractor let a design and construct contract to supply a turnkey cathodic prevention solution to provide this design life.

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Fig. 5-22 Global views of MT3 Bridge 4.

It should be pointed out that the CPrev system came in complement to the high quality standard concrete mix combined with concrete cover in average of 7 cm to comply with the local and European governing civil engineering codes.

From a structural standpoint the piers have been designed to withstand the recurrent extreme cyclonic storms from the Indian Ocean and to withstand the impact of a tanker ship.

Each of the 48 massive reinforced piers stand at an average of 18 m tall and are 8.6 m in equivalent diameter (ellipsoidal shape). The piers themselves are hollow, with a wall thickness of 1.1 m. They are supported by a large concrete foundation 20 m diameter and 4 m high that rests on the sea bed which serves as a gravity base foundation.

The piers were conventional steel reinforced concrete, precast on-shore at the construction yard in two separate elements which were then lifted and positioned in the sea as shown in Fig. 5-23.




Fig. 5-23 Typical view the pier precast elements.

The project specification calls for full protection for the total immersed area of the piers and the tidal/splash area of 3 m high. The intent of the cathodic prevention is to ensure full corrosion protection of the hollow part of the submerged pile (which may be subject to oxygen concentration cells) and the described tidal/splash area. The immersed part of the piles ranges from 5.5 to 9.5 m.

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It should be emphasized that the massive concrete foundation was not intended to receive protection (from a contractual stand point). The foundation's reinforcing steel, however, needed to be included in the design philosophy due to the possibility of current drain as per the DNV definition. The design calculation leads to a current demand for the submerged area of the piles ranging from 0.7 to 1.3 A whereas the concrete foundation stands at a 2.73 A drain.

This is based on a steel current design density of 3 mA/m² of steel. The total current demand for the submerged part, therefore, ranges from 3.4 to 4.47 A depending on the pier height.

The anodic system was divided, in each block, in several zones electrically independent, and each zone was fed by a different output of the feeding source, in order to ensure an effective control of the current to all the concrete surface.

The anode system design, Fig. 5-24, relies on 6 submerged discrete disc anodes distributed around the circumference of the piers at two different depths. Zone 2 is considered to be the primary prevention location (where the most severe submerged corrosion may take place). A dedicated power supply has been assigned to zone 3 to concurrently protect the lower part of the pile and to control the current drain to the concrete foundation.

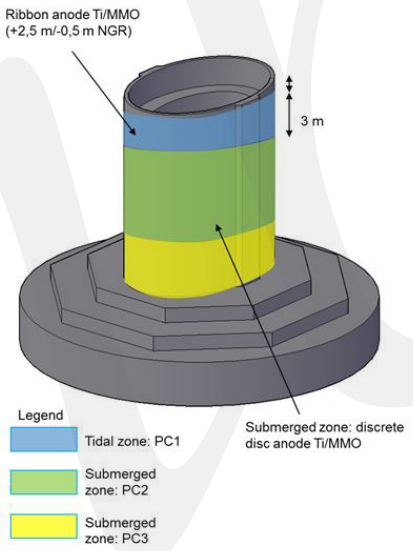



Fig. 5-24 Typical view the pier precast elements.

The anode system for the tidal/splash area relies on a "typical" (for CPrev system) activated titanium mesh ribbon installed within the rebar cage. The current requirement of zone 1 ranged from 0.64 A to 0.68 A based on a design current density of 3 mA/m² of steel. The latter was based on the upper value given in ISO 12696 (2 mA/m² for prevention) with a contingency factor of 50 %. Detailed steel surface area calculations were conducted to take into consideration all steel comprised within 25 cm deep of the concrete wall (which is 1.1 m total) as it was considered to be the anticipated throwing power of the anode system

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and less steel density remains between the main inner and outer steel mats as it was mainly cross link bars.

The steel surface area per square metre of concrete surface ranged from 2.466 $m^2$ to 3.177 $m^2$. The variation of the local current requirement within the various steel density of the structure did not imply the use of reduced spacing from the 300 mm centre to centre selected design spacing.

By definition CPrev installation works take place in conjunction with the construction works. This implies that full cooperation and communication is required during the construction stage between the various construction contractor personnel in charge of different aspects of the construction work and between the construction parties (i.e. joiners, steel fixers, site personnel in charge of the CPrev installation).

Critical aspects of the CPrev installation are described thereafter and the successful installation of the CPrev system strongly relies upon not only on a comprehensive testing schedule implemented in the installation procedure but also the full time assignment of CP certified personnel in the reinforced concrete sector to conduct these tests.

The testing plan with associated test or approval to proceed sheets were:

- Calibration of reference electrode;
- Continuity assessment of steelwork;
- Embedded anode system installation;
- Immersed anode mounting assembly installation;
- Negative & positive cable installations;
- Reference electrode, potential decay probe and test connection installations;
- Testing prior to concrete pouring operation (and complete closure of formwork);
- Testing during concrete pouring operation;
- Post concreting testing at several time intervals (from construction yard to offshore).

The installation of a cathodic prevention system of the 5.4 km long marine viaduct at the Reunion Island, considered as the largest CPrev in France, has proved to be very successful with all components installed satisfactorily and no damage occurring to any component.

The successful installation of the CPrev system strongly relied upon not only a comprehensive and rigorous testing schedule but also on the assignment of CP certified personnel in the reinforced concrete sector to conduct these tests.

Polarization results indicated that the atmospherically exposed part of the concrete meet the criteria for an operating current density of 1.5 $mA/m^2$ of steel. The submerged part equipped with “traditional” seawater anode system performed at an operating current density of 3 $mA/m^2$, which is considered to be relatively high for a water saturated and oxygen limited concrete according to the available literature. The presence of un-polarized bare steel items may contribute to this current demand.

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When designing a CP system for a submerged reinforced concrete structure using immersed anodes, care must be exercised to consider any metallic item in close proximity of the CP object which may affect the CP performance depending on the type materials, surface and surface condition (coated or not).

### 5.9.2 Reference

- Hallopeau, X. et al, "Field application of cathodic prevention on a marine viaduct", *Corrosion 2020*, paper no. 15034 (Houston, TX: NACE, 2020).
- Hallopeau, X. et al, "Application of Cathodic Prevention on a Marine Viaduct", *Material Performance*, Vol. 10, October 2020.

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# 6. Realkalisation

## 6.1 Foreword

Realkalisation (Fig. 6-1) is a method used for the repair of reinforced concrete subject to carbonation by increasing the alkalinity. It involves electrochemical reactions. The aim of this method is to prevent reinforcement corrosion by increasing the pH to values higher than 10.5. A direct current is applied between a temporary metal anode set on the concrete surface and the rebar embedded in the concrete for a limited period of time. At the rebar/concrete interface, the rebar is cathodically polarized and the pH increases due to the hydroxyl ions formation. Additionally, the external carbonate solution migrates inside due to the applied electrical field. Together with the carbonate solution, the cations (e.g. alkaline ions) migrate towards the rebar and the anions (e.g. hydroxyl ions) produced at the rebar migrate towards the anode.

*Comments:*

*Typically, a direct current of less than 5A per 1m² reinforcement surface or around 1A per 1m² concrete surface is applied for one to two weeks. (see 6.4.4.3)*

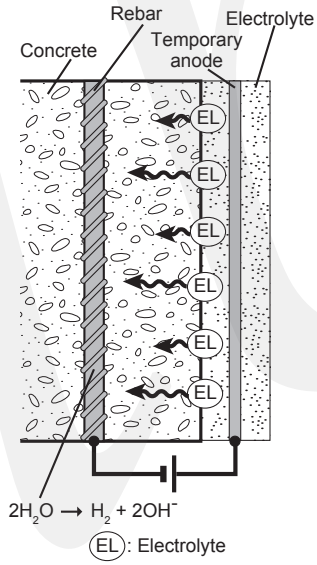



Fig. 6-1 Illustration of realkalisation.

This chapter was written by Andrade, Bouteiller Habuchi, Hallopeau, Kobayashi (main author), Miyaguchi, and Ueda.

https://doi.org/10.35789/fib.BULL.0102.Ch06

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*Comments:*

*The electro-osmosis is the reason why further carbonation does not lead to depassivation again. The buffer of carbonate/bicarbonate whose pH is 10.4 is generated.*

*Also, the following reaction occurs around the rebar during this period, whereby the surrounding concrete pH is increased:*

$$2H_2O + O_2 + 4e^- \rightarrow 4OH^- \quad (6\text{-}1)$$

*If a sufficient amount of oxygen is not supplied, the following alternative reaction takes place:*

$$2H_2O + 2e^- \rightarrow 2OH^- + H_2 \quad (6\text{-}2)$$

*After the treatment, the following chemical equilibrium is established, whereby the pH is maintained around 10 to 11:*

$$R_2CO_3 + CO_2 + H_2O \leftrightarrow 2RHCO_3 \quad (6\text{-}3)$$

*where R: Alkaline ion*

## 6.2 Basics

### 6.2.1 When to adopt this method

Realkalisation is used for structures that are suffering rebar corrosion by carbonation (reduction of concrete pH). This treatment prevents further progress of rebar corrosion and an outbreak of new corrosion by recovering alkalinity of the concrete.

If structural performance has already been reduced, other repair techniques (patching or filling of the cracks) shall be employed instead of or in addition to the realkalisation.

Special care needs to be taken when applying the realkalisation method to ASR-damaged concrete or prestressed concrete (PC) structures.

*Comments:*

*Additional reinforcement would be required if the rebar corrosion has progressed significantly to the extent that the mechanical performance of the structure has already degraded.*

*When applied to concretes containing reactive aggregates, care shall be taken as regards to the accumulation of alkali around the rebar that acts as the cathode.*

*The possibility of hydrogen embrittlement needs to be taken into consideration for prestressed concrete (PC) structures with PC tendons. To avoid this, current shall be applied intermittently for PC structures (an example can be seen as Case Study of Chapter 7 Chloride extraction/Desalination).*

### 6.2.2 Materials and systems

The system for applying direct current between rebar inside a structure and an anode metal consists of the following members:

- Electrolyte Solution

  Sodium carbonate ($Na_2CO_3$), potassium carbonate ($K_2CO_3$), lithium carbonate ($Li_2CO_3$).

- Anode metal

  Mixed metal oxide-coated titanium, noble metal-coated alloy, stainless steel, etc.

- Electrolyte retainer

  Used for retaining the electrolyte solution on a concrete surface during the treatment.

- DC power supply

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*Comments:*

*It is essential to use an alkaline electrolyte solution that can sufficiently infiltrate the concrete by migrating from the surface to the rebar forced by the electrical field in order to develop the buffer couple with carbon dioxide and water as shown in the reaction.*

*Anode metal may suffer electrolytic corrosion during the period of the repair work. Therefore, metals that are resistant against electrolytic corrosion such as titanium shall be used as anodes. If carbon steel is used as an anode, it shall have a sufficiently large cross section, and it will likely stain the concrete surface.*

*The following are examples of electrolyte retainer:*

- *Cellulose fibre layer (Fig. 6-2): Sprayed on the concrete surface before applying the anode and after to cover the anode metal. Applicable on complex structure surfaces. The drawbacks are easy escape of electrolyte solution through evaporation and leakage.*



Fig. 6-2 Electrolyte retainer - Cellulose fibre layer.

- *Panel (Fig. 6-3): Set on the concrete surface to form a closed space filled with the electrolyte solution. While capable of retaining electrolyte solution well, panels can hardly be adopted for complex surfaces.*



Fig. 6-3 Electrolyte retainer - Panel.

- *Sheet (Fig. 6-4): Simply put the sheets (i.e., non-woven fabric cloth) on the concrete surface and anode metal. Only applicable on flat surfaces.*



Fig. 6-4 Electrolyte retainer - Sheet.

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## 6.3 Stakeholders' roles and qualifications

### 6.3.1 Owner

The owner must determine the following prior to the actual repair:

- Remaining service life of the structure;
- Performance requirements of the structure; and
- Expected carbonation depth after the repair.

The owner must share the design plans and construction records of the original structure with the designer and the contractor.

### 6.3.2 Designer

The designer must be an expert on the realkalisation technique and shall preferably be qualified or certified.

### 6.3.3 Contractor

The contractor must be an expert on the realkalisation technique and shall preferably be qualified or certified.

*Comments:*

*ISO 15257:2017 "Cathodic protection - Competence levels of cathodic protection persons - Basis for a certification scheme" can be referred to.*

## 6.4 Design

### 6.4.1 Assessment of existing conditions

The target structure must be inspected closely to assess the current conditions before designing the repair system. Investigations shall be conducted aiming at determining:

1. The carbonation depth;
2. Conditions of cover concrete - The width and length of cracks, loss of cross section and spalling of concrete, the shape of the concrete member, and presence/absence of a surface coating or other repair;
3. The rebar arrangement;
4. Environment – Temperature (and carbon dioxide concentration only in the special circumstance).

*Comments:*

*For proper installation of the anode system on the concrete surface, the shape of the concrete member, and conditions such as cracks and section loss in concrete must be investigated beforehand.*

*The rebar arrangement must be specified for the circuit design, such as connection points between the rebar and lead wires, the number of connections, the DC power supply capacity, the number of power supplies, and so on.*

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### 6.4.2 Service life

Unless the structure is located in an environment where the impregnated electrolyte is easily washed away, the possibility of re- deterioration is low (no reports have been made so far).

### 6.4.3 Codes and standards and other relevant references

- EN 14038-1:2016 Electrochemical Realkalisation and Chloride Extraction Treatments for Reinforced Concrete - Part 1: Realkalisation
- JSCE Concrete library 157:2020, Guideline for Electrochemical Corrosion Control Method
- NACE standard SP0107-2017
- NACE report 01104 Electrochemical Realkalisation of Steel- Reinforced Concrete—A State-of-the-Art Report

### 6.4.4 Design Procedure

#### 6.4.4.1 Setting a target carbonation depth

A target carbonation depth after the realkalisation is set such as to restore the pH of the concrete around the rebar.

The entire concrete cover usually shows high alkalinity immediately after the repair.

#### 6.4.4.2 Arrangement of anode

1) Anode metal layout, and 2) type of electrolyte retainer (see 6.2.2) are selected based on the shape of the structure, the shape of the concrete surface, and the surrounding environment.

*Comments:*

*The surrounding environment here includes the tolerability to electrolyte leakage, potential risks for the third party, and whether the target structure is located indoors or outdoors.*

#### 6.4.4.3 Current density and period

Generally, an electric current of 1A is applied per 1$m^2$ steel surface or 1A per 1 $m^2$ concrete surface. The current density shall be determined for each application based on the rebar amount, imperviousness of concrete, and cover depth.

Current is applied generally for a period of 2 weeks or less. The current application period shall be determined for each case based on the current density and carbonation depth. Electro-osmosis phenomenon shall be noted and verified.

The electric circuit shall be designed based on the imperviousness of concrete, cover depth, DC power supply capacity, and lead wire capacity.

*Comments:*

*When applying a current per concrete surface area, the amount of rebar inside shall be taken into account. At least a current of 0.3A per 1$m^2$ concrete surface is necessary for realkalisation. On the other hand, application of 20A/$m^2$ or more can cause significant cracks within about 10 days.*

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*Care must be taken when there are large differences in the cover depth and the concrete imperviousness, since uneven current distribution will result in variations in the realkalisation effect. The electric circuit must be divided into several zones if such is this case.*

#### 6.4.4.4 DC power supply

The DC power supply must have a sufficient capacity for supplying a necessary amount of current.

*Comments:*

*Any physical contact with the concrete surface, lead wire, exposed metal, and so on during the process will cause electrical shock. For safety reasons, the voltage applied to the system shall be less than 50V.*

#### 6.4.4.5 Design of pre-treatment

Damaged concrete and reinforcement must be repaired before the realkalisation process.

The material for patch repair shall have a strength larger than that of the substrate concrete. Crack sealers must be able to close the cracks.

*Comments:*

*All the spalling, falling, cracks and other defects must be repaired beforehand to ensure that current will flow between the anode metal and the rebar. While compressive strength of repair materials is not required for the realkalisation process, the repair materials shall have the necessary strength.*

#### 6.4.4.6 Design of post-treatment

When the structure is exposed to water flows that can wash away the electrolyte, the necessity of applying surface coating for longer service life of the repair shall be considered.

*Comments:*

*Coating materials shall be removed before realkalisation if existing. If it is planned to apply surface coating after the execution of realkalisation, it shall be selected appropriately by considering the high humidity and high alkalinity (because of the electrolyte) of the concrete surface to be applied immediately after the realkalisation process.*

### 6.4.5 Supporting documents

Along with records of design details, a maintenance plan shall be prepared.

*Comments:*

*The maintenance plan must be prepared beforehand. After the repair, the anode system is dismounted and the structure is restored to its original state. The concrete structure can therefore be maintained in the same manner as that for ordinary concrete structures. It shall be mentioned if any of patching, crack sealing, and surface coating has also been carried out, and any particularities that need attention shall be specified in the maintenance plan.*

## 6.5 Execution

### 6.5.1 Preparatory works

Confirm the location and area of the treatment on site. Check if the temperature is higher than the freezing point of the electrolyte.

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Carry out the following preparatory works (see 6.4.4.5):

- Removal of deteriorated concrete and surface coating;
- Crack sealing;
- Patching;
- Restoration of reinforcement;
- Removal of metal parts in cover concrete (bolt, separator, etc).

*Comments:*

*Electrically insulating surface coating must be removed beforehand. Gypsum and plaster are conductive but can dissolve in the electrolyte.*

*Exposed and embedded metal parts can cause short-circuiting and must be removed by using a rebar detector or similar tools.*

*Metal objects within the execution area, such as guardrails, water pipes, and so on, shall be either 1) temporarily welded to the rebar, 2) temporarily removed, or 3) separated from the anode by more than 20 cm to avoid electrolytic corrosion.*

## 6.5.2 Execution procedure

### 6.5.2.1 Connection of rebar to DC power

Rebar is electrically connected to an anode of DC power supply. Electrical continuity of reinforcement must be checked.

*Comments:*

*Exact locations of rebar shall be determined by using a non- destructive detector so as to minimise unnecessary damage to concrete by drilling. An appropriate number of connection points between lead wires and the rebar shall be determined based on the current capacity of the lead wire and the diameter of the rebar. The electric current through the wire will be excessive if there are too few lead wires. On the other hand, too many lead wires will require unnecessary drilling. Usually, one connection per 30m$^2$ of concrete surface is appropriate.*

### 6.5.2.2 Installation of anode and supply of electrolyte

The anode metal is set in accordance with the prepared electric circuit design. The anode shall be set over the entire area of the realkalisation treatment. An electrolyte retainer (see 6.2.2) is set over the anode and the electrolyte solution is supplied. The amount of electrolyte is determined in consideration of expected loss caused by absorption into concrete and evaporation.

*Comments:*

*Since the reinforcement steel bars are electrically connected to each other, the circuit cannot be divided on the cathode side. Therefore, any necessary circuit division shall be achieved on the anode side.*

*The anode metal is set several centimetres apart from the concrete surface. A lattice spacing of anode mesh of 20cm or less has been known to be effective in making the realkalisation uniform.*

*A reference electrode is sometimes installed in order to check the half- cell potential of rebar.*

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#### 6.5.2.3 Connection of anode to DC power

The installed anode is connected to a cathode of DC power supply.

#### 6.5.2.4 Settlement of DC power

The DC power supply is installed.

#### 6.5.2.5 Application of direct current

Current is applied with a constant current control.

### 6.5.3 Finishing

Switching off the current, removal of system, repair of drilled holes, etc.

After finishing current application, the entire anode system is dismantled. Drilled holes and core boring holes are filled.

## 6.6 Quality control

### 6.6.1 Quality control of materials

The materials used in the process such as the electrolyte, patching material, crack fillers, cables, etc. must meet quality standards.

### 6.6.2 Quality control before intervention

Check if all the steel bars are electrically connected to each other in each electric circuit. Electric conduction must be secured by taking an appropriate measure if any defect is detected (see 6.4.4.5, 6.5.1).

*Comments:*

*All rebars must be electrically connected to each other in each electric circuit.*

### 6.6.3 Quality control during intervention

After starting the current application, the direction and magnitude of the current flow shall be monitored.

The electrolyte solution penetrates during the treatment forced by the electrical field. It shall be replenished to keep the concrete surface wet throughout the current application. The pH of the concrete surface must be kept over 10. The DC power supply, anode metal, electrolyte, and lead wires shall be checked regularly.

Special care must be taken in handling hazardous electrolyte solutions.

*Comments:*

*The electric current will vary to some extent since the cover depth, amount, and arrangement of rebar in the RC member are not uniform. Current variations within ±30% are acceptable. Any deviation from this range can indicate a connection failure between a lead wire and the rebar, or an electrical short circuit between the anode and the rebar. A voltage higher than a preset value may be an indication of a disconnection in the wire or a lack of electrolyte.*

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*Current and voltage shall be checked not only at starting the current application but also after the electrolyte has penetrated well into the concrete. It may also be salutary to check the half-cell potential of rebar with an internal/embedded or an external reference electrode during the current application. The potentials will be around -1100 mV (SCE) during a typical realkalisation current flows.*

*The concrete surface shall be checked if it is wet with the electrolyte at least once a week during the realkalisation process.*

### 6.6.4 Quality control after intervention

It will depend on the agreement with the owner. Both carbonation depth and rebar passivity level can be monitored over time. At least carbonation depth must be investigated.

If possible, also the rebar electrical potential shall be measured immediately and after several days in order to verify its evolution towards values in the passivity range.

*Comments:*

*The alkalinity of concrete shall be checked as far as to the backside of the rebar.*

*The half-cell potential of the rebar is substantially lowered immediately after the realkalisation because of being polarized by a large current. It can take from several months to one year until the potential returns to its intrinsic value that reflects the rebar corrosion behaviour. A reference electrode embedded near the rebar may be useful.*

## 6.7 Monitoring & maintenance

### 6.7.1 Monitoring

The structure that underwent re-alkalisation must be regularly inspected. In addition to routine visual inspections, the state of steel corrosion shall be examined with an electrochemical method, such as a half-cell potential method, every few years. Interpretation of data shall be done by an expert. Preferably, the carbonation depth shall be investigated every several to 10 years.

*Comments:*

*After the realkalisation treatment, the alkalinity of concrete around the steel is restored promoting re-passivation of rebar. For such a structure, an ordinary periodical inspection may be carried out.*

*In periodic visual inspections, attention shall be paid as to 1) whether there are new cracks or corrosion stains on the concrete surface, and 2) whether new spalling or falling in the cover concrete is observed.*

### 6.7.2 Maintenance

Surface coating may be replaced regularly (optional). See Chapter 2.

### 6.7.3 Post-intervention documentation

The contents of design, results of execution, quality controls, and maintenance procedures shall be appropriately recorded and kept.

*Comments:*

*See 6.4.5.*

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## 6.8 Case study 1

Project Title: Repair of a historical building
Location: Osaka, Japan
Client: Osaka city municipal government
Designer: Denka Co. Ltd.
Contractor: Denka Co. Ltd.

### 6.8.1 Description of the structure

A steel framed reinforced concrete (SRC) castle building (8 floors, 54.8m high), located in a public park in an inland area (Fig. 6-5), was built in 1931 and repaired by realkalisation method in 1994. The building is now used as a museum.



Fig. 6-5 SRC building.

### 6.8.2 Inspection highlights:

The outer wall concrete surface did not show severe spalling or a significant loss of rebar cross section. However, a partial delamination was observed, with traces of past patch repairs. The long-term performance capability of the building as a concrete structure was questionable.

To determine deterioration causes, concrete core samples were taken from the outer wall. The average carbonation depth of the samples was 29.4 mm, with a maximum of 67 mm.The average rebar cover depth was 50 mm.

### 6.8.3 Description of the intervention project:

To preserve the historic value of this beloved local landmark, a non- destructive repair technique was preferred, and therefore the realkalisation method was employed rather than concrete replacement, and applied to the wall of 5’100 $m^2$.

### 6.8.4 Description of the intervention works:

After treating areas suffering a sectional loss of concrete with patch repair, titanium mesh was set on the concrete surface as a temporary anode (Fig. 6-6), and cellulose

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fibre saturated with a sodium carbonate solution was sprayed thereon (Fig. 6-7). A direct current of 1A (per unit concrete surface) was applied for seven days, with a maximum voltage of 40V. Fig. 6-8 shows an example of the decrease in the carbonation depth during the realkalisation process. The depths were measured in samples taken from the concrete wall by using a core drill. The carbonation depth reached 0mm on the fifth day.



Fig. 6-6 Setting up metal mesh.



Fig. 6-7 Spraying cellulose fibre saturated with sodium carbonate solution.

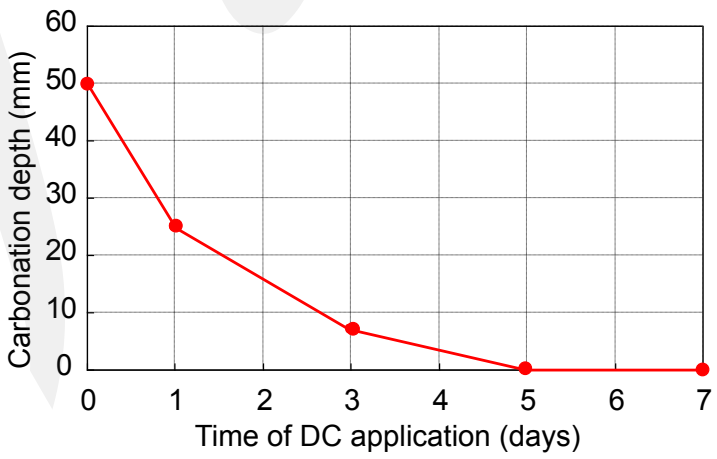



Fig. 6-8 Carbonation depth vs. DC application duration.

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### 6.8.5 Description of maintenance operations:

Fig. 6-9 shows the distribution of alkalinity of the wall measured 10 years after the treatment. After 20 years since the intervention, the building shows no sign of corrosion or concrete spalling.

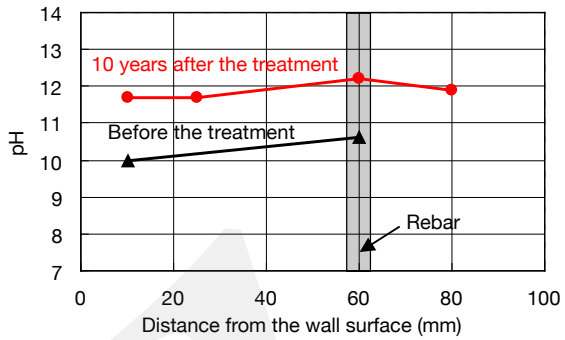



Fig.6-9 Alkalinity of the concrete wall.

## 6.9 Case study 2

| | |
|---|---|
| Project Title: | Repair of the Camille de Hogues Bridge by realkalisation |
| Location: | Châtellerault, France |
| Client: | DRAC (Nouvelle Aquitaine) |
| Designer: | RENOFORS |
| Contractor: | Communauté de communes du pays Châtelleraudais |

### 6.9.1 Description of the structure

Camille de Hogues Bridge (CHB) (Fig. 6-10) was built between 1899 and 1900 in Chatellerault by the François Hennebique Company to cross the Vienne River. It is one of the first reinforced concrete bridges in France. Consisting of 3 arches, it is 150 m long and 8 m wide. CHB was listed as Historical Monument in 2002. After more than 100 years, it needed an extensive repair.

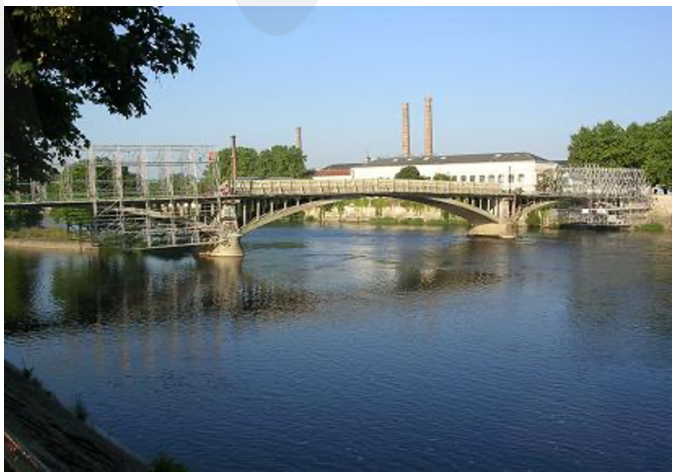


Fig. 6-10 Camille de Hogues Bridge across Vienne River, France.

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### 6.9.2 Inspection highlights

CHB was built with concrete using Ordinary Portland cement (≈380 kg/m$^3$) and a mix of siliceous aggregates together with the Hennebique stirrups system consisting of different shapes of reinforcing bars (Fig. 6-11).



Fig. 6-11 Camille de Hogues Bridge with the Hennebique stirrups system.

The bridge was suffering from generalized heavy corrosion of the rebars leading to cracking (Fig. 6-12) and severe spalling. Carbonation induced corrosion (low quality concrete compared to current standards, 20 MPa compressive strength and 21% porosity) has probably been increased by the absence of a deck waterproofing.



Fig. 6-12 Severe cracking of the beams.

### 6.9.3 Description of the intervention project

In July 2006, a restoration operation consisting of a combination of standard patch repair and an impressed current realkalisation treatment (Novbeton) was selected by the architect in charge of the work.

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## 6.9.4 Description of the intervention works:

Due to an erratic network of bars and cages, a connection to the rebars had to be made every 1 m² as a minimum (Fig. 6-13). Then a steel anode (3 mm in diameter, 5x5 cm mesh) was embedded in a cellulose pulp soaked with a potassium carbonate solution. Finally, a current density of 1A/m² of steel was applied over a 15 days period (Fig. 6-14). It is to be noticed that, due to permanent wind, the poultice had to be regularly moistened.



Fig. 6-13 A connection to the rebar was made every 1 m².



Fig. 6-14 The steel anode mesh was embedded in a paper pulp soaked with a potassium carbonate solution.

## 6.9.5 Research works:

In addition to the intervention work, a one-year research program was carried out on one of the arches, as part of a larger "realkalisation project" aiming at evaluating the efficiency, side effects, and durability of such electrochemical treatments. Fig. 6-15 represents zone 9 which was one of the investigated zones on the CHB. It consisted of 2 standard round shaped- rebars (one horizontal and one oblique, both 20 mm in diameter) and 9 flat Hennebique stirrups (4 to 4.5 cm wide).

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On the scheme: the two pentagonal-shaped points refer to the $R_p$ measurement spots, the white dot corresponding to the connecting point. The four triangular-shaped points refer to the powders drilled respectively before treatment, just after treatment, 2 and 12 months after treatment. Efficiency of the realkalisation treatment was followed according to pH evolutions (Fig. 6-16), potential mappings (Fig. 6-17), resistivity mappings and corrosion rate measurements.



Fig. 6-15 Cathodes and anodes connected to a generator with an impressed current density of 1A/m² of steel.



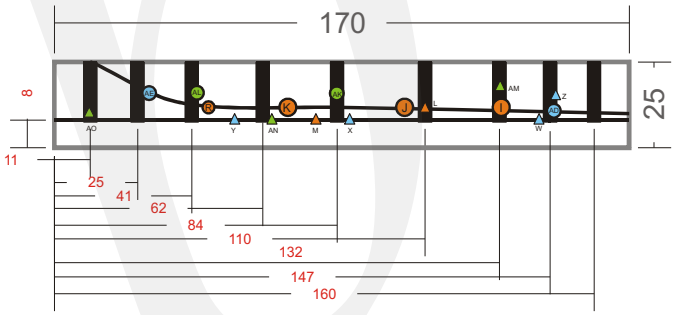




Fig. 6-16 Rebar network on zone 9.

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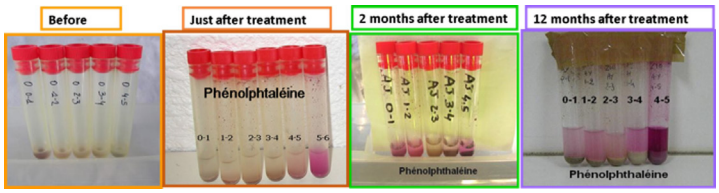



Fig. 6-17 Evolution of pH versus treatment.

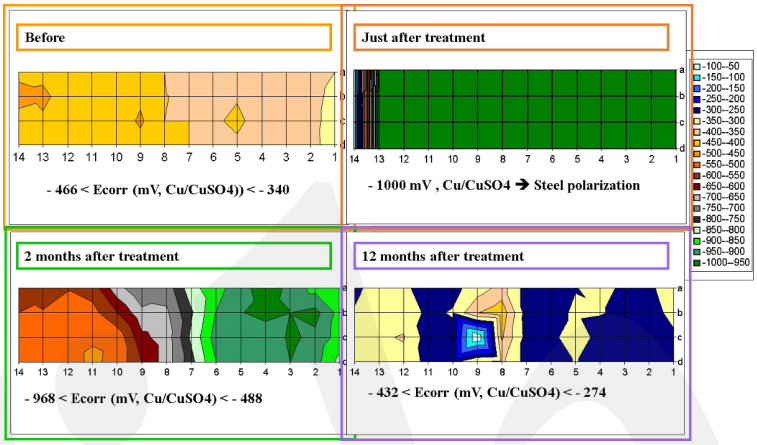



Fig. 6-18 Evolution of potential mappings versus treatment.

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# 7. Chloride extraction/desalination

## 7.1 Foreword

Chloride extraction (Fig. 7-1) is an electrochemical repair technique for extracting chloride ions from concrete structures. A temporary anode system consisting of a metallic anode and an alkaline electrolyte solution is set on the concrete surface, and direct current is applied between the anode and the reinforcement (= cathode) in the concrete for a limited period of time.

*Comments:*

*Typically, a direct current is applied between the reinforcement (cathode) and an external anode in electrolytic contact with the concrete. A current density of less than 5A per 1m² reinforcement surface or around 1A per 1m² concrete surface is applied for about eight weeks. (see 7.4.4.3)*

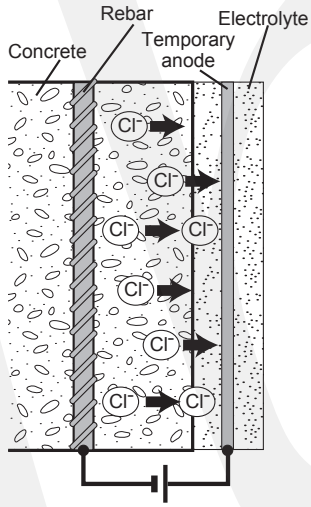



Fig. 7-1 Illustration of chloride extraction/desalination.

*Comments:*

*At the reinforcement (= cathode), the following reaction takes place:*

$$2H_2O + O_2 + 4e- \rightarrow 4OH- \quad (7\text{-}1)$$

*If a sufficient amount of oxygen is not supplied, the following alternative reaction takes place:*

$$2H_2O+2e- \rightarrow 2OH- + H_2 \quad (7\text{-}2)$$

*The hydroxide ion migrates into the surrounding concrete, and the hydrogen produced needs to escape through the concrete cover; this is why the current density is limited; too high current density will harm (crack, spall) the concrete.*

---

This chapter was written by Andrade, Bouteiller Habuchi, Hallopeau, Kobayashi (main author), Miyaguchi, Paula, and Ueda.

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## 7.2 Basics

### 7.2.1 When to adopt this method

Chloride extraction is used for structures that are suffering chloride attack. This treatment prevents further progress of rebar corrosion and an outbreak of new corrosion by removing chloride.

The aim of this treatment is to extract the chloride ions from the concrete cover and therefore to decrease or stop the corrosion.

If a large amount of chloride has penetrated beyond the rebar, other repair techniques shall be employed instead of, or in addition to, the chloride extraction.

Special care needs to be taken when applying the chloride extraction method to Prestressed Concrete (PC) structures.

ASR-damaged concrete also needs special care.

*Comments:*

*Chloride that has penetrated deeper than the rebar (when the rebars are arranged in multiple layers, "the chloride that has penetrated deeper than the deepest rebar") cannot be removed much by desalination treatment. Thus, other methods, such as permanent cathodic protection, need to be employed if the structure contains a large amount of chloride beyond the rebar.*

*Surface coating shall be provided after the chloride extraction treatment in coastal environments and for surfaces where de-icing salt has been used, in order to prevent re-ingress of chloride into concrete.*

*The possibility of hydrogen embrittlement needs to be taken into consideration for Prestressed Concrete (PC) structures, i.e., with prestressed steel tendons. To avoid this, current shall be applied intermittently for PC structures (an example was considered as Case Study in this Chapter).*

*When applied to concretes containing reactive aggregates, care shall be taken as regards to the accumulation of alkali around the rebar that acts as cathode. In such cases, lithium based electrolytes are often used to control expansion of reactive aggregate.*

### 7.2.2 Materials and systems

The system for applying direct current between rebar inside a structure and an anode metal consists of the following members:

- Reinforcement in structure;
- Electrolyte Solution

  Calcium hydroxide ($Ca(OH)_2$), lithium hydroxide (LiOH), lithium carbonate ($Li_2CO_3$), etc;
- Anode metal

  Mixed metal oxide-coated titanium, noble metal-coated alloy, etc;
- Electrolyte retainer

  Used for retaining the electrolyte solution on a concrete surface during the treatment;
- DC power supply;
- Monitoring system.

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*Comments:*

*It is essential to make the electrolyte solution saturate sufficiently between the anode on the concrete surface and the rebar, so as to 1) lower the electric resistivity of concrete for increased circulation of direct current, and 2) to increase the mobility of chloride ions in the electric field and 3) to transfer chloride ions from concrete to the electrolyte solution.*

*In principle any electrolyte can be used as long as it is not deleterious to the concrete or environment. In practice a saturated solution of calcium hydroxide is most widely used with excess solid to maintain alkalinity and avoid acid attack. Lithium based salts are used to suppress the formation of expansive gel if aggregates susceptible to ASR are present.*

*Anode metal suffers electrolytic corrosion during the period of the repair work. Therefore, metals that are resistant against electrolytic corrosion such as titanium shall be used as the anode. If carbon steel is used as the anode, it shall have a sufficiently large cross section.*

*The following are examples of electrolyte retainer.*

- *Cellulose fibre layer (Fig. 7-2): Sprayed on the concrete surface to cover the anode metal. Applicable on complex structure surfaces. The drawback is easy escape of electrolyte solution through evaporation and leakage.*



Fig. 7-2 Electrolyte retainer - Cellulose fibre layer.

- *Panel (Fig. 7-3) : Set on the concrete surface to form a closed space filled with the electrolyte solution. While capable of retaining electrolyte solution well, panels can hardly be adopted for complex surfaces.*



Fig. 7-3 Electrolyte retainer - Panel.

- *Ponding: Impounding of electrolyte solution on a concrete surface can be achieved simply, although only applicable on slabs. Also, electrolyte retention is poor.*

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## 7.3 Stakeholders' roles and qualifications

### 7.3.1 Owner

The owner must determine the following prior to the actual repair:

- Desired remaining service life of the structure;
- Performance requirements of the structure; and
- Expected effectiveness of the repair.

The owner must share the design plans and construction records of the original structure with the designer and the contractor.

### 7.3.2 Designer

The designer must be an expert of the desalination technique and shall preferably be qualified or certified.

### 7.3.3 Contractor

The contractor must be an expert of the desalination technique and shall preferably be qualified or certified.

*Comments:*

*ISO 15257:2017 "Cathodic protection - Competence levels of cathodic protection persons - Basis for a certification scheme" can be referred.*

## 7.4 Design

### 7.4.1 Assessment of existing conditions

The target structure must be examined closely to assess the current conditions before designing the repair system.

- Conditions of concrete cover - The width and length of cracks, loss of cross section and spalling of concrete, the shape of the concrete member, and presence/absence of the surface coating;
- The rebar arrangement;
- Chloride distribution - In-depth profile of chloride penetration from the surface to the backside of the rebar;
- Environment - Estimation of chloride supply from the environment after the repair.

*Comments:*

*For proper installation of the anode system on the concrete surface, the shape of the concrete member, conditions such as cracks and chips in concrete must be investigated beforehand.*

*The rebar arrangement must be specified for the circuit design, such as connection points between the rebar and lead wires, the number of connections, the DC power supply capacity, the number of power supplies, and so on.*

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*In addition, the presence of prestressed tendons must be checked.*

*Generally, only penetrated chloride can be removed effectively; mixed-in chloride is hard to remove and the remaining chloride content is generally too high to stop ongoing corrosion.*

### 7.4.2 Service life

The service life of the repair ends when the rebar has corroded again due to accumulation of chloride around the rebar.

*Comments:*

*The service life of the repair can be estimated by calculation of the chloride diffusion (see 7.4.4.1).*

*Also, the deterioration of concrete repair causes rebar corrosion and the end of service life.*

### 7.4.3 Codes and standards and Other relevant references

- EN 14038-2 Electrochemical Realkalization and Chloride Extraction Treatments for Reinforced Concrete. Part 2: Chloride Extraction.
- JSCE Concrete library 157:2020, Guideline for Electrochemical Corrosion Control Method
- NACE standard SP0107-2017
- NACE report 01101 Electrochemical Chloride Extraction from Steel Reinforced Concrete – A State-of-the-Art Report 7.4.4

### 7.4.4 Design procedure

#### 7.4.4.1 Setting a target chloride amount/density

A target chloride amount/density at the rebar is set.

#### 7.4.4.2 Arrangement of anode

1) Anode metal layout, and 2) type of electrolyte retainer (see 7.2.2) are selected based on the shape of the structure, the shape of the concrete surface, and the surrounding environment.

*Comments:*

*The surrounding environment here includes the tolerability to electrolyte leakage, potential risks for the third party, and whether the target structure is located indoors or outdoors.*

*Chloride that was left in concrete and that will ingress into concrete after the treatment transfers mainly by diffusion. Expected changes of chloride distribution with time after the repair throughout the remaining service life shall be estimated by calculating re-diffusion of chloride at the time of planning the repair.*

#### 7.4.4.3 Current density and period

Generally, an electric current of 1A is applied per 1 $m^2$ steel surface or 1A per 1 $m^2$ concrete surface. The current density shall be determined for each application based on the rebar amount, imperviousness of concrete, cover depth, and chloride density.

Current is generally applied for a period of 8 weeks. The current application period shall be determined for each case based on the current density, chloride density around the rebar, and chloride extraction efficiency.

The electric circuit shall be designed based on the imperviousness of concrete, cover depth, DC power supply capacity, and lead wire capacity.

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*Comments:*

*When applying a current per concrete surface area, the amount of rebar inside shall be taken into account. At least a current of 0.5A per 1m$^2$ concrete surface is necessary for the electrophoresis of chloride ions in concrete. On the other hand, application of 10A per 1m$^2$ concrete surface or more can cause significant cracks within about 10 days. The total current shall be calculated and given in the report.*

*Care must be taken when there are large differences in the cover depth and the concrete imperviousness/ resistance since uneven current distribution will result in variations in the chloride extraction effect. The electric circuit must be divided if such is this case.*

#### 7.4.4.4 DC power supply

The DC power supply must have a sufficient capacity for supplying the necessary amount of current.

*Comments:*

*Any physical contact with the concrete surface, lead wire, exposed metal, and so on during the process will cause electrical shock. For safety reasons, the voltage applied to the system shall be less than 50V.*

#### 7.4.4.5 Pre-treatment

Damaged concrete and reinforcement must be repaired before the chloride extraction process.

The material for patch repair shall have a larger strength than that of the substrate concrete. Crack sealers must be able to close the cracks.

*Comments:*

*All the spalling, falling, cracks and other defects must be repaired beforehand to ensure that current will flow between the anode metal and the rebar. While compressive strength of repair materials is not required for the chloride extraction process, the repair materials shall have a necessary strength. Polymeric concretes shall not be used, as its electric resistivity is too high.*

#### 7.4.4.6 Post-treatment

When the structure is exposed to a chloride-rich environment, the necessity of applying surface coating for longer service life of the repair shall be considered.

*Comments:*

*The necessity to provide surface coating shall also be considered at the design stage if it is expected that the structure will be exposed to chloride and that the chloride amount around the rebar may increase during the remaining service life of the structure. Coating material shall be selected appropriately by considering the high humidity and high alkalinity (because of the electrolyte) of the concrete surface immediately after the desalination process.*

### 7.4.5 Supporting documents

Along with records of design details, a maintenance plan shall be prepared.

*Comments:*

*The maintenance plan must be prepared beforehand. After the repair, the anode system is dismounted and the structure is restored to its original state. The concrete structure can therefore be maintained in the same manner as that for ordinary concrete structures. It shall be mentioned if any of patching, crack sealing, and surface coating has also been carried out (also their locations, extent, etc.), and any particularities that need attention shall be specified in the maintenance plan.*

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## 7.5 Execution

### 7.5.1 Preparatory works

Confirm the location and area of the treatment on site. Check if the temperature is higher than the freezing point of the electrolyte.

Carry out the following preparatory works (see 7.4.4.5):

- Removal of deteriorated concrete and surface coating;
- Crack sealing;
- Patching;
- Restoration of reinforcement;
- Removal of metal parts in cover concrete (bolt, separator, etc).

*Comments:*

*Electrically insulating surface coating must be removed beforehand. Gypsum and plaster are conductive but dissolve in the electrolyte.*

*Exposed and embedded metal parts can cause short-circuiting and must be removed by using a rebar detector or similar equipment.*

*Metal objects within the execution area, such as guardrails, water pipes, and so on, shall be either 1) temporarily welded to the rebar, 2) temporarily removed, or 3) separated from the anode by more than 20cm to avoid electrolytic corrosion.*

### 7.5.2 Execution procedure

#### 7.5.2.1 Connection of rebar to DC power

Rebar is electrically connected to a DC power supply.

*Comments:*

*Exact locations of rebar shall be determined by using a non- destructive detector so as to minimize unnecessary damage to concrete by drilling. An appropriate number of connection points between lead wires and the rebar shall be determined based on the current capacity of the lead wire and the diameter of the rebar. The electric current through the wire will be excessive if there are too few lead wires. On the other hand, too many lead wires will require unnecessary drilling.*

#### 7.5.2.2 Installation of anode and supply of electrolyte

The anode metal is set in accordance with the prepared electric circuit design. The anode shall be set over the entire area of the chloride extraction treatment. An electrolyte retainer (see 7.2.2) is set over the anode and the electrolyte solution is supplied. The amount of electrolyte is determined in consideration of expected loss caused by absorption into concrete and evaporation.

*Comments:*

*Since the reinforcement steel bars are electrically connected to each other, the circuit cannot be divided on the cathode side. Any necessary circuit division shall be achieved on the anode side.*

*The anode metal is set several centimetres apart from the concrete surface. A lattice spacing of anode mesh of 20cm or less has been known to be effective in making the desalination uniform.*

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#### 7.5.2.3 Settlement of DC power

The DC power supply is installed.

#### 7.5.2.4 Application of direct current

Current is applied with a constant current control.

### 7.5.3 Finishing

After finishing current application, the entire anode system is dismantled. Drilled holes and core boring holes are filled.

## 7.6 Quality control

### 7.6.1 Quality control of materials

The materials used in the process such as the electrolyte, patching material, crack fillers, cables, etc, must meet quality standards.

### 7.6.2 Quality control before intervention

Check if all the steel bars are electrically connected to each other in each electric circuit. Electric conduction must be secured by an appropriate measure if any defect is detected.

*Comments:*

*All rebars must be electrically connected to each other in each electric circuit to achieve a uniform electric current flow through the rebar.*

### 7.6.3 Quality control during intervention

After starting the current application, the direction and magnitude of the current flow shall be monitored.

The electrolyte solution shall be replenished to keep the concrete surface wet throughout the current application. The alkalinity of the concrete surface must be kept over pH10. The DC power supply, anode metal, electrolyte, and lead wires shall be checked regularly.

Special care must be taken in handling hazardous electrolyte solutions.

*Comments:*

*The electric current will vary to some extent since the cover depth, amount and arrangement of rebar in the RC member are not uniform. Current variations within ±30% are acceptable. Any deviation from this range can indicate a connection failure between a lead wire and the rebar, or an electrical short circuit between the anode and the rebar. A voltage higher than a pre-set value may be an indication of a disconnection in the wire or a lack of electrolyte. Current and voltage shall be checked not only at the start of current application but also after the electrolyte has penetrated well into the concrete.*

*The concrete surface shall be checked if it is wet with the electrolyte at least once a week during the desalination process. The alkalinity shall also be checked frequently so that it does not fall below pH10 to avoid generation of chlorine gas.*

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### 7.6.4 Quality control after intervention

Chloride distribution must be investigated.

*Comments:*

*The effects of chloride extraction can be confirmed by comparing the chloride distributions before and after the execution of treatment.*

## 7.7 Monitoring and maintenance

### 7.7.1 Monitoring

The structure that underwent desalination must be regularly inspected. In addition to routine visual inspections, the state of steel corrosion shall be examined with an electrochemical method such as a half-cell potential method etc. every few years. Preferably, the chloride distribution shall be investigated every several to 10 years.

*Comments:*

*After the desalination treatment, the amount of chloride around the steel is reduced and the passive film will likely regenerate on the rebar surface. For such a structure, an ordinary periodical inspection for chloride-exposed structures may be carried out.*

*In periodic visual inspections, attention shall be paid as to 1) whether there are new cracks or corrosion stains on the concrete surface, and 2) whether there is observed new spalling or falling in the cover concrete.*

*The half-cell potential of the rebar is substantially lowered immediately after the desalination because of being cathodically polarized by a large current. It can take from several months to one year until the potential returns to its intrinsic value that reflects the rebar passivity. A reference electrode embedded near the rebar may be useful.*

### 7.7.2 Maintenance

Surface coating may be replaced regularly (optional). See Chapter 2.

*Comments:*

*See 7.4.4.6.*

### 7.7.3 Post-intervention documentation

The contents of design, results of execution, quality controls, and maintenance procedures shall be appropriately recorded and kept.

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## 7.8 Case study 1

| | |
|---|---|
| Project Title: | Secondary School at Bombarral, Leiria, Portugal |
| Location: | Bombarral, Leiria, Portugal. |
| Client: | Parque Escolar, E.P. |
| Designer: | Zetacorr, Lda. |
| Contractor: | STAP, S.A. |

### 7.8.1 Project Details

Rehabilitation of a school building (Fig. 7-4), with reinforced concrete structure, in part severely affected by rebar corrosion due to penetrated chloride contamination (Fig. 7-5). As part of the rehabilitation project, the desalination technique was implemented, with the objective of reducing to acceptable levels the chloride content of the concrete adjacent to the reinforcement bars, thereby eliminating the risk of corrosion. Resorting to this technique enabled the integral rehabilitation of the affected part of the reinforced concrete structure which was otherwise pending demolition and replacement.



Fig. 7-4 Secondary school at Bombarral, Leiria, Portugal.



Fig. 7-5 Surface degradation of pavement associated to corrosion of slab reinforcement.

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Implementation of the electrochemical desalination technique consists of the temporary application of a direct current between the concrete reinforcement (cathode) and the grid of an anode, attached to the surface of the concrete member, embedded in an electrolytic solution. Due to the passage of the direct current, the chloride ions (charged negatively) are attracted to the anode (charged positively) and migrate towards the exterior. Simultaneously, an electrolysis phenomenon occurs at the interface steel/concrete, creating an alkaline environment, leading to the repassivation of the reinforcement. Before starting the desalination process, all areas of deteriorated concrete were properly repaired. In some of the areas, due to severe cross section loss, reinforcement sections had to be reinstated through the replacement of corroded rebars with new ones. The treatment of the three blocks which make up the school building was carried out separately, the installation of the system involving the following steps:

- Preliminary tests, namely to check the reinforcement electric continuity, to evaluate the chloride content in control areas, etc;
- Application of the anode, consisting of a welded steel mesh between felt layers (Fig. 7-6);
- Establishment of anodic and cathodic (reinforcement) connections;
- Installation of a "watering" system to enable proper moisturizing of the anode supporting felt layers, ensuring uniform current distribution to all the concrete surface in each zone;
- Testing to ensure the absence of short-circuits between
- anode and cathode;
- Application of electric current (Fig. 7-7) and monitoring of system operation;
- Amperage, monitoring in each zone, in order to determine total charge (A.h);
- Core drilling in the control areas to check the concrete chloride content at reinforcement levels.



Fig. 7-6 Installation of the steel mesh (anode) between felt layers.

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The system was set to supply an average current density of 1 A/m$^2$ of steel reinforcement. The anodic system was divided, in each block, in several electrically independent zones, and each zone was fed by a different output of the feeding source, in order to ensure effective control of the current to all the concrete surface. Duration of treatment in each block (around 500 m$^2$ of concrete surface) varied between 4 and 7 weeks.

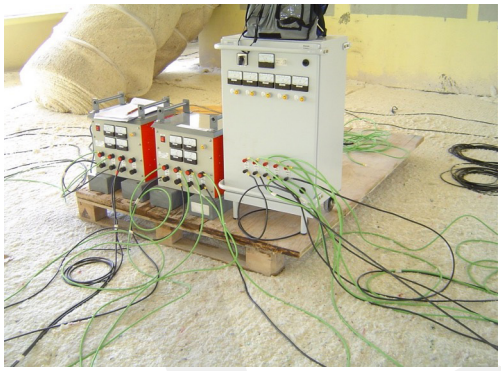


Fig. 7-7 Example of current feed sources with connections.

Criteria used to establish the treatment completion were based on the applicable standards (Fpr CEN/TS 14038-2, NACE SP0107-2007 and NACE Item 24214), namely:

- reduction of chloride concentration in the concrete next to the reinforcement to values below 0.4% of cement weight;
- minimum 600 A.h/m2 of steel of supplied electrical charge during treatment.

Results of chloride content in cores extracted before and during treatment are presented in Fig. 7-8, 7-9, and 7-10 for Blocks A2, A3 and A4 respectively.

Due to the much higher chloride content in Block A2, duration of treatment was longer than for Blocks A3 and A4.

These results show the effectiveness of the desalination technique in removing the chloride ions at the reinforcement level in all treated blocks.

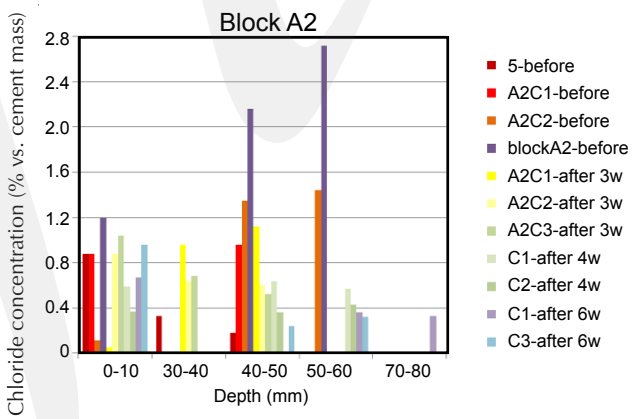



Fig. 7-8 Variation of the chloride content with depth before, during and after treatment, in Block A2.

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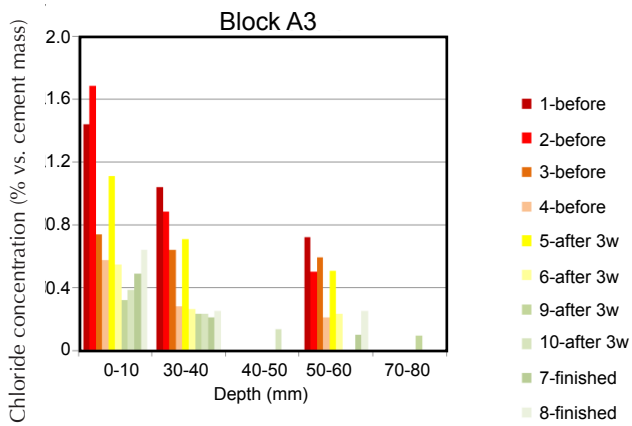



Fig. 7-9 Variation of the chloride content with depth before, during and after treatment, in Block A3.

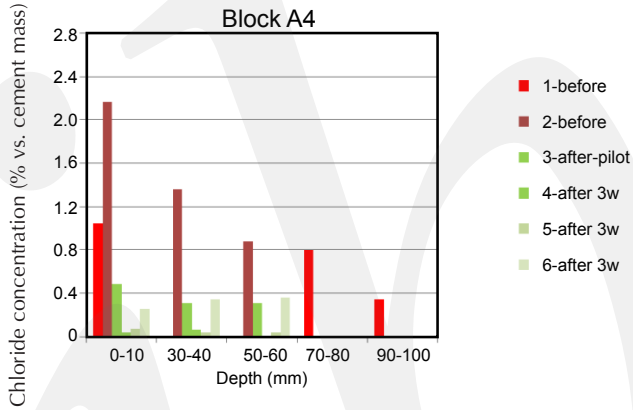



Fig. 7-10 Variation of the chloride content with depth before and after treatment, in Block A4.

## 7.9 Case study 2

| | |
|---|---|
| Protection/Repair/Strengthening Method: | Chloride Extraction/Desalination |
| Project Title: | Prestressed concrete road bridge |
| Location: | On a coast line of Sea of Japan |
| Client: | Central Nippon Expressway Co. Ltd, Japan |
| Designer: | Denka Co. Ltd, Japan |
| Contractor: | Denka Co. Ltd, Japan |

### 7.9.1 Description of the structure

A prestressed concrete bridge (Fig. 7-11) extending to 3,373 m as a stretch of an express way on the coast of the Sea of Japan. Built in the 1980s. Chloride extraction/ desalination was applied to its superstructure in 2003.

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Fig. 7-11 Prestressed concrete bridge.

### 7.9.2 Inspection highlights

The concrete surface did not show any serious damage, such as cracks, delamination, and spalling. However, the harsh environment has caused chloride penetration. In some parts, chloride amounted to about 2.0kg/m$^3$ (vs. unit concrete volume, roughly equivalent to 0.5% vs. unit cement mass) near the reinforcement steel.

### 7.9.3 Description of the intervention project

As preventive measures, 1) Surface lining and 2) Surface coating by polymer cement mortar (PCM) were applied to the structure to improve the durability. A large amount of lithium nitrite was mixed to PCM as a corrosion inhibitor. For the areas with a critical chloride content at rebar depth, these measures were assumed to be not effective enough, and therefore the chloride extraction method was adopted.

### 7.9.4 Description of the intervention works:

The areas to be treated with chloride extraction were determined based on the chloride distribution in the concrete. The diffusion coefficient of chloride was estimated for each area to predict future chloride distribution. Chloride extraction may cause alkali accumulation around the rebar and accelerate ASR. Therefore, the concrete was tested beforehand to confirm there was no ASR residual expansion potential.

Rebar was exposed at two or more locations per 30m$^2$ of concrete surface and lead wires were connected (Fig. 7-12). Titanium mesh was fixed on the concrete surface as an anode, and cellulose was sprayed (Figs 7-13 and 7-14).

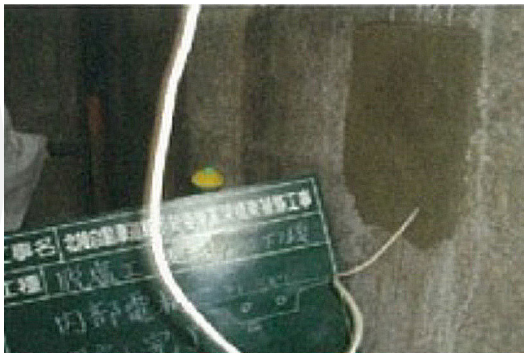


Fig. 7-12 Connection of lead wire to the reinforcement.

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Fig. 7-13 Titanium mesh set on the concrete surface.



Fig. 7-14 Cellulose fibre sprayed on the concrete surface.

Using a DC power supply with a capacity of 40V, a current was applied with a unit density of 1.5 A per 1 $m^2$ concrete surface, instead of the standard 1.0 A/$m^2$ for chloride extraction, considering the significant amount of chloride penetration. Current was applied intermittently, in a cycle of 5 days ON and 2 days OFF, to prevent hydrogen embrittlement failures of the PC tendon and deterioration of the bond between the tendon and the concrete.

The alkalinity of electrolyte solution, current density, and voltage were monitored routinely. The presence of short circuits and the dryness of the cellulose fibre layer were checked based on the voltage.

The chloride distribution was investigated after 30 and 40 days. The chloride amount around the rebar has dropped below the target value (1.2 kg/$m^3$) after 40 days. Since it was confirmed from the chloride distribution at this point that the chloride amount around the rebar after 100 years, as calculated based on the Fick's diffusion law, would be less than the corrosion threshold value (1.2 kg/$m^3$), the treatment was stopped. Chloride distributions before and after the chloride extraction are shown in Fig. 7-15.

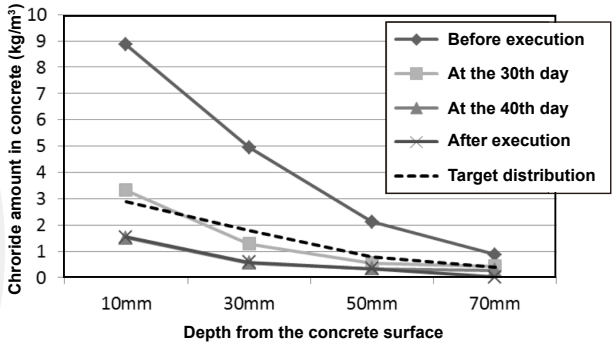



Fig. 7-15 Chloride distribution in cover concrete (2.0 kg/$m^3$ vs. unit concrete volume is roughly equivalent to 0.5% vs. unit cement mass).

To prevent re-penetration of chloride into concrete, a surface coating was applied.

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### 7.9.5 Description of maintenance operations:

The surface coating should be investigated periodically, as the robustness of the coating is a prerequisite to prevent re-penetration of chloride. After 10 years since the execution of chloride extraction, no change has been observed in the structure including the surface coating.

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# 8. Patch-repair (hand-applied mortar)

## 8.1 Foreword

Patch-repair by the hand-applied mortar method consists of restoring contaminated or faulty concrete of members in both minor and localised areas near the concrete surface with mortar that is hand- applied.

The commonly available generic repair material used in the patch- repair of concrete structures by the hand-applied mortar method consists of a polymer-modified hydraulic cement mortar, which exhibits low shrinkage, high thixotropy and high adhesive bond and includes an aggregate with appropriate size for the area to be repaired.

Patch-repair is a popular technique due to its simplicity and low cost.

## 8.2 Basics

### 8.2.1 When to adopt this method

The patch-repair of concrete structures by the hand-applied mortar method is one of the repair methods that can be employed to perform patch repairs. The method can be used in the case of shallow concrete replacements and thin overlays, on the top (horizontal), vertical and overhead surfaces (Fig. 8-1), in which the repair thickness is at least 6 mm and recommended not more than 30 mm, and the individual area to be repaired is around 0.5 $m^2$.



Fig. 8-1 Rehabilitation of service stairs using patch-repair by the hand-applied mortar method. Left side view of the stairs.

This chapter was written by Garcia, Monteiro, Ribeiro S (main author), and Ribeiro J.
https://doi.org/10.35789/fib.BULL.0102.Ch08

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The governing factor for determining if a hand-repair is feasible it is the volume of concrete that is cut away from loadbearing structural members. For larger areas or volumes, which may affect the structural capacity, other approaches are more effective for both technical and economic reasons.

*Comments:*

*See Fig. 1 "Common causes of defects and deterioration" from ISO 16311-3.*

It is a simple, rapid and economical repair method, which does not require specialised equipment and usually involves few restrictions to the conditions of application on site. It can be adopted to repair construction defects and those from on-going deterioration, mostly in atmospherically exposed concrete structures, both unreinforced and reinforced.

Patch-repair by the hand-applied mortar method is used to repair existing deterioration and may not necessarily extend the service life of the structure when reinforcement corrosion is involved, in the knowledge that it may lead to future additional repairs in the parts that were not previously repaired, as no signs of deterioration were visible.

It is the repair strategy to be adopted: i) for concrete restoration (aesthetic); ii) for increasing the physical and chemical resistance; and iii) for preserving or restoring passivity (protection of reinforcement), in cases where the load-carrying ability is not a primary concern. In these situations, the patch-repair by the hand- applied mortar method can be described as a non-structural or protective repair. In cases where the repair material is required to accommodate compressive stresses or when it exceeds the depth of the reinforcement, the patch-repair will be described and considered as a structural repair.

However, it must be pointed out that patch-repair by the hand- applied mortar method tends to be prone to problems deriving from the incomplete encapsulation of reinforcing bars with the repair material and from the poor bond strength between both.

It must be pointed out that, when patch-repair by the hand-applied mortar method to prevent or restore passivity (protection of reinforcement) is used to repair corrosion-induced damage, it may lead to initiate or accelerate the corrosion of steel bars that are no longer passivated, and of which the location, type, scope and emergence cannot be estimated beforehand. This phenomenon is also called “incipient anode effect”, “anodic ring effect”, or “halo effect”. Typical cases of initiation or acceleration of corrosion due to the incipient anode effect, may typically occur in the following cases:

Case 1) in areas adjacent to the patching repair, where steel bars embedded in the existing concrete are depassivated (e.g. around the interface, Fig. 8-2).

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Fig. 8-2 Active corrosion in dark areas adjacent to the patching repair (incipient anode effect). Tunnel side wall (Broomfield J et al, 2013).

Case 2) in areas of partially exposed reinforcing bar, where part of the circumference remains depassivated after patching and is in contact with the existing concrete (chloride-contaminated and /or carbonated concrete) (Fig. 8-3).

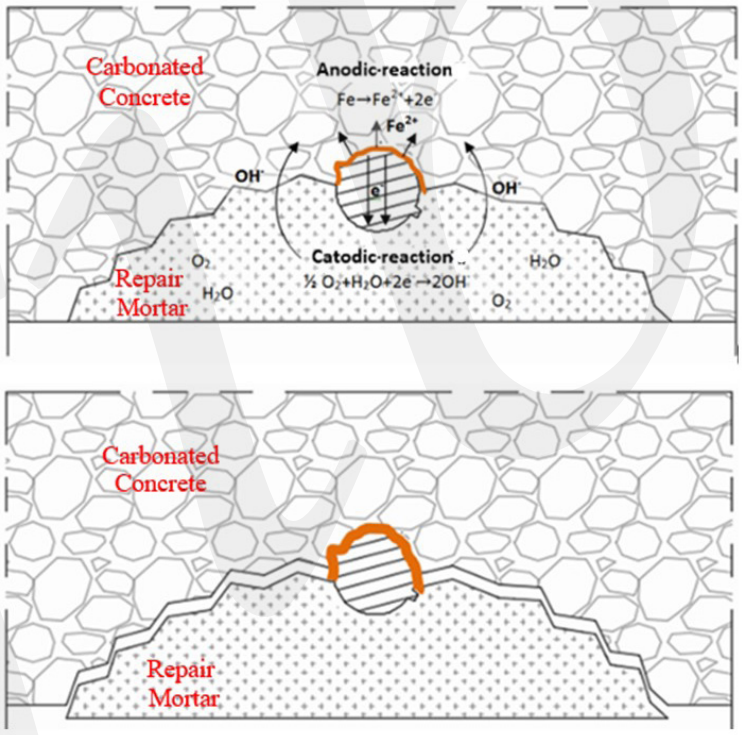



Fig. 8-3 Active corrosion in the back surface of the exposed steel bar that remains depassivated and in contact with the existing concrete after patching (incipient anode effect) (Monteiro A, 2018).

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Case 3) when steel bars or new strengthening bars, that are embedded in the patch-repair material, are electrically connected to depassivated steel bars in the opposite face of the member;

Case 4) when the barrier coating (e.g. epoxy-coatings) applied to the exposed steel bars is or remains damaged after patching, next to the existing contaminated concrete.

*Comments:*

*See ISO 16311-3, through Method 3.1 (Hand-applied, localized patches) of Remedy 3 (Concrete restoration), Method 5.3 (Adding mortar or concrete) of Remedy 5 (Increasing physical resistance), Method 6.3 (Adding mortar or concrete) of Remedy 6 (Resistance to chemicals) and Method 7.2 (Replacing contaminated or carbonated concrete) of Remedy 7 (Preserving or restoring passivity).*

*See ISO 16311-4, for execution of repairs and prevention.*

The incipient anode effect is a phenomenon that depends not only on the properties of the patching materials, the surface treatment of the steel and its electrical connections, but also on the exposure conditions of the existing concrete, as well as compatibility between hand-applied mortar and concrete substrate, on interfacial properties, in-service conditions, and mechanical loadings.

*Comments:*

*Report Nº. MERL-2014-87, Compatibility Issues in Design and Implementation of Concrete Repairs and Overlays. Project ID: 0385. Reclamation Managing Water in the West.*

The corrosion of reinforcing steel, in the concrete near a patch repair, due to the patch-repair method, called "incipient anode effect", "anodic ring effect" or "halo effect", is attributed to macrocell corrosion. Macrocell corrosion occurs when a formation of spatially separated anodes (previously "naturally" cathodic protected) and cathodes (previously the anodic site) is found; the prevailing understanding being that the electrochemical incompatibility (Report Nº. MERL-2014-87, Compatibility Issues in Design and Implementation of Concrete Repairs and Overlays. Project ID: 0385. Reclamation Managing Water in the West.) between the repair material and the existing concrete is the main causing agent (Raupach, M. 1996).

Another aspect that needs to be considered when employing patch repair by the hand-applied mortar method is that it may be visually obtrusive. Indeed, hand-repair of architectural concrete is complicated, but it generally works well once it is engineered for compatibility purposes. The repairs may have a different surface texture and may look differently from the rest of the structure.

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## 8.2.2 Materials and systems

### 8.2.2.1 General

The repair mortars for the hand-applied repair of concrete structures are currently available as proprietary materials (pre-formulated and pre-packed repair materials factory batched products). They are pre- batched materials, which have been developed for a precise application under specific environmental conditions. They are subject to routine factory production control by the manufacturer and to initial type-testing (performance tests and identification tests) to confirm the values declared by the manufacturer, including classes.

They have specific instructions as regards mixing, curing and procedures for application to the support surface and the producer supplies information of the essential characteristics related to the intended uses. For this reason, they are normally more expensive than site batched repair mortars.

Systems include two or more materials which are used together, or consecutively, to undertake repair of concrete structures (e.g. hydraulic mortar with a surface coating).

### 8.2.2.2 Hydraulic mortars (HM)

These are mortars based on a hydraulic binder with finely graded aggregates, which may include admixtures and additions, and when mixed with water, they are set by hydration reaction.

Hydraulic mortars can be applied for extensive patch repairs and for concrete cover, being recommended that the thickness does not exceed 30 mm. The maximum aggregate size should not exceed one-third of the repair thickness ($D \leq 10$ mm).

*Comments:*

*See section 3.5 from ISO 16311-4.*

### 8.2.2.3 Polymer modified hydraulic cement mortars (PHCM)

These are cement mortars with a new component – the polymer –, which is usually added to the mixing water just like any other chemical admixture, but at a dosage ranging from 10 % to 60 % by weight of cement (Ribeiro, S et al, 2008). Therefore, the designation of these polymeric products as admixtures extends the concept of admixture in accordance with ISO 22965-2. These polymers are used as partial or total replacement for the mixing water and do not affect the passivating alkalinity of cement hydraulic mortars.

*Comments:*

*See section 3.7 from ISO 16311-4.*

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Furthermore, PHCM mortars are available in a two pack: of a liquid and powder: one containing the polymer emulsion and the other the pre-blended sand, cement and additions. For redispersible polymer powder, all the components are pre-blended in one single pack (a powder bag). Before application, the constituents shall be mixed on site with water (or without, for higher dosages of polymer emulsion), according to the technical sheet of the PHCM mortars.

PHCM mortars are recommended in cases where the concrete cover is less than 25-30 mm but more than 12 mm.

The polymers in the hydraulic mortar have no adverse effect on the fire resistance of polymer hydraulic cement mortars due to their low dosage in the cement-paste. They also improve the bond between the hydraulic mortars and the existing concrete and reduce the permeability to water and to carbon dioxide.

*Comments:*

*The following polymers are used to modify the hydraulic mortars: watery polymer dispersions such as polymer emulsions (e.g. lattices being a stable milky white fluid) or redispersible polymer powders (e.g. styrene/vinyl acrylic redispersible polymer powders); water soluble polymers; liquid resins and monomers.*

*Most polymer emulsions are supplied with approximately 25% to 50% by weight of polymer solids. Different types of polymers have been used as modifiers for cement mortars, which have been specially manufactured for this purpose. These include acrylic, methacrylate or modified resins as redispersible powders or aqueous dispersions; vinyl mono-, co- and ter-polymers as redispersible powders or aqueous dispersions; styrene butadiene co-polymer, generally as aqueous dispersions; natural rubber lattices; and epoxies.*

*Tests performed on PHCM confirmed the beneficial action of polymer modifiers on the tensile strength and, in particular, on the flexural strength of cement hydraulic mortars, whereas in compressive strength, a reduction was observed at earlier ages (According to Ribeiro, S et al, 2008). It is considered that this mechanical behaviour results from the polymer action on the atrophy of hydrated $Ca(OH)_2$ crystals and on the reduction of microcracks density in the paste-aggregate interface, which leads to a noticeable improvement in flexural and tensile strengths. On the other hand, the delay in cement hydration and the increase in the closed porosity of PHCM are the main causes for the reduction in the compressive strength, in particular during the first months. The PHCM mortar shrinkage has a tendency to decrease with the increasing content in polymer modifiers, being hence designated as a "low shrinkage" (see Table 3 of EN 1504-3, 2015) repair material. At certain polymer modifier contents, it may act as a shrinkage reducing admixture (SRA), by decreasing the surface tension of the interstitial solution (surface tension of water) and increasing the contact angle when the polymeric particles are deposited on the surface of pores. Tests performed on mortars have demonstrated that polymer modifiers can also be used as reducers of the modulus of elasticity for cement-based materials. The increase in polymer modifier contents decreases more significantly the modulus of elasticity of PHCM mortars than the compressive strength, unlike what happens with cement materials.*

*Values for the laboratory performance of the repair mortars are given in EN 1504-3.*

### 8.2.2.4 Polymer mortars (PM)

These are mortars based on a polymer binder with finely graded aggregates which set by polymerization reaction. Polymer binders generally consist of two components, a reactive polymer base and a hardener or catalyst, which are cured at ambient temperature.

Polymer mortars shall be used for small-volume repairs and thinner layers or for low concrete cover (6 mm).

Polymer mortars are more expensive than polymer modified cement mortars and cement mortars, and shall be adopted only in special cases, such as chemical attack (the efficiency shall be demonstrate depending the type of chemical attack).

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*Comments:*

*Typical binders are, for example, epoxies, unsaturated polyesters, cross-linkable acrylics and one or two-component polyurethanes. Additives may also be added to give specific properties such as plasticizers, flexibilizers, accelerators, retarders, materials that regulate the rheology, as well as pigments and fillers.*

*General polymer mortars have higher coefficients of thermal expansion and less resistance to fire or high temperatures than cementitious materials. In cases where chlorides in the original concrete are a significant factor, polymer mortars may assist in limiting the migration of chlorides back into the patch repair from the original concrete.*

*See section 3.8 from ISO 16311-4.*

*Compatibility properties*

*Shrinkage of patch-repair by the hand-applied mortar shall be minor for not leading to cracking and debonding, since usually most of the shrinkage of the substrate concrete has already developed. The creep of the hand-applied mortar may reduce the tendency to cracking by stress relaxation. Values for the laboratory performance shall be checked by trial systems.*

*Site values of the adhesive bond of the repair material do not need to be greater than the surface tensile strength of the substrate. Values for the laboratory performance shall be checked by trial systems.*

*In a patch-repair by the hand-applied mortar method, the adhesive bond between the reinforcing steel and the surrounding hand applied mortar may be weakened by the difficulty in properly compacting the hand-applied mortar and by the presence of voids formed behind the exposed bar, both resulting from the consistency of the repair mortars.*

*Compatibility for a non-structural (protective) repair shall be achieved by arrangement of dimensions and properties of the materials (e.g., shrinkage, thermal compatibility) similar to that of the existing concrete, by ensuring that the interface adhesion bond strength is not exceeded.*

*In addition, the compatibility for a structural repair shall consider the properties compressive strength and modulus of elasticity (being those of the patch-repair by the hand-applied mortar not less than those of the substrate concrete), so as to ensure that the repair is able to carry the design loads but to prevent risk of creating stronger "contact" localized areas, allowing for any changes that may take place during the service-life of the structure.*

*Materials and systems to be used shall be selected in accordance with requirements given in international standards; regional standards; national standards; national technical approvals; project specifications.*

#### 8.2.2.5 Bonding agents

A bonding agent is a repair system used to promote the adhesion of the hand-applied mortar (a new and fresh material) to the concrete substrate (an old and hardened material), for enhancing or achieving a permanent bond.

*Comments:*

*See section 3.1.7 and 3.1.8 from EN 1504-1.*

For HM, a cement-based bonding agent (Portland cement or a blend of Portland cement and fine aggregate filler with a mixture proportion of approximately one to one by mass) is usually applied to the prepared surfaces.

For PHCM, a polymer-cement slurry is normally used as bonding agent (with the same polymer emulsions as the PCM).

For polymer mortars (PM), a polymer of a similar nature to that forming the binder in polymer mortars can be applied to the dry surface (e.g. epoxy-resins) as bonding agent.

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*Comments:*

*Critical aspects that may compromise the effectiveness of using polymeric bonding agents are highlighted:*

- *the use of polymeric bonding agents may reduce the bond between existing concrete and the repair mortar if the bond coat sets before the application of the subsequent repair mortar;*
- *surface preparation indicated on the product data sheet must be followed;*
- *reduction of bond between existing concrete and repair mortar in fire situations and under high and negative temperatures;*
- *the durability of polymeric bonding agents is usually unknown for this type of application.*

### 8.2.2.6 Protective coatings of concrete surfaces

Protective coatings of concrete surfaces are repair systems used in cases where the performance of patch-repair by the hand-applied mortar is not suitable to prevent or reduce further deterioration, such as:

1. insufficient protection against chloride or $CO_2$ penetration, or both;
2. the minimum concrete cover required for durability purposes cannot be achieved;
3. the control of anodic areas is not achieved (Fig. 8-2); or,
4. the formation of incipient anodes, in areas of the exposed reinforcing bar where part of the circumference remains depassivated after patch-repair and is in contact with chloride-contaminated and /or carbonated concrete (see Case 3, section 8.2.1, Fig. 8-3).

It will be necessary to specify and implement a surface protection system to be applied in addition to the hand applied mortar, according to chapter 2 "Coating of concrete surfaces" of this guide.

The performance of surface protective coatings will depend, not only on their formulation, but also on the adhesion to the patch-repair by the hand-applied mortar. In addition, it shall also satisfy the aesthetic requirements and the service life of the repair.

*Comments:*

*See EN 1504-2 for performance requirements of surface protection systems for concrete.*

### 8.2.2.7 Active coatings of exposed reinforcement

Active coatings of exposed reinforcement are repair systems used in cases where patch-repair by the hand-applied mortar is not suitable to prevent or reduce further deterioration, such as:

1. the insufficient protection against chloride or $CO_2$ penetration, or both,
2. when the minimum concrete cover required for durability purposes cannot be achieved.

Active coatings as a repair system of patch-repair by the hand- applied mortar method can be applied directly to reinforcement bars (uncoated or coated) where they are exposed, after cleaning.

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These coatings contain electrochemical active pigments, which may either act as anodic inhibitors or provide localized cathodic protection.

*Comments:*

*See EN 1504-7 for performance requirements of reinforcement corrosion protection*

*The repair of structures containing epoxy-coated steel reinforcement presents issues that differ from structures containing uncoated steel reinforcement. The coated steel should be treated to avoid creating corrosion cells that may result in accelerated corrosion of the reinforcement, according to ACI 364.3T-10.*

#### 8.2.2.8 Corrosion inhibitors

A corrosion inhibitor is a repair system used to extend the time to corrosion initiation or to reduce the corrosion rate of embedded steel bars in chloride contaminated concrete.

*Comments:*

*See chapter 4 Corrosion inhibitors for steel rebars.*

It can be applied to the steel bars where they are exposed, on the hardened patch-repair mortars surface (penetrating into the repair mortar to reach the steel bar surface), or it can be added to the patch- repair mortar composition, in order to delay the corrosion onset and reduce the corrosion rate by increasing the concrete resistivity.

Corrosion inhibitors can be used to mitigate the risk of initiation or acceleration of corrosion of steel bars no longer passivated in all typical cases of incipient anode effects (see section 8.2.1).

All corrosion inhibitors shall be tested on repair mortars before use and, when applied to the reinforcing bars, their effect on bonding shall also be considered.

*Comments:*

*See EN 1504-7 for performance requirements of reinforcement corrosion protection*

#### 8.2.2.9 Barrier re-coating of exposed reinforcement

Barrier re-coating of exposed reinforcement are repair systems used when barrier-coated steel bars (e.g. epoxy-coated steel bars) are exposed in a repair area. The steel bars shall be cleaned inside the repair area and re-coated with the same or similar barrier coating (e.g. epoxy), in particular for the cases referred to in section 8.2.2.7.

### 8.2.3 Equipment

The patch-repair of concrete structures by the hand-applied mortar method requires hand tools, such as wire brushes, cutting discs, drills, chipping hammers up to about 5 kg with common tips such as point, flat chisels and bushing head, spiral mixing paddles, hand mixers, trowel tools, brushes (Fig. 8-4 and 8-5).

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Fig. 8-4 Metal cleaning for surface preparation with a rotary wire brush used with power drills.

Due to the stiffness and the thixotropic character of hand applied mortars, the trowel technique requires considerable effort to compact them into localized patch repairs.



Fig. 8-5 Removing contaminated concrete using a chipping hammer.

## 8.3 Stakeholders' roles and qualifications

### 8.3.1 Owner

The Owner shall provide the designer with all the existing information related to the structure to be repaired, such as: original design project and as-built documentation, as well as any documentation referring to inspections and post-construction interventions.

It is the Owner´s responsibility to inform other parties on the possible environmental hazards and exposures (e.g. industrial structures with aggressive and/or biologically/ chemically active environment). Otherwise the proper selection of materials cannot be made.

An Owner´s brief shall be delivered to the Designer as a formal document in which the owner´s requirements will be written down.

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The Owner's criteria shall be based on: urgency; cost (time, budget and/or quality limitations); objective and performance of the repair; service life expected for the repaired structure; future service loads to be carried; architectural and aesthetic requirements (requirements for the appearance throughout the lifetime of the repair); service and environmental restrictions.

The Owner shall contract a supervisor to represent him in all the technical issues of the repair works.

The Owner shall accept the repair design proposed by the Designer (all Owner´s brief criteria has been addressed and approved by Supervisor).

The Owner shall set up a bid for Contractors and select the best technical and economical option in agreement with the Design.

### 8.3.2 Designer

The Designer shall identify and understand the reasons for the deterioration and shall determine its extent, based on a detailed inspection of the concrete structure.

Taking into account the inspection of the concrete structure (e.g. Report) and surroundings; the evaluation of specific (site-related) environmental effects; and, the Owner's criteria, the Designer shall prepare a full specification for the repair works, i.e., the execution specification, after thoroughly evaluating the structural and operational considerations of the selected repair method and materials.

Since neither the number of repairs, nor the area and depth of individual repairs can be established until close access is provided, then, the documentation shall include a mechanism for the inevitable changes in quantities that will occur as the work progresses. The Owner shall be made aware of the fact that the quantities and costs involved are uncertain.

The Designer shall provide the contractor with the execution specification and shall include the project specification according to ISO 22966 and with reference to the national provisions relevant in the place of use.

Instructions on inspection and maintenance to be undertaken during the remaining design service life of the repaired part of the concrete structure shall be provided to the Owner.

*Comments:*

*According to 3.8 of the EN 13670 - documents covering all drawings, technical data and requirements necessary for the execution of a particular project.*

### 8.3.3 Contractor

The Contractor shall have sufficient experience and expertise of repair works and shall be responsible for carrying out the repair in accordance with the contractual requirements, to follow the requirements of the project specification and use products with the specified characteristics, and present a report from an independent entity attesting those characteristics.

The Contractor's personnel shall have adequate knowledge and experience in the various tasks of the repair works (specific qualifications). It should be emphasized that the application of mortars in patch-repairs should be performed by experienced personnel in good rendering practices.

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### 8.3.4 User

The User shall respect the anticipated use of the structure and follow the maintenance plan, if made available.

## 8.4 Design

### 8.4.1 Assessment of existing conditions

The assessment of existing structures shall comply with the requirements and procedures established in ISO 16311-2 and in clause 4 of ISO 16311-3.

It shall include the framework for the assessment, the site and laboratory investigation and data collection, evaluation and verification, recommendations, and finally, the report.

Generally, a preliminary inspection will be carried out, possibly including some limited testing, with a view to characterise the nature of any problems and to define the Sampling Plan for the structure, which is to be carried out in the detailed inspection. Afterwards, a detailed inspection and a full survey, normally involving a testing program (in situ/ laboratory tests), will be performed to confirm the causes for the problem. In addition, some surveys will be carried out to assess the extent of the problem and the technical specifications for the repair work will then be included.

*Comments:*

*See ACI 364.1R-19*

*Special reference can be made to the following actions to be carried out to assess and characterise the relevant existing conditions:*

1. *Classifying the service and exposure conditions (e.g. height of the structure element; areas exposed to rain, washed-out areas, sheltered concrete; wind speed and orientations; any chemical aggressiveness) and their effects on carbonation and chloride penetration, considering the age of the structure;*
2. *Checking the environmental factors that may affect the progress of the planned repair work and defining the operational program to overcome these limitations; assessing the effect of repair works on the environment and defining the suitable means of protection that may be needed; as well as defining the suitable prevision for the presence of severe conditions to avoid interrupting the planned repair work;*

*See Fig. A.1 of ISO 16311-3, which gives the phases of a repair design:*

3. *Identifying the defects and their causes and assessing both the short and long term implications: assessing if the said defects are either a localized or a generalized problem, that occurs all over the structure, and determining if they have affected the aesthetics, the durability or the load-bearing capacity of the structural members;*
4. *Assessing the existing concrete quality;*
5. *Quantifying or estimating the extent of the necessary repair works.*

### 8.4.2 Service life

Patch-repair by the hand-applied mortar method shall ensure that no deterioration will occur in the repaired area, at least during the remaining lifetime of the structure.

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### 8.4.3 Codes and standards and other relevant references

- ISO 13822, 2010, Bases for design of structures- assessment of existing structures, International Standard.
- ISO 16311-1, 2014, Maintenance and repair of concrete structures - Part 1: General principles, International Standard.
- ISO 16311-2, 2014, Maintenance and repair of concrete structures - Part 2: Assessment of existing concrete structures, International Standard.
- ISO 16311-3, 2014, Maintenance and repair of concrete structures - Part 3: Design of repairs and prevention, International Standard.
- ISO 16311-4, 2014, Maintenance and repair of concrete structures - Part 4: Execution of repairs and prevention, International Standard.
- ISO /FDIS 22965-1, 2007, Concrete – Part 1: Methods of specifying and guidance for the specifier, International Standard.
- ISO /FDIS 22965-2, 2007, Concrete – Part 2: Specification of constituent materials, production of concrete and compliance of concrete, International Standard.
- ISO 22966, 2009, Execution of concrete structures, International Standard.
- EN 1504-10, 2017, Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 10: Site application of products and systems and quality control of the works, European standard.
- EN 1542, 1999, Products and systems for the protection and repair of concrete structures - Test methods - Measurement of bond strength by pull-off, European standard.
- EN 13395-4, 2002, Products and systems for the protection and repair of concrete structures. Test methods. Determination of workability. Application of repair mortar overhead, European standard.
- EN 13670, 2009, Execution of concrete structures, European standard.
- ACI 364.1R-07, Guide for Evaluation of Concrete Structures before Rehabilitation.
- ACI 364.3T-10, TechNote Treatment of exposed epoxy-coated reinforcement in repair, American Concrete Institute.
- ACI 364.6T-02 (11), Concrete Removal in Repairs Involving Corroded Reinforcing Steel, American Concrete Institute.
- *fib* Bulletin No. 51, Structural Concrete – *Textbook on Behaviour, design and performance*, Second edition, Volume 1, November 2009, 308 pp, ISBN 978-2-88394-041-3.
- ICRI Technical Guideline No. 310.IR-2008.
- Model Code 2010, Vol 1, final draft, *fib* – Bulletin 55, *fib*.
- Report Nº. MERL-2014-87, Compatibility Issues in Design and Implementation of Concrete Repairs and Overlays. Project ID: 0385. Reclamation Managing Water in the West.

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- Raupach, M. "Chloride-induced macrocell corrosion of steel in concrete-theoretical background and practical consequences." Construction and building materials 10.5 (1996): 329-338.
- Broomfield J, Fischer J; Mietz J, Schneck U "Case Studies". *Materials and Corrosion* 2013, 64, Nº. 2, DOI: 10.1002/maco.201206649.
- Ribeiro S, Gonçalves A., Branco F "Styrene-butadiene polymer action on compressive and tensile strengths of cement mortars". *Materials and Structures 2008*, 41, 1263-1273, DOI 10.1617/s11527-007-9324-z.

## 8.4.4 Design assumptions

Availability of a comprehensive design of the structure. Compliance with the general assumptions of section 8.2.1. of *fib* Model Code 2010.

When patch repair by hand-applied mortar is to be performed, it is important to consider the following design assumptions:

1. The existing concrete is considered to be contaminated when the acid-soluble chloride content by mass of cement exceeds 0.4% (determining or estimating the cement content of concrete often is required) or when it is carbonated;
2. The bond of exposed steel bars to hand applied-mortar is weakened due to the difficulty in properly compacting the hand- applied mortar and due to the presence of voids formed behind the bars (see the comments of section 8.2.2.4);
3. The patch repair contribution to the strength of the element depends on the elastic modulus, as well as on the shrinkage and creep of the hand-applied mortar, with reference to the existing concrete;
4. The volume of concrete that is cut away from loadbearing structural members may affect the structural capacity;
5. The repair mortar may need to be applied in several layers in order to build up the required profile;
6. Despite its capacity to restore the corrosion protection on the reinforcement steel or to preserve its passivity, this method may lead to new active corrosion in the repaired area, in the existing concrete or at the interface (incipient anode effect) and, hence, its location, type, scope and emergence cannot be estimated at present.

Assessment of the initial risk shall be stated and shall comply with clause 4.2 of ISO 16311-3.

*Comments:*

*According to EN 1504-9, traditionally, a figure of 0,4% of chlorides by weight of cement was used as the threshold above which reinforcement corrosion would occur. More recent research shows the figure can be much lower than this, sometimes below 0,2%, although in certain environmental conditions much higher values can be tolerated. Therefore, it is important to calibrate the risk of corrosion against the actual prevailing conditions of each structure and no "safe" limit should be assumed.*

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### 8.4.5 Design procedure

The design procedure can be initiated on basis of the repair strategy to be adopted by the patch-repair method, such as: i) restoring concrete (aesthetics); ii) increasing the physical and chemical resistance; iii) preserving or restoring passivity (protection of reinforcement).

The design procedure shall include the intended use of the repair mortar, the requirements for substrate, repair mortar and patching works, as well as the specifications for performance compliance, detailing and the safety appraisal after repair and prevention, in compliance with the requirements and procedures established in clauses 6, 7 and 9 of ISO 16311-3.

*Comments:*

*See Table 1 of EN 13670.*

Furthermore, it shall include the quality plan for the execution of the works; the acceptance of repairs; the remedial works; and the documentation, in compliance with clause 8 of ISO 16311-3 and ISO 16311-4.

#### 8.4.5.1 Detailing

The detailing drawings shall clearly illustrate: characteristics of the on-site system trial panels (e.g. number and location of tests); locations of the repair areas; instructions for removing deteriorated concrete; instructions for relocating existing reinforcing bars or for the application of new reinforcement; instructions for the application of the repair materials and systems; references to execution tolerances (if not specified in the drawings) and other applicable detailed specifications.

The configurations for removing damaged/deteriorated concrete shall be illustrated on drawings and shall be as simple as possible while minimizing the number of re-entrant corners, as they are susceptible to cracking, and shall preferably have square or rectangular shapes with square corners (Fig. 8-6). The range of angles permitted for the cut edges from which the concrete is to be removed (repair areas) shall be illustrated on drawings to reduce the possibility of debonding with the top surface of the adjacent sound concrete (Fig. 8-7).

*Comments:*

*According to fib Bulletin 65, Detailing consists of determining the dimensions of structural components and reinforcement layout and geometry in local areas of the structure and specifying the structural details*

*According to chapter 2, of Structural Design, Bulletin 1, Volume 1, Model Code 2010, Detailed design can be defined as the stage in the design process where all structure details are designed and calculated, such as detailed drawings for construction (e.g. working drawings – formwork drawing, reinforcement drawings) and all specifications for the required types and qualities of the building materials.*

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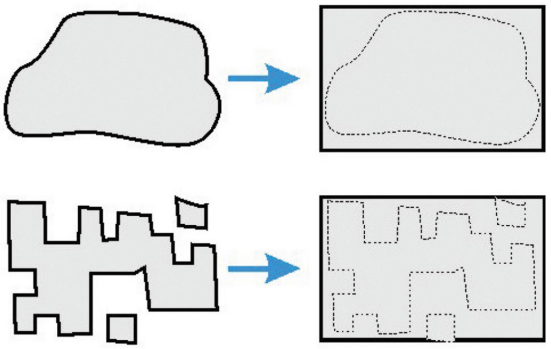


Fig. 8-6 Areas of deterioration and recommended removal configurations (ICRI Technical Guideline No. 310.IR - 2008).

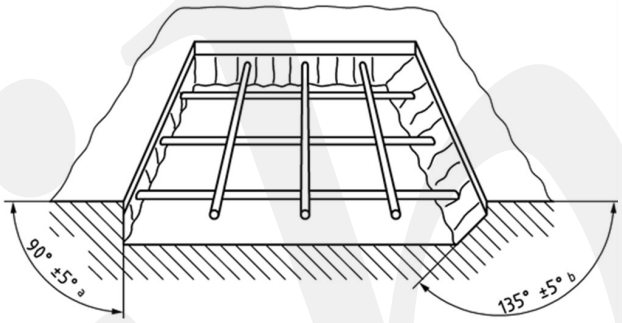



Fig. 8-7 Range of angles for the cut edges of repair areas (Fig. A. 1. of Annex A from ISO 16311-4).

The detailing drawings shall illustrate the instructions for relocating reinforcement bars and for application of new reinforcement, as well as the instructions for the application of repair materials and systems, according to chapter 4 "Internally applied steel reinforcement" of the *fib* Bulletin 103: *Guide for Strengthening of Concrete Structures*.

### 8.4.5.2 General specifications

All relevant national standards or provisions valid at the construction site shall be listed.

The execution class for the works as a whole or for an individual component shall be specified in accordance with ISO 22966. Patch- repair by the hand-applied mortar method is frequently classified as execution class one or two.

If necessary, a finishing method, specified to meet the architectural concrete requirements, must be included.

*Comments:*

*See Table 1 of EN 13670.*

### 8.4.5.3 Material specifications

Requirements for the use and approval of additions, admixtures, fibres, aggregates, cements, mixing water and water required for wetting the substrate shall be specified according to applicable reference documents (e.g. clause 5 of ISO /FDIS 22965-2).

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Maximum chloride content of the repair mortar, satisfying the maximum limits established in standards or provisions valid in the place of use, shall be specified. Values for laboratory performance are given in EN 1504-3.

Specific performance properties (colour, thermal compatibility, wear, freeze-thaw and sulphate resistance, creep in compression; bond strength; water permeability, accelerated carbonation resistance, maximum gas permeability, maximum chloride diffusion coefficient, minimum cement content, maximum water to cement ratio, minimum introduced air content, compatibility with cementitious materials, resistance to UV light) of repair mortars required for compliance with repair and prevention remedies shall be specified, in accordance with clause 6 of ISO /FDIS 22965-2; the corresponding actions in case of non-compliance shall also be specified.

Requirements for the storage of repair materials shall be specified or shall be in conformity with the manufacturer's requirements.

The bond (e.g. pull-out test) of the repair material with the substrate and the bond between the layers of the repair material shall be specified based on clause 6.3.4 of *fib* MC 2010.

Maximum shrinkage/expansion (or minimum bond strength under restrained shrinkage/expansion conditions) of the repair material shall be specified. Values for laboratory performance are given in EN 1504-3.

*Comments:*

*According to Table A.3 of EN 1504-10, the measured adhesion bond of repair materials depends on the mortar properties and on the surface tensile strength of the substrate, but can never be greater than the latter. Acceptable site values are within the range of 1.2 MPa to 1.5 MPa for structural repair and correspond to a minimum value of 0.7 MPa, for non-structural repair. Values for laboratory performance are given in EN 1504-3.*

*The conditions of application and limitations of the hand applied mortar shall be specified, as well as the final hardened condition.*

*It shall be specified whether the repair mortar is to be built up in layers to prevent sagging or slumping. The layer thickness and the time between the application of layers shall be specified and shall comply with the requirements defined by the manufacturer of the product.*

*In case of use of a proprietary repair material it shall be specified that the application of the repair material shall comply with the manufacturer´s requirements, and the basic requirements for the common use of the repair mortar shall be declared (written certification).*

*The compacted and curing conditions of the repair mortar shall be specified and shall comply with EN 13670 and ISO 22966. The curing class shall be specified.*

*The method and period of curing shall be specified by taking into account the nature of the repair material, the thickness of the repair and the environmental conditions. In case of use of curing compounds, it shall be specified that these must be removed before the application of a surface protective coating.*

*See Tables 3, 4 and B-1 of EN 1504-3 and EN 1504-2.*

*See clause 8.5 of EN 13670.*

*See EN 1542.*

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### 8.4.5.4 Preparation of substrate specifications

The depth of concrete removal and the volume of repair material necessary to meet the concrete cover requirements shall be specified. In cases where the concrete cover is not sufficient to fulfil the required service life, a coating may be specified in addition to the hand applied mortar (see section 2 Protective coatings of concrete surfaces), in accordance with Table 1 of ISO 16311-3.

If the patch area is to be aligned (regulated) with the existing surface, the depth of cut shall also be specified. It is recommended to not be less than the maximum aggregate size of the repair material.

Furthermore, the following aspects shall be specified: the length of removal [mm] of the cutting out of the existing concrete along the length of the exposed reinforcement and beyond the point at which the bar is visible; the location where the replacement steel is to be inserted; and the extent of additional cutting out to achieve an adequate overlapping of bars.

The technique for correcting the edges from which the concrete is to be removed shall be specified (Fig. 8-6 and 8-7).

If necessary, to mitigate any possible risk of incipient anode effect, an additional system for protection of the repaired areas shall be specified.

Clause A.6.3.2 of ISO 16311-4, for the clearance around the reinforcement and the minimum distance between the reinforcement bar (a), indicates at least 15 mm or the maximum aggregate size of the repair material plus 5 mm, whichever the greater, to allow proper compaction; and (b) 50 mm or more beyond the extent of the corrosion along the length of the bar (Fig. 8-8).

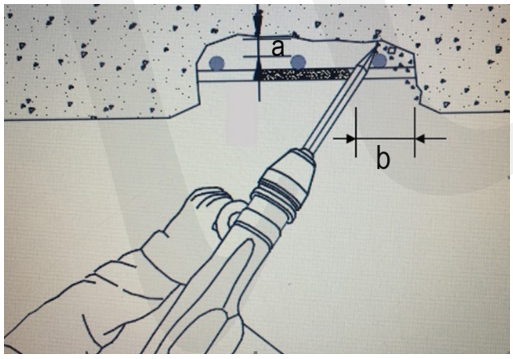



Fig. 8-8 Distances to be specified on concrete removal.

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The technique of concrete removal shall be specified in order to reduce the presence of a "bruised layer", i.e., a substrate with a degree of microcracking that reduces bond. This is best achieved by a light mechanical percussion hand tool with a maximum weight of 4 kg. As it is not possible by visual inspection of the substrate surface to check for the presence of microcracking and scale, the final layer with a depth of 10 mm (the "bruised layer") shall be removed, and according to ISO 16311-4, it shall be carried out by blasting methods (e.g. water blasting or grit blasting/shotblasting).

*Comments:*

*See table 4 of ISO 16311-4.*

After removing the "bruised layer", concrete surface roughness should be characterized and the target roughness, as well as the corresponding measurement method, shall also be specified. Currently it is sufficient to specify the technique used for roughening and the minimum removal depth.

After roughening, the condition of the existing concrete shall be specified. Where PC or PCM mortars are used without a bonding agent, the surface of existing concrete, after removal, shall be well pre-wetted but shall be free from water on the surface at the time of application. Where a bonding agent is used, the condition of the substrate shall be specified according to the Technical Sheet of the bonding agent.

#### 8.4.5.5 Preparation of reinforcement specifications

The preparation of reinforcement shall be specified in accordance with clause 7.3 of ISO 16311-4, which is considered in chapter 4 "Internally applied steel reinforcement" of the *fib* Bulletin 103: *Guide for Strengthening of Concrete Structures*. It shall include the specifications regarding the aspects as follows: the method to be used to measure the concrete cover; the cleaning procedure; the technique to measure the loss of section of the reinforcement bars due to corrosion; the conditions to add or replace bars; the length of the lap splice and type; and the number and diameter of the additional bars.

The maximum acceptable cross-section loss of the existing reinforcing bars, above which these must be replaced or strengthened, shall be specified.

#### 8.4.5.6 System trials specifications

Initial on-site trials or mock-ups shall be specified, with a view to check and optimise repair materials/systems and execution procedures, to test the compatibility between those materials and the existing concrete, and initial testing shall be performed for verifying if the required performance is achieved by the materials.

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Trial testing can be performed in the laboratory and on site, directly on the existing concrete or on trial panels applied to the structure, or also on mock-ups (in large repair works).

The approval for trial repairs shall be specified based on the following: sampling and acceptance criteria, testing ages, visual inspection, as well as colour and texture match with the existing concrete.

The following aspects shall be specified: the number and location of trial panels for each type of material to be performed and tested during the trial repairs; the Sampling Plan (method of sampling, the number of samples, their dimensions and the ages of taking samples/cores in each trial repair area); the age of testing; as well as the procedure to be followed to test the samples.

Trial panels/samples repairs shall be carried out using materials of the same formulation as that to be used in the repair works (placement, thicknesses, degree of restraint, compaction, curing, and exposure conditions). It shall be specified that if the formulation of materials is changed during the course of the works, additional trial sample repairs shall be carried out.

Trial sample repairs shall be chosen, as far as reasonably possible, to encompass the majority of positions and orientations of the surfaces to be repaired.

### 8.4.6 Supporting documents

The results of the entire inspection should be summarized in a final report. This report is described in Annex E of ISO 16311-2.

*Comments:*

*The final report is described to generally include a brief description of the following basic areas during the evaluation process:*

- *purpose and scope of investigation or appraisal;*
- *to whom the investigation or appraisal was performed (i.e. client) and who performed it (i.e. author(s) and their organization);*
- *existing construction and documentation;*
- *field observations and condition survey;*
- *sampling and material testing;*
- *evaluation;*
- *findings and recommendations.*

On completion of the repair and prevention design, the Design documentation requirements shall be provided to the owner of the structure. These documents are described in clause 8 of ISO 16311-3 and clause 10 of ISO 16311-4.

*Comments:*

*These documents generally include:*

- *documentation of the repair and prevention design, including any test result pertinent to the design;*
- *documentation of any quality control and assurance requirements for the execution of the repair and prevention design;*
- *instructions for inspection and maintenance to be undertaken during the remaining design service life of the repaired part of the concrete structure.*

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## 8.5 Execution

### 8.5.1 Preparation of existing concrete

- Identification and marking of reinforcement concrete areas with either exposed anomalies or to be detected by hammering.

  These areas shall have an even geometry with sections formed by rectangles, as per specifications of the Design drawings (Fig. 8-6).

- Identification of the reinforcement mesh and concrete cover.
- Removal of weak, damaged and contaminated concrete from around and along the exposed corroded or non-corroded reinforcement steel and, where necessary, as specified in the Project Specifications.

*Comments:*

*Special caution should be taken to locate and avoid cutting or damaging embedded reinforcing bars, prestressing strands, posttensioning tendons, or electrical conduits. Cutting into these items can be life threatening and may significantly affect structural integrity.*

- Correction of the edges from which the concrete is to be removed by cutting (e.g. sawn or disc cut), as specified in the Project Specifications.
- Visual inspection to reveal the presence of cracking on the surface substrate. Special attention must be paid to the area near the exposed reinforcement.

*Comments:*

*When required, cracks shall be sealed according to chapter 12 of this compendium.*

- Removal of the "bruise layer" (not less than 10 mm), where the presence of microcracking and scale cannot be visually detected, by blasting techniques to restore the integrity of the existing concrete after removal of deteriorated concrete.

*Comments:*

*The concrete surface shall achieve sufficient roughness to promote adhesive bonding and mechanical interlocking with the repair mortar.*

*After roughening, the cleaning of the substrate shall be carried out after concrete removal and correction of the edges of repair areas, and shall be performed immediately before application of repair mortars, if the cleaned substrate was not previously protected from further contamination.*

- Repair materials shall be stored in an environment in conformity with the manufacturer's requirements.

### 8.5.2 System trials

Initial on-site trials or mock-up repairs for approval shall be carried out as specified in the Project Specifications, not less than three weeks before the corresponding work is scheduled to start.

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Trial repairs shall be assessed by the Designer to check if the required standard is achieved by the materials and methods proposed by the Contractor. Trial repair areas shall be used as a standard against which subsequent work shall be judged.

### 8.5.3 Preparation of reinforcement

Preparation of reinforcement (cleaning of exposed reinforcement and removal and installation of reinforcement) as specified in the Project Specifications.

*Comments:*

*See section 7 from ISO 16311-4.*

Reinforcement protection shall only be used when specified by the designer, and the degree of cleaning of the reinforcement shall also be specified accordingly.

### 8.5.4 Application of repair materials

The general assumptions of section 4.1 of EN 13670 shall be satisfied.

The execution of repairs and preventive measures shall comply with the requirements and procedures indicated in clauses 6, 7 and 9 of ISO 16311-3 and ISO 16311-4. It shall include the choice for the products, systems, methods and equipment to be used; as well as the quality control and the health and safety procedures.

Surfaces shall be clean and sound prior to the use of the repair mortar.

The repair mortar shall be mixed according to the manufacturer´s recommendations.

The use of a forced-action mortar mixer is normally recommended to ensure thorough wetting of the ingredients and, hence, achieve a minimum water to cement ratio at the required consistency. Mixing times shall not be extended and excessively long mixing cycles shall also be avoided, because these may entrain excessive amounts of air (entrapment of air).

Special care must be taken when bonding agents are used, as these may reduce bond if they set or dry out much earlier than the application of the subsequent repair material. Bonding agents shall only be used when specified by the designer and the substrate conditions shall be specified accordingly.

Wetting of the repair surface shall be carried out as specified in the Project Specifications. Where concrete substrate is to be thoroughly pre-wetted, care must be taken to ensure that its surface is free from water at the time of application of the hand applied mortar.

Curing of the substrate surface shall be carried out as specified in the Project Specifications.

The application of the hand applied mortars shall be performed in a similar manner to renders and shall comply with ISO 22966. If overhead application is intended, care must be taken to ensure that the selected material is suitable for such application.

The repair mortar shall be worked into the prepared substrate and shall be compacted without introducing entrapped air pockets and in such a way that the required strength is achieved and the reinforcement is protected against corrosion.

The repair mortar shall be applied in layers not exceeding the specified thickness (in the design and by the manufacturer of the mortar).

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The mixed material shall be applied to the primed surface in layers without exceeding the manufacturer´s recommended thickness and using a technique to prevent entrapped air.

If subsequent layers are to be applied, the surface shall be thoroughly roughened to provide a good mechanical interlocking with the following layer. The time between layers shall be in accordance with the manufacturer's recommendations.

Intermediate stages and the final repair work shall be protected from rain for at least 24 hours after the repair has been completed. The repair shall be protected from direct sunlight during replacement and for at least 48 hours after completion.

It is also essential that the environmental condition (e.g. temperature, humidity) shall comply with the manufacturer's instructions.

### 8.5.5 Finishing and curing

The final layer of all repairs shall be finished to provide a close textured and accurately coloured level surface that matches the appearance of the surrounding concrete and which is in compliance with the standard achieved in the successful trial repairs (according to design specifications).

Care should be taken to avoid the formation of cracks, air pockets and segregation induced by finishing tools.

The final surface texture of the repair mortar shall be compatible with any surface protection coating or systems that may be applied.

Curing of CM and PCM shall comply with ISO 22966, which establishes curing classes. Wet curing (e.g. damp hessian) is necessary for CM and PCM, while a protection against evaporation, such as the use of a plastic sheet to prevent a rapid drying out, especially in hot drying and in cold dry windy conditions, is essential for polymer modified cement mortars and polymer mortars.

Greater care is required with multiple layers to prevent the occurrence of a rapid early drying out, which will interfere with the hydration of the cement.

*Comments:*

*Hydraulic mortars (HM) and polymer hydraulic cement mortars (PHCM):*

*Successive layers shall be applied as soon as the preceding coat has become sufficiently stiff to support the weight of the additional build-up layer but is still adequately tacky to provide bonding. The time between layers' application shall be in accordance with the time specified in the manufacturer´s recommendations. If sagging occurs, the material shall be completely removed and reapplied at reduced thickness.*

*In case the last layer applied completely dries out before over-laying, the surface shall be prepared according to the manufacturer´s recommendations and shall comply with 7.2.2, 7.2.3, and 8.2.1 of ISO 16311-4 and with 7.2.2, 7.2.3, 7.2.5 and 8.2.1. of EN 1504-10.*

*The final build-up layer within a repair shall not be less than 10 mm thick and shall be levelled off or profiled using trowelling techniques to produce a finish as specified.*

*Curing to retain moisture shall be implemented immediately upon completion of the repair. The surface of the mortar shall be protected from strong sunlight and drying winds.*

*The repair shall be cured according to the method and for the period recommended by the manufacturer of the repair system.*

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*Polymer mortar (PM):*

*Surfaces shall be dry unless a bonding agent specifically designed for damp surfaces is used.*

*The bonding agent shall be applied to the substrate by ensuring that it is well brushed into the prepared surface by taking care to avoid "puddling" in depressions. In cases where, due to the nature of the substrate, the bonding agent is absorbed by the concrete, a second application of a bonding agent shall be performed. The second coat shall be applied within 16 hours after the application of the first coat.*

*The polymer mortar shall be applied whilst the bonding agent has an adequate surface tack.*

## 8.6 Quality control

### 8.6.1 Quality control of Materials

The mix design and production of the repair mortar, as well as all the components, shall be subject to a quality control procedure in accordance with the national technical documents and standards valid in the place of use, to ensure that they meet the specified identification and performance requirements.

For the acceptance of the proprietary mortar (pre-formulated and pre-packed repair mortar), the marking, labelling or written certification shall comply with the material specifications approved by the designer.

*Comments:*

*See Annex ZA of series EN 1504.*

*The materials and intervention operations shall be inspected and documented as required for the execution class stated by the designer in the design specifications, in accordance with ISO 22966 and ISO 16311-4.*

*The quality control related to the application of surface protection systems on the repair material shall be in accordance with chapter 2 of this guide.*

*A specific quality control plan shall be provided by the designer, or developed by the contractor and approved by the designer, establishing tests or inspections, actions in case of non-conformity, acceptance criteria, in accordance with clauses 9.3 and A.5.4.19 of EN 1504-10.*

### 8.6.2 Quality control before intervention

The inspection procedures and on site tests to be conducted to check The Birth Certificate Document (BCD) shall include the results of an initial inspection of the repaired structure to meet/check the minimum requirement in construction, as specified in the execution standard (EN 13670) and as assumed in the design.

It shall include the direct input parameters for the future condition control of the repair structure, such as:

- Cover thickness of the reinforcement;
- Carbonation resistance of the repair mortar.

*Comments:*

*The inspection procedures and on site tests to be conducted to check the quality of the preparatory works shall comply with the requirements specified on table 5 of EN 1504-10: substrate conditions before preparation and condition and requirements before application.*

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### 8.6.3 Quality control during intervention

Before the application of the repair mortars, the following aspects shall be inspected and the frequencies of tests or inspections shall be in accordance with ISO 16311-4:

- Substrate conditions:
- Cracking;
- Roughness;
- Cleanliness;
- Temperature;
- Wet conditions, when using polymeric mortars.

Reinforcement conditions:

- Cleanliness;
- Measurement of the diameter of reinforcement bars.

Environmental and repair mortar conditions before and/or during application:

- Daily ambient temperature, and daily precipitation and wind;
- Consistency of the repair mortar;
- Segregation and/or bleeding of the repair mortar;
- Setting of bonding agents;
- Cleanliness of the repair areas;
- Dew point, when using polymeric mortars.

*Comments:*

*See EN 1504-10 guidelines for quality control procedures during repair steps.*

*See frequencies of tests or inspections of ISO 16311-4*

### 8.6.4 Quality control after intervention

After the application and hardening of the repair mortars, the following aspects shall be inspected and documented:

- Concrete cover;
- Shrinkage cracking of the repair mortar (Wetting the surface, and letting it dry, helps to visually detect cracks);
- Bond strength of the repair mortar;
- Colour and texture of finished surfaces.

*Comments:*

*See EN 1504-10 guidelines for quality control procedures during repair steps.*

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## 8.7 Monitoring and maintenance

### 8.7.1 Monitoring

A regular inspection of the structure by visual observation shall be carried out to detect any visible surface defects (e.g. cracks, spalling, colour changes) and to assess the formation of incipient anodes in the vicinity of repaired areas (possible acceleration of the reinforcement steel corrosion in the adjacent areas of the repair and in the face members opposite to the repair area). Potential mapping as a tool to assess the corrosion process at the reinforcing steel in the vicinity of repaired areas is recommended.

Monitoring shall also include a regular visual inspection of the structure to detect any inappropriate behaviour of the structure, such as efficiency and cleanliness of the drainage system and water leaks.

When surface protection products and systems are employed, it is necessary to account for their performance, particularly for polymeric products, due to their limited lifetime especially when exposed to sunlight (see chapter 2 in this Guide).

*Comments:*

*The procedures, data and results from tests shall be stated in the maintenance and monitoring report, as well as the actions carried out for critical conditions and defects.*

### 8.7.2 Maintenance

Upon completion of concrete repair works, a maintenance management system shall be implemented to ensure that the required future maintenance is carried out, according to clause 5 of ISO 16311-1.

Basic maintenance of this repair method shall include the following activities:

- Cleaning of the drainage systems;
- Cleaning of concrete surfaces and recoating;
- Repair operations identified in monitoring.

### 8.7.3 Post-intervention documentation

Monitoring and maintenance information shall be recorded and preserved by the owner or supervisor during the remaining service life of the structure, and shall be made available to those responsible for the quality management of the structure, including:

1. Documentation of the protection and repairs that were carried out, including information on all used materials and any test results;
2. Documentation of any quality control and assurance requirements completed during the execution of the monitoring and maintenance;
3. Instructions on inspection and maintenance to be undertaken during the remaining design service life of the concrete structure.

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## 8.8 Case study

| | |
|---|---|
| Project Title: | Rehabilitation of the façades and roofing of the buildings A to E of the High School José Gomes Ferreira |
| Location: | R. Prof. José Sebastião e Silva, Benfica, Lisbon, Portugal |
| Client: | DREL - Direção Regional da Educação de Lisboa |
| Designer: | Teixeira Trigo, Lda |
| Contractor: | BEL Betões - Engenharia e Reabilitação de Estruturas, S.A. |

### 8.8.1 Description of the structure

The construction of High School José Gomes Ferreira began in 1975 and its operation was initiated in 1980. Its architectural quality was recognized in 1982 with the awarding of a Honourable Mention from the Valmor Prize and from the International Association of Art Critics. The building consists of five elongated bodies of different lengths, like the five fingers of a hand, which are separated by uncovered corridors with stars, and each one ending in a cylindrical volume topped by terraces (Fig. 8-9). This configuration makes it possible to fully take advantage of the external spaces creating a harmonious ensemble with the large garden area that surrounds it.




Fig. 8-9 East view of the buildings before intervention. Location plan. 2003.

In addition, the simple geometry of pure forms contributes to attracting the ambient light and to provide the building with its own energy, in which each space finds its formal and functional vocation, in particular the classrooms, amphitheatres and the different areas that are distributed across the second and third floors.

The buildings are grouped by structurally independent bodies of square plan, with sides of 21.60 m, as for example, buildings “A” and “E” consist of a structural body; buildings “B” and “D” are formed by two bodies and building “C” integrates three bodies (Fig.8-9). In all the bodies that make up the school there is an apparent geometrical rigidity that is subverted inwardly by linear circulations, topped by the introduction of the diagonal, as well as by spiral stairs vertically bounded by sloping skylights. The number of floors was influenced by the natural slope of the ground. Buildings “A”, “B” and “D” have a partial

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basement, building "E" has a full basement and building "C", in the middle, has two basement floors with an accessible and partial flowery terrace cover at ground level. Common joints between bodies, with a 2 cm thickness, were filled with resilient material.

The five buildings integrate 25 non-specific classrooms and 24 classrooms devoted to specific disciplines.

The five buildings are complemented by a gymnastics pavilion divided into four gymnasiums.

The frame structure comprises flat slabs, columns with a 0.40 m square cross-section integrated in an orthogonal structural grid of 7.20 m and direct foundations by isolated footings.

The specified requirements for the structural elements were a characteristic 28 days compressive strength of 22.5 MPa for the concrete and a A40 T class for the steel reinforcement. The beams, columns and walls have a 20 mm bar cover and the faces in contact with the ground have a 30 mm bar cover.

The building facades consist of cavity walls, with the interior and exterior faces being made of masonry and reinforced concrete panels respectively (Fig. 8-10, Fig. 8-11).




Fig. 8-10 View of the facades before intervention. 2003.



Fig. 8-11 General view of the facades of the bodies. Exterior panels. 1975-1980.

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The structure of all the buildings was constructed in accordance with the traditional methods and the cladding panels of white concrete were after the completion of the construction works.

The 0.08 – 0.10 m thick panels are formed by a single layer of a diameter less than Ø8//0.20 m. Two reinforcement layers, with a 0.08 m thickness, were only used in the outer arcades (Fig. 8-12, Fig. 8-13) and in the semi-circular panels of the safety barrier in the roof (Fig. 8-14, Fig. 8-15, Fig. 8-16).




Fig. 8-12 West view of the buildings in which the covered arcades soften the constructions before intervention. 2003.



Fig. 8-13 Deterioration of one arcade before the intervention. 2003.

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Fig. 8-14 Semi-circular panels of the cast-on-field safety barrier of the roof. 1975-1980.



Fig. 8-15 Exposed corroded reinforcement on the external face of a semi-circular panel of the safety barrier of the roof. 2003.



Fig. 8-16 Deterioration of the inner face of a semi-circular panel of the safety barrier of the roof, with spalling of concrete and exposed corroded reinforcement. 2003.

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The model most commonly used in the facades, except for the east one, has a length in plan of 3.50 m and a height of one floor, and the central zone integrates an opening for the span (2.40 m x 2.30 m) (Fig. 8-17).



Fig. 8-17 Model type of facades. 2003.

The span is shadowed by orthogonal elements of white exposed concrete, with a 0.70 m spacing and a 0.10 m thickness. The models were performed by stages: firstly, the panels around the span, then, the two horizontal shadowing elements, and finally the six smaller vertical gauges of the element. These elements were previously cast-on-field.

## 8.8.2 Inspection highlights

In 2003, less than 25 years after completion, repair works were carried out as a result of the school buildings showing multiple defects related to reinforcement corrosion that was induced by carbonation of the white concrete panels of the facades (Fig. 8-13, 8-15, 8-16, and 8-18 to 8-22). Furthermore, two buildings presented cracking, which resulted from differential settlements with visible deformations of the external paving stone. Additionally, in several areas water leaked from the terrace roof and skylight.



Fig. 8-18 Local areas with exposed corroded reinforcing steel. 2003.

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Fig. 8-19 Shadowing elements delaminated due to corrosion of reinforcement. 2003.



Fig. 8-20 Spalling of concrete due to the small cover depth of the bars positioned perpendicularly to the face of the panel. 2003.



Fig. 8-21 Delamination of the concrete due to corrosion of reinforcement accelerated by the wet-drying cycle. Deteriorated joint. 2003.

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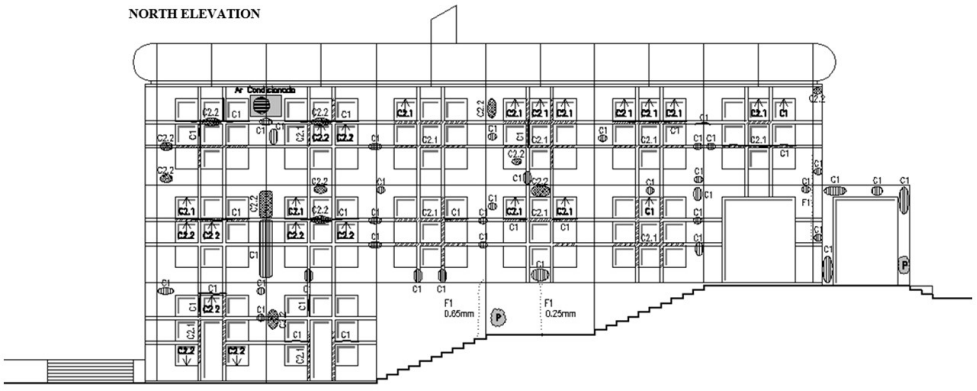
Fig. 8-22 Exposed corroded reinforcement on a horizontal area. 2003.

### 8.8.3 Assessment of the structure

Inspections and tests were conducted with a view to complete the existing information on the concrete quality. The inspection plan was focused on the following aspects:

- Identification and record of defects (Fig. 8-23);
- Mechanical properties of the concrete;
- Water and gas permeability of the concrete;
- Chloride content in the concrete;
- Concrete cover depths and reinforcement location;
- Carbonation depth in the concrete.

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**Legend**

| Symbols | Code | Description | Cause of the defect |
|---|---|---|---|
| | F1 | Cracks | Shrinkage; Overload; Plastic settlement; impact |
| | C1 | Cracking with delamination of the cover | Reinforcement corrosion |
| | C2.1 | **Transversal reinforcing** Spalling of the cover with exposed reinforcing steel | Reinforcement corrosion |
| | C2.2 | **Longitudinal reinforcing** Spalling of the cover with exposed reinforcing steel | Reinforcement corrosion |
| | J.0. | Joint | Lack of joint preparation; old joint |
| | Hole | Hole due to the use of tie bar | |
| P C | Paint or graphite | Paint or/and graphite of concrete | |

Fig. 8-23 Record of defects.

Given the many different exposure and conservation conditions of the cover panels of the facades, it was necessary to select several areas to be subject to the tests for assessing the conformity of the repair materials (Fig 8-24).




Fig. 8-24 Test panels for assessing the conformity of the repair materials. South elevation. 2003.

## 8.8.4 Results of tests and investigations

Mechanical properties of the concrete: The compressive strength of the concrete was obtained from compressive tests performed on seven cores collected from the structures (buildings "B" and "D" Fig. 8-9) with a diameter equal to length of 90 mm and concrete homogeneity achieved by the Schmidt rebound Hammer (buildings "A" and "E", Fig. 8-9).

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Concrete from the panels of façades was classed as a C20/25, according to the European Standard EN 13791, being a C16/20 at 28 days by estimation (Table 8-1). In each 78 test locations, 9 tests were performed with the Schmidt rebound hammer. The average rebound hammer index corresponded to 37 with a standard deviation of 3.98 and a coefficient of dispersion of 11% (Fig. 8-25).

Table 8-1 Characteristic in-situ compressive strength

| Cores | CB1 | CB2 | CB3 | CD1 | CD2 | CD3 | CD4 | Average | STDEV |
|---|---|---|---|---|---|---|---|---|---|
| $f_{is}$ a) | 31.0 | 20.5 | 30.1 | 36.4 | 32.6 | 45.5 | 32.5 | 32.7 | 7.48 |
| Section 7.3.3 of EN 13791:2007 $n=7$, $k=6$ | | $f_{m(7),is}$ (MPa) = | | | 32.7 | Section 6 of EN 13791:2007 - In-situ compressive strength class according to EN 206-1 | | C20/25 | |
| | | $f_{ck,is} = f_{m(7),is}$ - 6 (MPa) = | | | 26.7 | | | | |
| | | $f_{is,\ lowest}$ (MPa) = | | | 20.5 | | | | |
| | | $f_{ck,is} = f_{is,\ lowest}$ + 4 (MPa)= | | | **24.5** | | | | |
| $f_{is}$ b) | 23.8 | 15.8 | 23.2 | 28.0 | 25.1 | 35.0 | 25.0 | 25.2 | 5.75 |
| Section 7.3.3 of EN 13791:2007 $n=7$, $k=6$ | | $f_{m(7),is}$ (MPa) = | | | 25.2 | Section 6 of EN 13791:2007 - 28 days In-situ compressive strength class according to EN 206-1 | | C 16/20 | |
| | | $f_{ck,is} = f_{m(7),is}$ - 6 (MPa) = | | | **19.2** | | | | |
| | | $f_{is,\ lowest}$ (MPa) = | | | 15.8 | | | | |
| | | $f_{ck,is} = f_{is,\ lowest}$ + 4 (MPa)= | | | 19.8 | | | | |

a) Length/diameter ratio =1.0, the strength result is to be compared to cube strength according to 8.2 of EN 12504-1:2019.
b) Considering a hardening coefficient of 1.30 between 28 days and the age of the concrete at the date of the test.

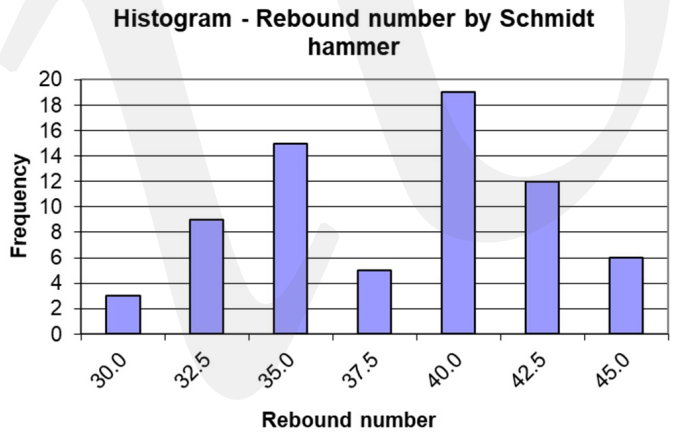



Fig. 8-25 Histogram of rebound number test result by Schmidt hammer.

Water and air permeability of concrete were assessed by the Figg method on three test locations. The results match a poor protective quality for embedded reinforcement: Air Exclusion Rating (AER)<8, according Table from P-6000 Phonoscope Operating Instructions.

The thickness of the concrete elements corresponded to 80 mm for the panels of façades and to 100 mm for the shadowing parts. The results obtained are presented in Table 8-2.

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Table 8-2 Concrete cover depth

| Bar diameter (mm) | Design depth of concrete cover (mm) | |
|---|---|---|
| | 80 mm thick elements | 100 mm thick elements |
| 6 | 34 | 44 |
| 8 | 32 | 42 |
| 10 | 30 | 40 |
| 12 | - | 38 |

Carbonation depth was measured by collecting 26 cores (Ø 30 mm) from the structure (Fig. 8-26, 8-27). Fig. 8-28 depicts the carbonation depth on the external face. The average carbonation depth in the external face corresponded to 28 mm, with a standard deviation of 8.7 mm and a dispersion coefficient of 28%.

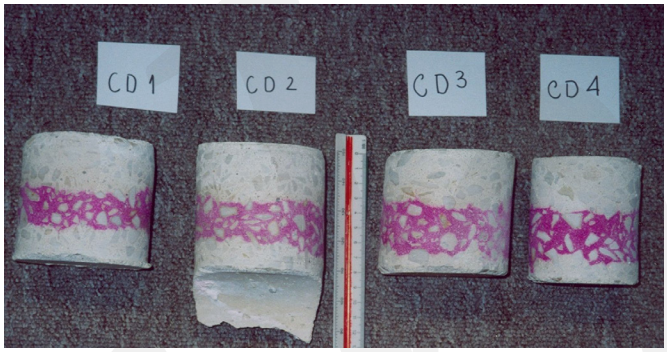



Fig. 8-26 Cored specimens (Ø 30 mm) collected from the panels to measure the depth of the carbonation layer near the surface. Red-purple areas by applying a phenolphthalein indicator solution were not yet carbonated. 2003.



Fig. 8-27 Marking of the position and depth of cover on one panel. 2003.

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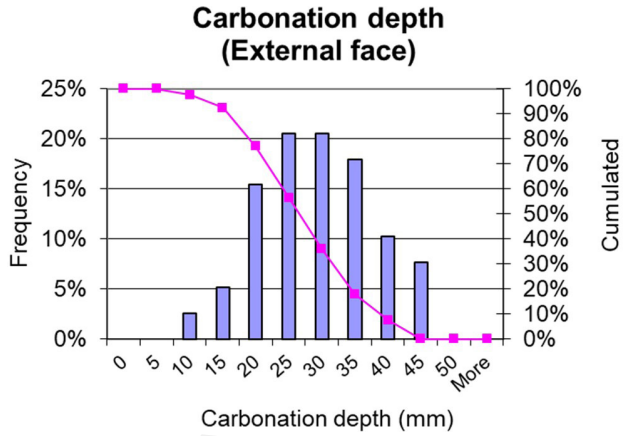



Fig. 8-28 Carbonation depth (External face).

Using Fick's law, the calculated diffusion coefficient of the concrete corresponded to 5.44 mm/year$^{0.5}$. Based on this formulation, Fig. 8-29 shows the estimation of the carbonation depth, provided that the concrete remains without any additional protection.

The average carbonation depth in the internal face corresponded to 19 mm, with a standard deviation of 10.7 mm and a dispersion coefficient of 56%.

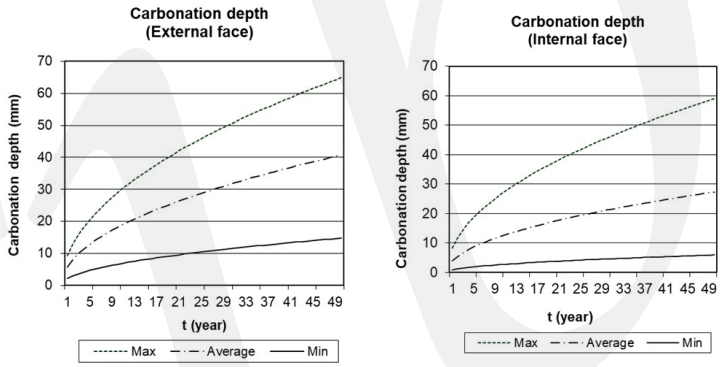



Fig. 8-29 Carbonation depth evolution.

The investigation revealed the following:

- The concrete presented low mechanical properties and showed some heterogeneity;
- The concrete presented high permeability;
- The chloride content was negligible;
- The carbonation depth was very high, with substantial variations throughout the structure, being in many cases deeper than the level of the reinforcement;
- The reinforcing bars had a small diameter and a very uneven distribution in relation to the concrete surface, which resulted in very small cover depths;
- It was possible to observe carbonation of the concrete in the internal face of the panels of the facades;
- The deterioration covered a large percentage of the white concrete surface.

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Therefore, it was possible to conclude that the embedded reinforcement of the panels of the façades and of the other white exposed concrete elements had withstood a uniform corrosion. Previously, the semi-circular panels of the safety barrier of the roof were cast-on-field and the applied concrete had a high slump due to the semi-circular framework. Therefore, the correct positioning of the two layers of the reinforcement was very difficult to perform. For these reasons, these panels showed severe corrosion (Fig. 8-16).

### 8.8.5 Description of the intervention project

The selection of the repair system was strongly influenced by the aesthetic constraints. The repair system had to ensure the preservation of the original aspect of the buildings which was highlighted by its façades of white exposed concrete. Although it could be suitable to apply a coating system with significant thickness to ensure an effective treatment against further carbonation progress, this would remove completely the texture of the white exposed concrete.

Restoring the passivity of the reinforcement by replacing the carbonated concrete was an economically and technically unfeasible repair option, due to the small cross section of the panels, and to the fact that, in some cases, the thickness of concrete to be removed was greater than that of the remaining section.

Given these restrictions, the interventions were focused on the areas where active corrosion was already present, by delamination or cracking of the concrete, and on the areas that despite not showing any sign of corrosion, the risk was deemed to be too high because the level of the reinforcement was less than the carbonation depth.

In order to reduce the permeability of the concrete, the external surface of the latter was coated with a final protective coating. The purpose was to reduce the internal humidity of the concrete and to increase its resistivity, which was expected to result in a decreased corrosion rate of the reinforcement.

### 8.8.6 Description of the intervention works

The repair of the panels consisted of the stages as follows:

Preparation of the substrate:

- Cleaning of the concrete surfaces by grit and water blasting to remove dust, loose material and contaminants;
- Locating and marking the configuration and geometry of repair areas, using a covermeter to provide supplementary information about reinforcement provision and distribution;
- Removal of the concrete to a 15 mm minimum depth by chipping hammer (Fig. 8-30, 8-31).

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Fig. 8-30 Repair of one panel of the façade. Removing concrete using a chipping hammer.



Fig. 8-31 Configuration of the repair areas.

Preparation of the reinforcement:

- Cleaning of reinforcement by grit blasting;
- Identified reinforcement with concrete cover less than 20 mm. Bars localised at surface were cut out when perpendicularly positioned to the face of the panel, or displaced to the interior of the panel;
- Placing supplemental reinforcement steel if the reinforcement steel has lost cross-sectional area.

Hand-applied mortars:

- Bonding: achieve a rough surface profile for the bond between existing concrete and repair mortar;
- Wetting the substrate with water;
- Hand application of low shrinkage hydraulic mortars, with a trowel, to be built up in two or three layers of 20 mm thickness (“Reabilita RBB 50 – SÉCIL”).

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Curing:

- Wet curing by water spraying, with an absorbent material covered with transparent plastic sheeting.

Finishing:

- Coating of the external concrete surface with a surface protection product. The final global coating of concrete surface kept the texture and colour of existing concrete and partly hid the hand applied patch repair with hydraulic mortars. The protective coating was obtained by mixing an additive based on acrylic resin ("SecilTEK AD 05") in watery dispersion, water and white cement (Fig. 8-26, 8-30, 8-31). The bottom face of the façade was additionally protected by a varnish coat ("IMLAR" - anti graffiti).

The repair option for the semi-circular panels was the demolition and substitution of new panels precast in place. A study of mix design was needed to ensure a higher durability for the new concrete as well as better workability.

For durability reasons, the depth of cover adopted was a minimum of 35 mm and particularly attention was carried out for the correct position of the reinforcement. Due the complexity of the geometry of the semi-circular panels, and the demand to ensure well demoulded surface as well as the high number of repetitions, metallic framework was adopted.

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# 9. Recasting (cast-in-place)

## 9.1 Foreword

Repair through recasting consists of replacing damaged or contaminated concrete with repair material (mortar or concrete) that is cast into forms fixed on the structural member, or into holes into pavements. This section briefly describes the principles, materials, design, execution and quality control of this repair method.

A more in-depth description of this method, framed in ISO standardization, will soon be published in an *fib* Technical Report prepared by TG 8.1.

## 9.2 Basics

### 9.2.1 When to adopt this method

This method is often adopted to replace concrete which is contaminated (e.g. due to $CO_2$ and chloride ingress) and faulty (e.g. due to steel corrosion, fire, seismic action, etc.) in vertical and horizontal surfaces. Its main advantages, compared with hand-applied mortar and sprayed concrete, regards the possibility of casting large thicknesses, achieving a uniform surface finish, and protecting the repair mortar or mortar against evaporation and physical impacts at early ages, which makes it easier to control the shrinkage of the repair material. It also usually ensures a better encapsulation of reinforcing bars.

### 9.2.2 Materials and systems

#### 9.2.2.1 General

Due to the typical narrower spaces to be filled and the contact with concrete, that has already undergone most of its shrinkage and creep, the repair material usually needs to satisfy the following requirements:

- Reduced maximum aggregate size;
- Good bond strength (typically measured on concrete substrate prepared in the laboratory);
- Low shrinkage;
- Compressive strength at least equal to that of the substrate (relevant for structural repairs only);
- Modulus of elasticity similar to or lower than that of the substrate, depending on the location of the repair in the structural element;
- Low compressive creep (being high tensile creep favourable).

The following materials are often adopted as repair material.

This chapter was written by Garcia, Lima, Mateos, Monteiro (main author), Ribeiro, and Ribeiro.
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#### 9.2.2.2 Hydraulic mortar or concrete

> mortars or concrete based on a hydraulic binder which is blended together with graded aggregates and can include admixtures and additions which, when mixed with mortar, set by hydrated reaction (ISO 16311-4)

This is the most conventional material used in repair by recasting and, therefore, has the advantage of being well known. However, other materials, such as polymer and polymer hydraulic cement mortars or concrete, bring benefits with regards to execution and control of volume changes.

#### 9.2.2.3 Polymer hydraulic cement mortar or concrete

> hydraulic mortars or concrete modified by the addition of a polymer (ISO 16311-4)

Polymers (e.g. styrene butadiene, acrylics and acetates) are widely used as admixtures to cement-based repair materials, since they have the ability to improve workability, reduce the water to cement ratio and, consequently, the shrinkage effects, and to increase chemical and freezing resistance, among other properties.

#### 9.2.2.4 Polymer mortar or concrete

> blended mixture of polymer binder and graded aggregate which set by polymerisation reaction (ISO 16311-4)

Polymer mortars or concrete are usually adopted only in special applications. Their main advantages are the rapid development of strength (useful when the repaired elements need to rapidly be back to functioning), high resistance to chemical and freezing-taw attacks and high tensile strength. However, they should be used carefully since polymers have a very different behaviour under elevated temperatures compared to hydraulic cement, and are not able to provide an alkaline environment to reinforcing steel.

*Comments:*

*Special care should be taken when polymer repair materials are used, since they may differ significantly from concrete regarding the (lower) modulus of elasticity, (lower) resistance to elevated temperatures, (lower) resistance to UV when directly exposed to sunlight, and impossibility of providing an alkaline environment to the steel reinforcing bars.*

#### 9.2.2.5 Bonding primers

Bonding primers (e.g. epoxies and cement-based) can be used for improving the bond between the repair material and the substrate. However, experience has shown that, for most cases, with a good preparation of the substrate, bonding primers are not necessary. Also, the use of polymeric primers can impair bonding if the moisture conditions of the substrate are not well controlled, or excessive time elapses between the time of application of the primer and the casting of the repair material.

It should be borne in mind that any structural polymeric material may impair the structural behaviour of the repaired element under elevated temperatures, such as those attained during a fire.

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## 9.2.3 Equipment for preparing the substrate

### 9.2.3.1 General

Several techniques can be used for removing the faulty concrete and preparing the substrate (i.e. cut edges, roughening and cleaning) before casting. In the next sections the most common ones are briefly described. The selection of the best technique depends mainly on: size and orientation (e.g. vertical, horizontal, overhead) of the area to be prepared; budget and duration intended for the repair works; availability of the necessary equipment; access conditions of the locations to be prepared; and environmental restrictions. The following sections describe the main techniques currently used for removing concrete and prepare the substrate.

### 9.2.3.2 Blasting techniques

Blasting can be described as the projection of a medium by a nozzle over the surface area to be repaired. It is mostly used for roughening the concrete substrate and, in some cases, for cleaning both substrate and steel bars.

Depending on the medium used, three types of blasting techniques are commonly used:

- Shotblasting, consisting of propelling shots (typically steel grit). Usually adopted in concrete floors for removing coatings and other superficial materials, and roughening the substrate;
- Grit blasting, consisting of projecting pressurized air with abrasives (e.g. sand, dry-ice), with or without water. It can be used for removing coatings and other superficial materials, and roughening the substrate;
- Water blasting, consisting of projecting pressurized water (with or without abrasives). It is the most recommend technique for cleaning and roughening the concrete substrate due to its very low risk of creating microcracking, and also for cleaning steel bars.

*Comments:*

*Shot and grit blasting usually require a final cleaning to remove the dust and other debris (e.g. remaining abrasives), for example, using water blasting, air blasting, vacuum, magnetic brooms, etc.*

### 9.2.3.3 Hydrodemolition

Hydrodemolition consists of projecting highly pressurized water (up to more than 250 MPa). It is used to remove faulty or contaminated concrete in all sorts of surface orientations and usually does not require additional cleaning.

*Comments:*

*An experienced operator can use hydrodemolition for removing concrete up to a desired strength (selective hydrodemolition).*

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#### 9.2.3.4 Mechanical percussion

Percussive equipment is probably the most used for removing concrete and roughening the substrate. However, one of the main disadvantages of this equipment, which are usually overlooked, is the large risk of inducing cracking in the concrete substrate (up to about 15 mm deep), which may compromise further bonding with the repair material.

The most used percussive equipment includes chipping hammers, jackhammers and scabblers.

### 9.2.4 Placement techniques

#### 9.2.4.1 General

The techniques currently used for casting the repair material into the forms can be divided into three main categories: pouring; form-and-pump; and preplaced aggregate concrete. The choice of which technique to use depends on the on-site conditions (e.g. level of confinement of the spaces to be filled with repair material) and on the experience of the contractor. These techniques are briefly described in the following sections.

#### 9.2.4.2 Pouring

Pouring consists of casting the repair material into forms only by the action of gravity. It is the most familiar placement technique where practically all types of repair material can be used.

Today, the adoption of this technique with self-compacting repair material is probably the most recommend method for cast-in-place repairs.

#### 9.2.4.3 Form-and-pump

Form-and-pump consists of casting with pressurized repair material that is pumped into forms that are tightly fitted to the structure.

This technique does not require vibration and is usually less prone to errors compared to pouring.

#### 9.2.4.4 Preplaced aggregate concrete

Preplaced aggregate concrete consists of placing coarse aggregate into the forms and then fill the voids with, typically, grout that is injected from the bottom of the forms.

This technique does not require vibration and is particularly suitable for the repair of confined or difficult-to-reach spaces.

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## 9.3 Stakeholders' roles

### 9.3.1 Owner

The owner is responsible for contracting the other parties involved in the repair process (construction supervisor, health and safety coordinator, designer, contractor, construction supervisor, health and safety coordinator, etc.) and should provide all the available information about the structure necessary for its condition assessment, design and execution. The owner should also define the intended remaining service life and maintenance activities of the structure or structural elements to be repaired.

After the repair, the owner should ensure the maintenance (and possible monitoring) of the repaired elements.

### 9.3.2 Designer

The designer should assess the condition of the structure, prescribe the necessary structural safety measures (e.g. propping, restrictions to the use of the structure, public safety measures, etc.), design the repair satisfying the structural and durability requirements (taking into account the execution techniques available in the place of use), prescribe the necessary quality control and maintenance activities, and provide the necessary technical assistance during the execution. The designer should justify all the design assumptions and calculations necessary for the repair, and elaborate the bill of quantities.

After the repair, the designer should produce the as-built report.

### 9.3.3 Contractor

The contractor is in charge of the execution (including the procurement of materials), and implementation of the necessary safety and health (and, possibly, environmental) measures and quality control plan.

The contractor should demonstrate that both the materials and the selected execution techniques satisfy the design requirements, and should report, as soon as possible, to the designer and to the owner, any non-conformities, incidents and deviations from the design requirements.

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## 9.4 Design

### 9.4.1 General

The design should be preceded by a proper condition assessment of the structure, or structural member(s), based on which the principles of the repair and the selected repair method are defined. It should address the removal of faulty or contaminated concrete (including the associated health and environmental risks, and the need for shoring, propping or even unloading of the structure), the application and curing of the repair materials, the control of execution, and the necessary maintenance measures.

*Comments:*

*Condition assessment usually involves: the analysis of the original design project; an on-site inspection where the concrete anomalies are identified and surveyed, including tests carried out on the structure and on concrete specimens (and sometimes of reinforcing bars) extracted from the structure; the evaluation of the severity and extension of the anomalies; the comparison of the as-built structure with the design specifications; and the estimation of the actual properties of the existing materials (concrete and reinforcement).*

*Most common on-site testing includes rebound-hammer and ultrasonic pulse velocity tests (mostly to detect concrete with low strength), half- cell potential, concrete resistivity and polarization resistance tests (to detect areas with higher risk of steel corrosion), covermeters (to measure concrete cover depth), and hand-held hammers and metallic chains (to detect delaminated concrete).*

*Most common concrete core testing includes determination of carbonation depth, chloride content, compressive strength and modulus of elasticity. In structures exposed to fire, core testing may include a variety of tests in order to assess concrete damage, such as thermogravimetric tests, stiffness damage tests, petrographic tests, etc.*

The repair should be designed for the remaining service life required for the structure, and satisfy the design codes valid in the place of use. When the repair materials are not covered by those codes, their success in this type of repair should be well-established.

The design should take into account aspects that are not typically addressed in the design of new structures, such as: the transfer of existing loads from the structure to the repair; the effect of differences in volume change between the repair material and the substrate (associated with shrinkage, creep and different modulus of elasticity); the potentially imperfect bond between the repair material and the substrate; and the reduction of stiffness of the repaired elements due to deformations in the interface between the repair material and the substrate.

*Comments:*

*Theses aspects are often taken into account by means of coefficients of monolithic behaviour for stiffness and strength.*

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### 9.4.2 Detailing

The detailing drawings should describe the following aspects with the necessary detail:

- The members to be shored, propped or unloaded;
- The areas to be repaired and their respective delimitation;
- The depth of concrete to be removed (which may depend on the strength of concrete if, for ex., selective hydrodemolition is adopted);
- The maximum acceptable loss of cross-section of the reinforcement and how to repair it;
- The steps of the execution procedure;
- The final geometry of the repaired areas, including the minimum clear space around all the exposed reinforcement before casting, the repair materials used, the cover depth requirements, the identification of the areas intended to achieve a particular finishing or to apply a specific coating, and reference to geometrical tolerances.

Whenever the reinforcing bars are corroded or depassivated, or when there are doubts about their bond to the surrounding concrete, they should be undercut and surrounded with the repair material.

*Comments:*

*The European Standard EN 1504-10:2017 recommends that the clearance around the reinforcement and the minimum distance between the reinforcement bar and the remaining substrate should be at least 15 mm or the maximum aggregate size of the repair material plus 5 mm whichever is the greater to allow proper compaction, and that chloride contaminate concrete should be removed on all sides of the reinforcement for a minimum of 20 mm.*

The configuration of the repair areas (Fig. 8-6) should be kept as simple as possible, avoiding re-entrant corners, in order to reduce the tendency to cracking.

The edges of the repair areas should be cut in angles that avoid undercutting the concrete and reduce the risk of debonding (Fig. 8-7). To avoid feather edging, the depth of the cut edges should not be less than the maximum aggregate size of the repair material.

### 9.4.3 Material specifications

Repair by recasting is often adopted to restore the performance of the original concrete (or, at least, reinstate it to an acceptable level). As a principle, the mechanical behaviour, durability and fire resistance of the repair materials should be compatible, as best as possible, with the expected performance of the existing structural materials.

*Comments:*

*For polymer repair materials, additional basic requirements may be necessary in respect of, for example their chemical compatibility with cementitious materials, aesthetic compatibility with the existing concrete surface, fire resistance, resistance to UV light, etc.*

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The main requirements for the properties of the repair material concern:

- Minimum compressive strength (at least the same as the specified for the original concrete);
- Maximum aggregate size (should not be less than concrete cover depth nor less than 1/3 of the minimum space between the substrate and the formwork);
- Workability;
- Mean modulus of elasticity (preferably close to that of the concrete substrate);
- Maximum shrinkage (or minimum bond strength under restrained shrinkage/ expansion conditions, usually established under standard exposure conditions);
- Durability requirements.

## 9.5 Execution

### 9.5.1 Trial testing

Due to the large uncertainty usually associated with the adaptability of the execution methods to the conditions of each repair and the compatibility between the repair materials and the substrate, it is highly recommended conducting preliminary tests on trials or mock-ups.

Trial tests should, at least, allow checking if the repair material is able to fill all the spaces inside the formwork and completely surround the exposed reinforcement with adequate compaction, the occurrence of cracking, and if the bond between the repair material and the substrate is acceptable.

The trials or mock-ups should reflect as best as possible the actual placement, compaction and curing conditions of the repair work in the structure.

### 9.5.2 Pre-intervention operations

Pre-intervention operations may include:

- Preparation of concreting plan;
- Cleaning of the worksite;
- Installation of the necessary scaffolds, shoring, propping, jacking and safety systems;
- On-site marking of the repair areas;
- Implementation of protective measures against rain, wind, contamination of the substrate, etc.

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### 9.5.3 Preparation of the substrate and reinforcement

The preparation of the substrate is usually divided into two steps: removal of faulty or contaminated concrete (and cutting of damaged reinforcement), including cutting of edges; and roughening and cleaning of the repair areas.

The depth of concrete removal should be kept to a minimum, without compromising the execution specifications. Several techniques can be used for removing concrete, such as those described in sections 2.3.3 and 2.3.4. The edges of the repair areas should be cut (e.g. with a saw) according to the configurations defined in section 4.2.

The substrate (including the cut edges) should be rough enough to promote bonding and interlocking with the repair material. Techniques such as those described in sections 2.3.2 to 2.3.4 can be used for roughening the concrete substrate, although blasting techniques and hydrodemolition are the most recommended due to their lower aggressiveness in respect of creating microcracking or partially spalling of concrete. When percussive techniques are used, the occurrence of damage in the substrate and excessive roughening should be evaluated and removed (e.g. with blasting techniques).

The loss of cross-section of the steel bars should then be evaluated and necessary new reinforcement should be implemented.

Finally, the substrate and the reinforcing bars should be free from dust, rust and other debris that can compromise bonding with the repair material. For cleaning the concrete substrate, pressurized water, pressurized air and vacuum cleaners are often used. The reinforcing steel bars are often cleaned with the help of wet sandblasting, water blasting and metallic brushes.

### 9.5.4 Casting and curing

The formwork should be installed shortly after the preparation of the substrate and reinforcement, and should satisfy the specific requirements for the placement technique adopted. Special care should be taken if polymeric bonding primers are used, since they need to be applied just before casting in order to avoid premature setting, compromising bonding between the repair material and the substrate.

When using hydraulic or hydraulic cement repair materials (with no bonding primers), the concrete of the substrate should be close to a surface saturated-dry condition.

The repair material should be placed ensuring that all spaces inside the forms are filled and the exposed reinforcement is completely surrounded, without air pockets, or excessive voidage or segregation.

Main placement techniques are described in section 2.4.

Curing of the repair material should comply with the design specifications. The surface of the repaired areas should be protected from evaporation immediately after casting and for as long as possible in order to avoid cracking induced by drying shrinkage.

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## 9.6 Quality control

The quality of the repair should be controlled and documented according to a pre-established quality control plan. It should address both the materials and the execution, and should establish the requirements for sampling and testing, as well as the respective conformity criteria.

*Comments:*

*Recommendations for the quality control of materials and execution, including testing, sampling frequency and acceptance criteria can be found in EN 1504-10: 2017.*

Quality control of materials should involve checking their identification and pre-qualification properties, and ensure that its consistency and properties in hardened state throughout the repair process is in conformity with the specifications.

All the planned pre-intervention operations (section 5.2) should be, at least, visually inspected.

Before casting, the substrate should be inspected for cracks, delamination, loose aggregates, cleanliness, roughness and temperature. The reinforcement should be inspected for cleanliness and section loss (e.g. due to corrosion). The formwork should be inspected for cleanliness and tightness. The environmental conditions (ambient temperature, precipitation, etc.) should also be checked.

During casting, the workability of the material should be controlled, as well as the occurrence of segregation and leaks from the formwork. The conformity of the performance properties of the hardened repair material should also be checked by means of standard specimens produced with material from the batches used for casting (e.g. compressive strength, in case of structural repairs; durability-related properties; etc.).

After the intervention, it is highly recommended to inspect the repaired areas for cracking and delamination, and measure the actual bond strength between the repair material and the substrate (e.g. through pull-off tests). Additionally, it is also recommended to inspect the actual cover depth and the final geometry (and aesthetics) of the repaired areas.

*Comments:*

*This type of repair is more prone to cracking and delamination than new construction, typically due to the drying shrinkage of the repair material. Cracking may lead to the detachment and falling of the repair material from the structure (especially if it is not anchored to any reinforcing bar), with potential consequences to the safety of people and property.*

## 9.7 Maintenance

This repair technique does often require special maintenance activities. Most common maintenance activities relate to periodical visual inspections for the detection of cracks (or corrosion signs) in the repaired areas and changes in the concrete environmental conditions assumed in the design (e.g. infiltrations, contaminations, etc.), and to maintaining the drainage system clean. Additional maintenance may also be necessary if surface protection systems (e.g. coatings) are applied on the repaired areas.

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## 9.8 Case study

| | |
|---|---|
| Project Title: | Structural rehabilitation of a jetty of the Trafaria Bulk Foodstuffs Terminal |
| Location: | Estuary of Tagus river, Trafaria, Portugal |
| Client: | Silopor - Empresa de Silos Portuários SA |
| Designer: | A2P CONSULT - Estudos e Projectos, Lda. |
| Contractor: | OPCA, Obras Públicas e Cimento Armado, S.A. |

### 9.8.1 Description of the structure

The structure (Figs. 9-1 and 9-2) consists of a 254 m long jetty comprising 9 concrete blocks (12.0 x 21.0 m) founded in the river bed connected with 8 ribbed slabs (20.4 x 19.0 m), and linked to 3 adjacent dolphins (10.0 x 10.0 m) by 3 pedestrian prestressed bridges (42.0 and 22.0 m long). The slabs are supported by 5 main, longitudinal prestressed beams (V1, V2), and 4 secondary, transversal beams (V4, V5).



Fig. 9-1 View of the jetty.

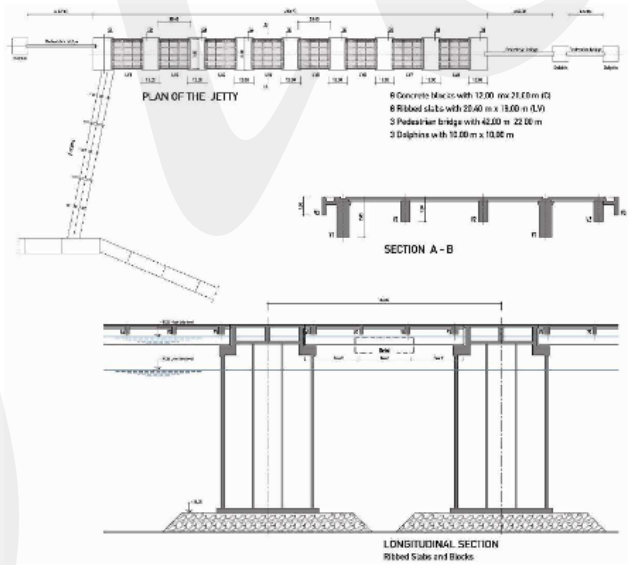



Fig. 9-2 Layout and cross-sections of the structure.

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The deck of the jetty supports the live loads of the terminal use and of three 6'500 kN cranes (1'200 kN live load + 5'300 kN dead weight each) used for loading and unloading ships with cereals.

Date of construction: 1982

Environment: Marine (sea water with a chloride concentration of about 19-21 g/l)

Material specifications of the original structure:

- Concrete grade: C25/30 ($f_{ck,cubes}$ = 30 MPa)
- Passive steel: A400 ($f_{syk}$ = 400 MPa, ribbed steel bars)
- Prestressing steel: A1900
- Minimum concrete cover depth: 35 mm

The specified concrete quality and cover depth are not suitable for a required service life of 50 years under such marine environment.

The structure showed a satisfactory performance for about 20 years. After that, it started to show signs of degradation associated with corrosion of the reinforcing steel.

### 9.8.2 Inspection highlights (2003/2004)

The visual inspection permitted the identification of an advanced state of degradation in the beams, slabs and pedestrian bridges, associated with chloride-induced corrosion. The concrete of the bottom face of the slabs was generally delaminated exposing the reinforcing steel bars (Fig. 9-3). The beams showed signs of cracking, spalling and delamination, mainly in the bottom surface (due to the higher concentration of steel bars, which increases the effect of the expansive forces resulting from corrosion), exposing the reinforcing steel bars (Fig. 9-3). Some of the beams V2 also had the prestress ducts exposed due to concrete delamination (Fig. 9-4). The highest beams (V1) showed less signs of degradation compared to others, which should be due to the fact that, unlike the others, the beams V1 are always reached by the high tides, limiting the access to oxygen necessary for the corrosion process. The pedestrian beams, which are too slender, showed several signs of delamination, exposing the prestressed ducts (Fig. 9-5).



Fig. 9-3 Spalling and delamination in a slab and beams V2 and V4.

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Fig. 9-4 Exposure of the prestressed duct in a beam V2.



Fig. 9-5 Exposure of the prestressed duct in a pedestrian beam.

The blocks did not show visible signs of degradation.

The following tests have been performed to diagnose the degradation, evaluate the risk of corrosion and support the repair design:

- Determination of the concrete cover depth of the beams;
- Determination of total chloride concentration of the concrete cover;
- Determination of the total chloride concentration of the grout inside the ducts of the pedestrian bridge that has collapsed;
- Half-cell potential measurements.

The cover depths showed a large scatter, with values ranging from 17 mm to 62 mm, with an average of 36 mm and a 5$^{th}$ percentile of 18 mm. The bottom faces of the beams showed lower cover depths than the lateral faces.

The critical chloride concentration (assumed as 0.4 wt.% of the cement) was reached up to about 45 mm deep, leading to a large proportion of reinforcing bars significantly susceptible to corrosion (i.e. depassivated).

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Chloride contaminated grout inside the ducts of the pedestrian bridge that has collapsed was also detected. Based on a structural analysis, it was estimated that, for the collapse to occur, at least 2 of the 3 prestressing cables must have ruptured.

Based on the half-cell potential measurements, active corrosion was detected in the bottom and lateral faces of the beams, and no signs of active corrosion in the dolphins were found.

It was concluded that the ribbed slabs and the pedestrian bridges needed urgent intervention.

### 9.8.3 Description of the intervention project (2004)

Objective of the intervention: extend the remaining service life of the ribbed slabs and pedestrian bridges for 50 years.

It was also decided to replace the pedestrian bridges since it was less expensive than repair.

The following intervention options for the ribbed slabs were proposed by the designer:

- Local repair of the damaged areas and implementation of a cathodic protection system to protect the whole structure.
- General replacement of the concrete cover of the exposed surfaces by a high performance microconcrete and application of a surface protection system (epoxy coating, which would not be directly exposed to sunlight) to protect concrete against chloride ingress.

The contractor has chosen the second option.

Concrete specifications:

- Min. cover depth: 50 mm (tolerance of 5 mm)
- Grade: C35/45 ($f_{ck,cubes}$ = 45 MPa)
- Cement content: 300 kg/m$^3$ < C < 400 kg/m$^3$
- Max. water to cement ratio: 0.40
- Max. aggregate size: 10 mm
- Min. moist curing period: 12 days
- Max. water penetration at 28 d (ISO 7031): $x^{average}$ < 20 mm; $x^{max}$ < 50 mm
- Max. water permeability coeff. (test method to be proposed by the contractor): 1x10$^{-12}$ m/s
- Max. chloride diffusion coeff. at 90d (NT Build 492): 1x10$^{-12}$ m$^2$/s
- Max. water absorption at 28d (Taywood test): a < 0.1 mm/min$^{0.5}$, x < 10 mm after 4h
- Max. porosity at 28d (LNEC E 395): 14%
- Min. bond strength at 28d (pull off): 1.0 MPa

Epoxy coat specifications:

- Interzone 954 epoxy paint
- Min. dry thickness: 300 µm
- Max. chloride diffusion coeff. (EN 838): D < 1x10$^{-10}$ cm$^2$/s

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Repair procedure for beams V1 and V2:

Two options were proposed:

- Preplaced aggregate concrete
  - Removal of concrete cover from the bottom of the beams until the depth of the second layer of reinforcing bars (Fig. 9-6a). This decision was due to difficulties associated with the high density of steel bars
  - Cleaning of the exposed reinforcing bars and reinforcing of bars with more than 20% of section loss
  - The substrate should be pre-wetted by successively applying water for 12h prior to the application of the repair material
  - Removal of the excessive water from the substrate and application of preplaced aggregate ($D_{max}$ < 15 mm) concrete at the bottom surface of the beams (placement of coarse aggregates followed by injection of cement grout, Fig. 9-6b)
  - Removal of concrete cover from the lateral faces of the beams up to a depth of 20 mm beyond the reinforcing bars
  - Cleaning of the exposed reinforcing bars
  - Cleaning of the substrate with water jets
  - Application of shotcrete in the lateral faces of the beams (Fig. 9-6c)
  - Curing and application of the epoxy coating

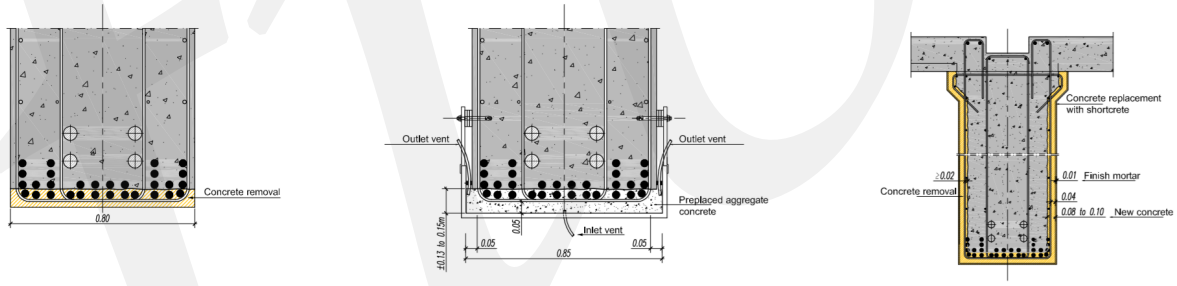



Fig. 9-6 Exposure of the prestressed duct in a pedestrian beam.

- Self-compacting concrete
  - The same procedure adopted for the beams V1 and V2 but removing the concrete cover from the bottom and lateral faces in the same operation, using only self-compacting concrete for the replacement of the concrete cover

Repair procedure for beams V4 and V5:

- The same procedure adopted for the lateral faces of beams V1 and V2, but for all the faces of the beams (Fig. 9-7), with the addition that one of the tops of the lateral faces of the beams, where the self-compacting concrete is placed, is cast with shotcrete

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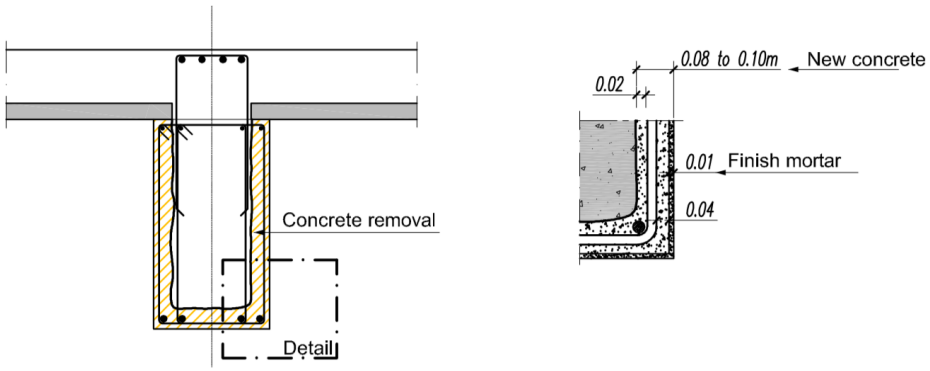



Fig. 9-7 Design repair of beams V4 and V5.

For the beams V3 only the local repair (patch) of the damaged areas and the general application of an epoxy coating was decided.

Repair procedure for the slabs:

- The same procedure adopted for the lateral faces of beams V1 and V2, but considering two options:
  - Wet shotcrete
  - Self-compacting concrete

The removal of concrete should be done with a light chipping hammer (max. 6 kg), followed by sandblasting to remove the loose material and eliminate the excessive roughness. The reinforcing bars should be cleaned with sandblasting to the degree SA 2.

No special maintenance and monitoring requirements have been prescribed.

Estimated quantities of the repairing works:

- Beams (V1 to V5): 4'400 $m^2$
- Slabs: 490 $m^2$

Estimated cost: 1.25 x $10^6$ € (cost/minimum salary = 3'415)

## 9.8.4 Description of the intervention works (2004/2005)

To find the best option for repairing the beams, the following trial testing program was executed:

- Two reinforced concrete blocks were moulded on site
- The concrete cover was removed from the bottom face of the first block and recast with preplaced aggregate concrete (Fig. 9-8)
- The concrete cover was removed from the bottom and lateral faces of the second block and cast with self-compacting concrete (Fig. 9-9)
- Both blocks were then cored (Figs. 9-10 and 9-11) in order to visually assess the quality (segregation, voids, heterogeneity, etc.) of both concretes

Due to the large heterogeneity detected on the interface between the old and the new preplaced aggregate concrete (thick layer of grout on the interface, Fig. 9-10), it was decided to use self-compacting concrete.

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Fig. 9-8 Frist trial block. Application of preplaced agg. concrete.



Fig. 9-9 Second trial block. Application of self-compacting concrete.




Fig. 9-10 Frist trial block. Coring of preplaced agg. concrete.

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Fig. 9-11 Second trial block. Coring of the self-compacting concrete.

Composition of the self-compacting concrete:

- Cement CEM I 42.5 R: 355 kg/m$^3$
- Fly ash: 117 kg/m$^3$
- Slag: 195 kg/m$^3$
- Sand: 704 kg/m$^3$
- Coarse aggregate ($D_{max}$ = 12.7 mm): 704 kg/m$^3$
- Admixture 1 - Glenium 26 scc: 18.1 kg/m$^3$
- Admixture 2 – Bettoretard: 2.33 kg/m$^3$
- Water: 188 l/m$^3$ ($w/b$ = 0.28)

Regarding the slabs, it was initially decided to use shotcrete. However, since an excess of voids was detected in the interface with the substrate, it was decided to use self-compacting concrete.

Prior to the execution, the slabs were propped and restrictions were imposed on the positioning of the cranes.

The procedure defined in the repair design (section 9.8.3 "Description of the intervention project (2004)") was executed. The replacement of concrete cover was carried out in phases in order to maintain, as best as possible, the reinforcing bars bonded to the concrete (Fig. 9-13).

In Figs. 9-12 to 9-15 are shown pictures of some of the execution steps.



Fig. 9-12 Main beam after the removal of the concrete cover.

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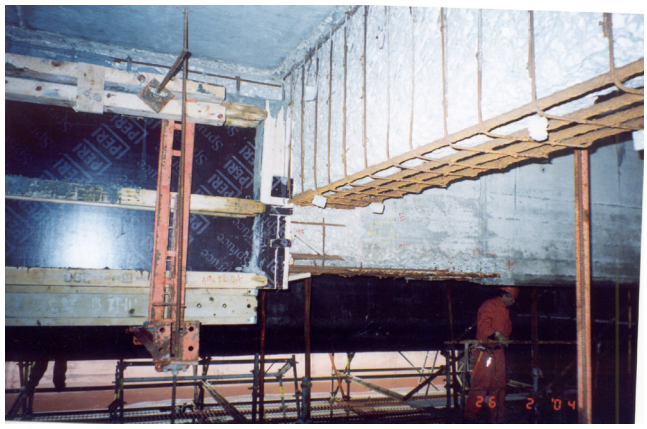


Fig. 9-13 Formwork in one of the main beams.



Fig. 9-14 Curing.



Fig. 9-15 Application of shotcrete on the top of the lateral faces of a beam.

## 9.8.5 Description of procedures adopted for quality control (2004/2005)

Quality control of concrete:

- Control of composition and workability
- Compressive tests: 3 specimens per each 5 $m^3$ of concrete applied
- Water absorption, water penetration, porosity and chloride penetration: 3 specimens per each 5 $m^3$ of concrete applied

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Quality control during intervention:

- Control of the preparation of the substrate, cleaning of reinforcement and application, curing and surface protection of concrete
- Adhesion between new and old concrete: 1 pull-off test per each 40 $m^2$ of repaired surface. If the results were lower than 1 MPa (except if the rupture of the specimen occurs within the old concrete), two additional tests should be performed within a diameter of 1 m containing the initial test location. If both new test results were not lower than 1 MPa, the execution is considered in conformity with the specifications, otherwise the need to introduce connectors should be assessed.

Results available:

- Compressive strength:
  - at 7 days: 46 to 64 MPa
  - at 28 days: 71 to 96.5 MPa
- Water absorption: 2 to 19 mm after 4h
- Water penetration: 7 mm of maximum penetration
- Porosity: 12.7 to 15.8%
- Chloride penetration (AASHTO-ASTM C1202-91)*:
  - at 28 days: 879 to 1652 Coulombs
  - at 80 days: 498 to 879 Coulombs
- Pull-off test:

  Slab LV5: 0.8 to 2.0 MPa (all ruptures occurred in the old concrete)

* Proposed by the contractor (and approved by the designer) due to difficulties in performing the NT Build 492 test method.

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# 10. Shotcrete

## 10.1 Foreword

Projection is one technique among others for placing construction materials as mortar, concrete and plaster.

Unlike a casting method, spraying does not require formwork to hold the concrete. It is projected onto a support called subjectile, mobilized by adhesion to form a monolithic unit or used as a formwork bottom.

Sprayed concrete (Shotcrete) is implemented by conveying at high speed a dry or wet blend of cement and aggregates through a pipe by means of compressed air (followed by water addition at the nozzle in case of dry process) and spraying onto a given surface to strengthen the structures.

Recently developed for specific cases, fibres are added, as for Fibre Reinforced Concrete (FRC), to provide an Ultra High Performance Concrete (UHPC).

The shotcrete allows strengthening the structures including various surface shapes. It is a suitable method for repair, strengthening, increasing the loading capacity of structures, or to build structures with particular shapes.

In all cases, the principle of projection remains the same. It consists of:

- mixing, homogenizing materials in a dry or wet state, transport them by pipeline, rigid or flexible, using mechanical pumps or compressed air;
- spraying the concrete, using compressed air, on the supports to be coated.

## 10.2 Basics

### 10.2.1 When to adopt this method

The use of concrete and spray mortars has been used for many works/structures and applications (Fig. 10-1 and 10-2).

- Concrete structures (bridges, footbridges, etc)
- Buildings, car parks, commercial centres
- Concrete and masonry dams
- Industrial structures
    - Cement preheating towers
    - Silos
    - Water cooling towers or tanks

This chapter was written by Appleton, Costa, Mercier, and Zivanovic (main author).

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- Marine structures
  - Berths and jetties
  - Locks
- Underground structures
  - Concrete and masonry tunnels
  - Culverts
  - Buried metal culverts under roads and motorway
  - Galleries



a) Repair of tunnel



b) Increase of service level (bridge)



c) Frost-degraded sidewalls (tunnel)



d) Embankment protection



e) Climbing wall



f) Buried metal culverts under roads

Fig. 10-1 Shotcrete applications.



Fig. 10-2 Restoration of structures damaged by fire.

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Typical applications of shotcrete include the following:

- Concrete structures repair after fire event
- Concrete structures repair after corrosion induced delamination
- Increase in concrete cover
- Increase of concrete cross-sectional area
- Addition of rebars
- Increase in flexural inertia
- New and free-form structures

### 10.2.2 Materials

The concrete used must have a cohesion capable of supporting its self-weight, while allowing sufficient workability during installation (shotcrete) and a perfect coating in the presence of steel reinforcements.

Mechanical properties allow design of the shotcrete reinforcement according to relevant standards described hereafter.

The spraying principle gives concrete mechanical resistance, compactness and durability that vary according to spraying techniques.

The diversity of parameters (related to equipment, materials, projection or to the specificities of the site) and their influence on the formulation constitute a complex whole which must be taken into account, for each case, in the research of a concrete formulation adapted to the implementation and the expected final needs.

The dry process is characterised by the use of mixtures with a powdery consistency to allow passage through the equipment hoses and transfer by compressed air for the projection onto the support.

The wet process is characterised by the use of plastic consistency mixes and must be pumpable (requires sometimes the use of specific additives and fibres to ensure concrete remains on the support).

Expectations (for both dry and wet process):

- Dry, homogeneous & no-segregation premix
- No plugs in pipes, no chaotic spray
- No excessive dust
- Controlled losses due to rebounds
- High adhesion to support
- Low Friction coefficient to reduce equipment wear
- Good cohesion of the premix to high speed flow
- Strong aggregate/cement paste interface

For the dry method, the composition of a mixture intended to be sprayed is an optimisation in order to reduce losses during projection.

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### 10.2.2.1 Selection of constituents

defined case by case depending on local sourcing & local standards, with full determination and traceability for each. Refer for example to ACI 506.2-95, EN14487-1 or equivalent standards.

- Cement
- Aggregates including min/max grading curves
- Cement admixtures
- Additions
- Water
- Fibres

Prescriptive requirements to the shotcrete (in addition to performance requirements of EN 14887-1):

- Minimum cement content
- Classes according to EN 14487-1 standard (table 8): J1, J2 and J3
- Max diameter aggregates:
  - 0/4 for mortar (finishing)
  - 0/8 for concrete (structural)
  - 0/12 for concrete (large volumes)
- Regular & continuous grading curve for sands, gravels and premix
- Limited content of fines (depending on the required strength class)
- Preferably rolled & flat aggregates
- Low porosity and water content aggregates (from 2% to 5%)
- Clean and dry sands and gravels
- Hexavalent Chrome content reducing agent
- No risk of ASR
- Content of additions (in % of cement weight)
  - Silica fumes: typically, 5% to 8% (increase thixotropy & cohesion, strength and durability)
- Content of admixtures (in % of cement weight / can be a combination)
  - Hardening accelerator: within a few hours (accelerate strength increase)
  - Set accelerator: within a few minutes
  - Stiffener: immediate action (increase thixotropy &cohesion)
- Contents of fibres
  - Steel fibres: typically, 30-35 kg/m$^3$ (anti-shrinkage, flexural strength, impact strength), to 150 kg/m$^3$ for UHPFRC spraying
  - Polypropylene fibres (anti-shrinkage, fire resistance)
  - Polypropylene macrofibres more and more used for flexural strength (typically 3 to 6 kg/m$^3$)

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Table 10-1 Shotcrete early strength classes according to EN 14487-1

| Fresh concrete temperature | Accelerated CEM I-based concrete | |
|---|---|---|
| | Admixture <5% | Additive >5% |
| 5 to 15 °C | J1 | J2 |
| 15 to 25 °C | J2 | J3 |
| 25 to 35 °C | J3 | J3 |

### 10.2.3 Techniques

Schematically, there are two techniques that differ regarding the location of the introduction of the mixing water into the material.

- In the dry process method (Fig. 10-3), the mixture of cement and aggregate, not added with water at the time of mixing, is propelled by compressed air in the pipe leading to the lance where the necessary water is added.

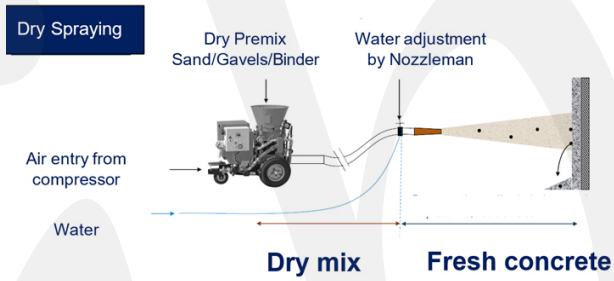



Fig. 10-3 Dry spraying process.

- In the wet spraying method (Fig. 10-4), the mortar or concrete is transported to the lance either by pumping (dense flow) or by compressed air (diluted flow).

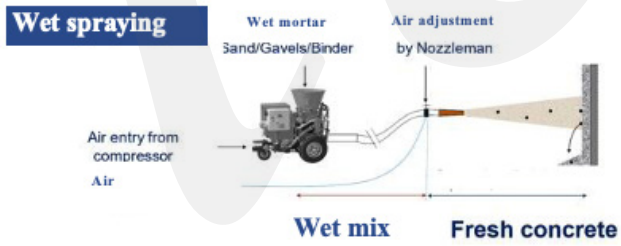



Fig. 10-4 Wet spraying process.

The choice of technique to be used will depend on:

- material to be sprayed,
- the nature of the work to be carried out.

Within the technique, and according to the reinforcement requirements to meet at the design stage, the shotcrete properties are selected to achieve the strength up to the use of metallic reinforcement elements (rebars, fibres). Post-tensioning tendons or strands can be combined with shotcrete technique when relevant.

The dry process prevails where concrete quality, compactness, bond strength and shrinkage limitation are predominant: typically for the repair and strengthening of structures.

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Another distinction for selecting either method is the volume and thickness of the repair. The dry method by the use of pre-bagged materials yields a lower production rate than the wet method in large shotcrete repairs.

Sprayed UHPFR of ultra-thin structural layer prevails where it is important to preserve the geometry (doesn't reduce the clearance of the structure), increase strength and durability and reduce execution time (Fig. 10-5).

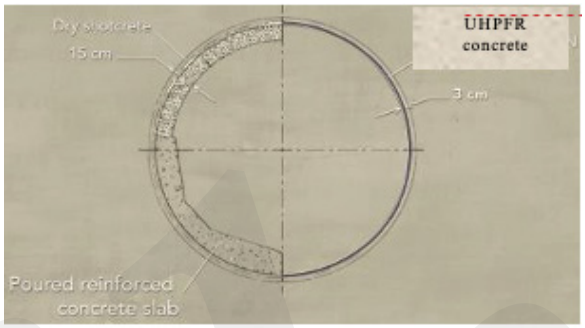



Fig. 10-5 Standard / UHP concrete.

## 10.2.4 Equipment

The main elements are the following (Figs. 10-6 to 10-8).

General equipment:

- Transportation and loading systems
- Spraying machine and spare parts
- Compressor and its fuel tank
- Pipes and their connections
- Lance and its nozzle
- Devices for the confinement, ventilation and filtration of work areas






Fig. 10-6 Mixing and transportation equipment.

Specific to dry spraying method:

- Silo equipped with a pre-wetting screw
- Pre-wetting mixer

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Specific to wet spraying method

- Mixing equipment
- Water pump and its tank

Selection of the equipment, depending on:

- Aggregate size
- Surface and volume to be treated
- Horizontal distance between the nozzleman and the spraying machine
- Vertical distance between the nozzleman and the spraying machine

Dry spraying machines mainly use the rotor or peristaltic principle.

For wet spraying process, piston pump machines are the most commonly used.



Fig. 10-7 Rotor machine.

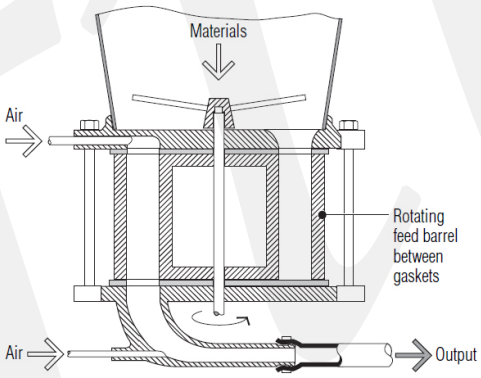





Fig. 10-8 Rotary-barrel machine.

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#### 10.2.3.1 Spraying equipment

It uses sealing gaskets on the top and bottom of the rotating barrel. Material is gravity fed from the hopper into the cells of the barrel in one area of its rotational plane and discharged downward from these cavities with air pressure at the opposite point in its rotation. Additional air is introduced into the outlet elbow to provide proper volume and pressure for material delivery down the hose (Fig. 10-9).



Fig. 10-9 Example of lance for dry spraying.

#### 10.2.3.2 Compressed air and fuel tank

The air flow rate is the most important point for a good process.

The compressor shall have sufficient capacity to provide sufficient quantity of clean, dry air at the required pressure in order to prevent any fluctuation in pressure and maintain sufficient velocity at the nozzle. The air flow rate is selected in relation to:

- shotcrete distance and difference in altitude between spraying machine and nozzlemen
- diameter and type of hoses/pipes
- concrete mix behaviour

## 10.3 Stakeholders' roles and qualifications

The design of the shotcrete solution should be developed by a qualified and experienced engineer.

The installation phase should be made by a specialist company having staff with appropriate skills and experience. Shotcrete work shall be performed only by fully qualified specialists.

Proper supervision and quality control should be provided during the installation phase.

*Comments:*

*It is commonly admitted that it takes 5 years for a shotcrete specialist called nozzleman to become fully efficient, as "Highly Skilled Nozzleman", and that it will take 5 more years of experience to become a "Shotcrete Project Manager" and then a trainer within a specialist company.*

*For example, in France, a certification scheme is established by Asquapro.*

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## 10.4 Design procedure

### 10.4.1 Assessment of existing conditions

Prior to the design of the strengthening system, the assessment of the existing structure must be done using existing codes or guidelines, e.g. ISO 13822:2001 or ACI 364.1R-07 or equivalent applicable standards.

The designer must know in particular:

- Characteristic of the support concrete (compressive strength, surface cohesion)
- Mechanical characteristics and sections of existing rebars
- Environmental conditions
- Composition of loading (before, during and after reinforcement)

### 10.4.2 Codes and standards and other relevant references

- American Concrete Institute (ACI) and ASTM standards:
  - ACI 506R-05 “Guide to Shotcrete,”
  - ACI 506.2-95 “Specifications for Shotcrete,”
  - ACI 506.4R-94 “Guide for the Evaluation of Shotcrete.”
  - ACI 506.1R-08, “Committee Report on Fiber-Reinforced Shotcrete”
  - ACI 506.5R-09, “Guide for Specifying Underground Shotcrete”
  - ACI 506-3R-91 – Guide to Certification of Shotcrete Nozzelman
  - ACI Committee 506 new document, “Guide for Contractor Qualifications for Specific Projects"
  - ACI 506.6T-17 “Tech Note for Visual Evaluation of Shotcrete Core Quality
  - ASTM C1140-03a - Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panels
- European standards
  - EN 14487-1:2005, Sprayed concrete Part 1: Definitions, specifications and conformity
  - EN 14487-2, Sprayed concrete Part 2: Execution of Sprayed Concrete works
  - EN 14488-6, Testing sprayed concrete
    - Part 1: Sampling fresh and hardened concrete
    - Part 2: Compressive strength of young sprayed concrete
    - Part 3: Flexural strengths (first peak, ultimate and residual) of fibre reinforced beam specimens
    - Part 4: Bond strength of cores by direct tension

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  - Part 5: Determination of energy absorption capacity of fibre reinforced slab specimens
  - Part 6: Thickness of concrete on a substrate
  - Part 7: Fibre content of fibre reinforced concrete
- EN 934-5, Admixtures for concrete, mortar and grout — Part 5: Admixtures for sprayed concrete

- French standards

  Engineering structures - Repair and strengthening of concrete and masonry constructions - Shotcrete – Specifications
  - French association ASQUAPRO (Fascicule 1 to 8)
- French Tunnels associations AFTES
  - Design and calculation of sprayed concrete for undergrounds works
- European association EFNARC
  - Specifications
  - Execution of spraying
  - Guidelines for specifiers and contractors
  - Checklist for specifiers and contractor
- United Kingdom Spray Concrete Association (SCA)
  - Introduction to spray concrete
- Concrete Institute of Australia
  - Recommended practice - Shotcreting in Australia

### 10.4.3 Design assumptions

Project shotcrete strengthening requirements for materials are defined according to EN 14487-2 section 4.1 and Annex A.1 or according to relevant countries local standards.

The project specification should include all necessary information and technical requirements for execution of the works.

- purpose of concrete spraying (structural or non-structural)
- requirements related to the construction works (phases, strength evolution, temperature, ambient conditions, available space/congestion of the work area)
- requirements for concrete properties (EN 14887-1 or equivalent)
- qualification/certification of personnel
- requirements for thickness (minimum, maximum, variation)
- requirements for surface finish and coating if any
- requirements for quality assurance plan for execution
- inspection category (EN 14487-1 or equivalent)
- requirements related to health and safety (dust emissions)

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- provision for welding, overlapping, bending, and anchoring for rebars
- restriction for curing compounds (if any)
- requirements for storage and movements of materials
- requirement related to waste deposit (rebounds)

Properties of hardened concrete according to EN 14487-1 (or equivalent standard)

- Class of compressive strength
- Class of exposure of each member and chloride content
- Adhesion strength to substrate and interlayer adhesion strength at 28 days
- Initial strength evolution - first hours
- Initial/final setting times, early strength evolution - first days
- Apparent density
- Air content
- Modulus of elasticity
- Porosity accessible to water, chloride diffusion coefficient
- Sulphate/sea water and frost resistance
- Sensitivity to ASR (sprayed concrete & concrete in place)
- Shrinkage
- Electrical resistivity (cathodic protection)
- Lifetime

In the case of UHPFRC, the material must meet the requirements of NF P 18470-2016 standard (compression strength at 28 days > 130 MPa, Porosity accessible to water < 6%, Chloride diffusion < 5. $10^{-13}$ $m^2/s$). There is no preferential orientation of fibres after spraying.

### 10.4.4 Design procedure

Calculation is carried out at SLS and ULS. It is done following Eurocode 2 Part 1.1 or according to local applicable standards for the project with a verification of stress, deflection. Crack opening verification should be done according to the environment class exposure.

The first step consists of checking stress state (safety condition) of the existing structure (initial state) from the known elements (geometries, concrete characteristics and steel reinforcement).

The second step is to check the stress state (safety condition) in the final state (with shotcrete). The initial state / final state balance allows the definition of the reinforcement sections to be brought within the limits of characteristics of materials (concrete, steel rebars).

It is also possible to reduce the stress state of the materials by a jacking operation of the element, which allows in the final phase that the reinforced shotcrete takes up part of the weight of the initial structure on release of jacking (release of load taken by jacking operation).

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It is also necessary to check the conditions of transfer of forces from the additional structure to the existing one. This verification must satisfy transfer conditions determined from the main reinforcement added rebars if any. Transfer in extremity of rebars (to be added) can be by anchoring in the elements to be reinforced (details in 6.2.5 of Eurocode 2).

### 10.4.5 Supporting documents

Results of all activities performed in the following stages shall be recorded:

1. Assessment, evaluation and diagnosis of the existing concrete structure
2. Planning, design and execution of the repair/strengthening operation
3. Maintenance and inspection of the structure, after the intervention

## 10.5 Execution

### 10.5.1 Preparatory works

#### 10.5.1.1 Surface inspection & reinstatement

Prior to the preparation of the support, the surface is inspected to detect any delamination or loose material (hammer sounding), identify cracks, signs of rust or honeycombs (visually) and to precisely limit the zones to be treated.

- break out the loose or damaged parts of the concrete
- fully expose the rebars (made visible)
- remove non-adherent rust
- clean until sand grains appear
- treat possible water inflow

#### 10.5.1.2 Preparation of the substrate

The fundamental principle of the shotcrete adhesion onto the hardened concrete substrate is not a chemical bond but the creation of connections between them. One crucial objective of the surface preparation is the removal of the laitance so that the sand grains and aggregates appear (Fig. 10-10).

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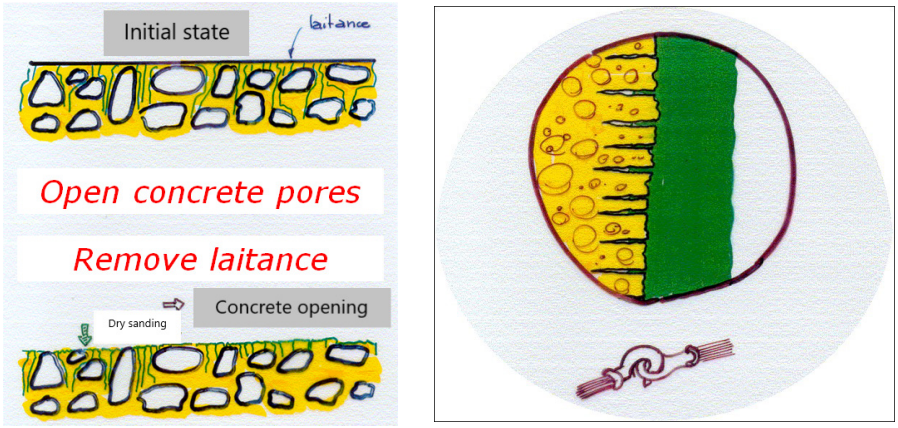



Fig. 10-10 Objectives of surface preparation.

Sand-blasting, hydro-sanding, high pressure water jetting (500 bar), hydro- demolition (2'000 bar), chipping, or scabbing can be used.

Dry sand-blasting is the most efficient surface preparation technique but requires specific provisions to manage dust emissions and abrading material projections. The substrate must be thereafter dry-air blown to remove any dust that would impair the adhesion (Fig. 10-11).



Fig. 10-11 Surface aspect after sandblasting.

Ultra-high pressure water jetting (2'000 bar) is used when concrete demolition is required over a few centimetres (to remove carbonated or chloride polluted concrete) (Fig. 10-12).



Fig. 10-12 Surface aspect after high pressure water jetting.

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High pressure water jetting (500 bar) often combined with a rotative nozzle can be used to remove a thin layer of laitance. The surface must be abundantly washed to eliminate the residues. After water blasting, any excess of water shall be removed from the surfaces using air blasting.

### 10.5.1.3 Rebars / metallic frame

If necessary in the design, structural reinforcements in shotcrete have the same role as in poured reinforced concrete (Fig. 10-13). They participate in the reinforcement, whether in the provisional phase or in the final phase. The calculation must comply with the same rule (see the chapter "design").

The use of UHPFR concrete allows reduction of the use of rebars and decrease of the thickness to be sprayed.



Fig. 10-13 Structural reinforcement (rebar / metallic frame) examples.

Bar diameter shall not exceed 25 mm. It is preferable to increase the number of bars and to reduce their diameter. The use of bars with a diameter of over 25 mm may require feasibility tests and the development of a specific spraying procedure.

- 1st technique consists of placing the reinforcements layer after layer after spraying a layer of concrete.
- 2nd technique consists of placing the entire reinforcement on the structure before concrete spraying regardless of the number of layers.

Spacing (Fig. 10-14):

- Between the substrate and the reinforcements: equal at least their diameter
- Between reinforcements: must equal at least 20mm.

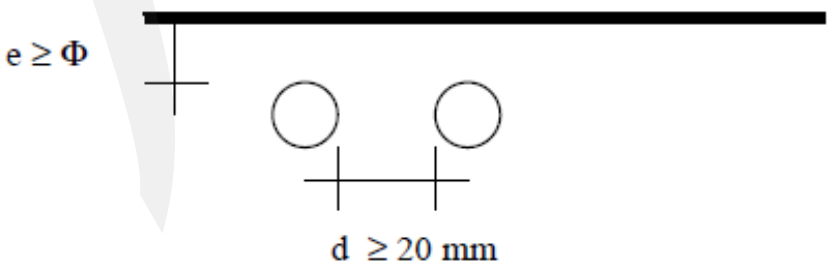



Fig. 10-14 Reinforcement spacing rules.

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Fixing to the substrate: Depending on the flexibility of the reinforcement layer or its weight, a minimum number of fixings (2 to 6 /m²) should be implemented.

These fixings are steel bars anchored or beaten (6 to 8 mm), anchors, waiting steel bars incorporated in a previous layer, metal brackets, etc.

Overlapping of bars (Fig. 10-15): It must be performed by superposition of bars in a plane perpendicular to the facing and not in a parallel plane. The overlapping length must comply with calculation requirements of codes.



Fig. 10-15 Overlapping of bars and fixing of rebars to the existing structures.

Connection of shear rebars (Fig. 10-16): see for example schemes in NF P 95 102 norm.




Fig. 10-16 Example of connection of shear rebars.

### 10.5.1.4 Formworks

They are only installed at the latest stage when formwork is required (Fig. 10-17). Timber formwork can be damped before spraying or coated with release agent to prevent absorption of moisture and adhesion of the shotcrete.

The formwork should be adequately supported (Fig. 10-17) and strengthened to prevent excessive vibration and deflection. Shoring and fixing must comply with the regulations in force and may be subject to justification by calculations.

For formwork of columns, different stages will be necessary during the process of spraying (Fig. 10-18).

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Fig. 10-17 Examples of formworks.

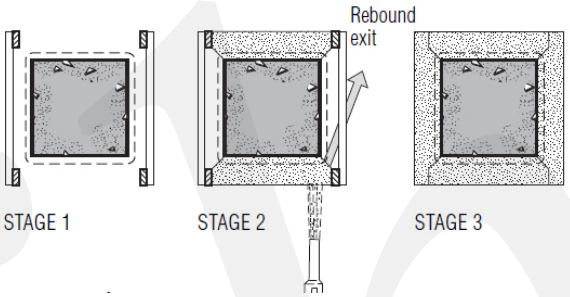



Fig. 10-18 Stages for formwork of column.

## 10.5.2 Execution procedure

### 10.5.2.1 Hydration of the substrate

Substrates likely to absorb the water from the fresh concrete shall be thoroughly dampened beforehand (6 hours before).

For the dry process, dampening is carried out before spraying, via air- blowing with addition of water at the level of the nozzle in order to produce a mist which facilitates water penetration into the concrete. The concrete must be sprayed onto a damp but not dripping surface. It is evaluated by visual examination. The surface must have a mat/dark appearance, with no standing water (Fig. 10-19).

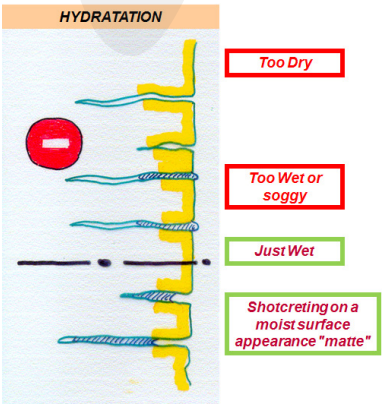



Fig. 10-19 Hydratation of the substrate principle.

Note: In case of intense heat, strong ventilation or when spraying on very porous surfaces, several successive dampening operations may be necessary.

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### 10.5.2.2 Mixing (in case of wet process)

It consists of weighing and adding water, additives (and fibres if any) to the premix in large bags (Fig. 10-20).



Fig. 10-20 Introduction of elements and mixing.

### 10.5.2.3 Spraying

Sprayed concrete is implemented in successive “layers” (Fig. 10-21). A “layer” is known as a coating of concrete which is left to set either before another spraying session or because the required thickness has been achieved.

Each layer of shotcrete is made in “runs”. A “run” is the coat of fresh concrete which the nozzleman sprays on progressively in order to obtain a layer.

The spraying force triggers losses – by rebound and sinking – in variable proportions. This projectile force enables compaction and thereby ensures the sprayed concrete's strength, compactness, waterproofing, reduction of shrinkage and bond-strength to the substrate. The shotcrete specialist is able to manage the thickness of the shotcrete needed during the projection phase.

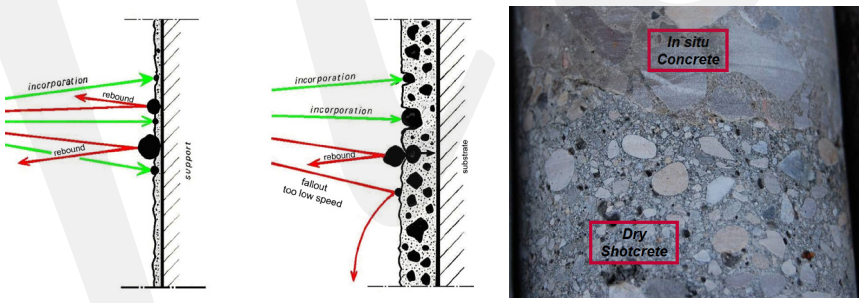



Fig. 10-21 Layer built-up by shotcreting.

The nozzle should be directed perpendicular to the face to apply at all times (Fig. 10-22). Distance from nozzle to the receiving face should be between 0.6 to 1.5m to achieve the highest degree of compaction and lowest rebound. The nozzle man has to move the nozzle with a slow and regular rotating movement. The shotcrete should emerge in a steady uninterrupted flow. The sprayer should firstly fill all over-breaks and zones of weakness if applicable.

Spraying should start from the ground and move methodically upwards (Figs. 10-23 and 10-24).

Where thick layers are applied, it is important that the top surface be maintained at an approximately 45˚ slope to avoid rebound material contaminating the shotcrete (Fig. 10-22).

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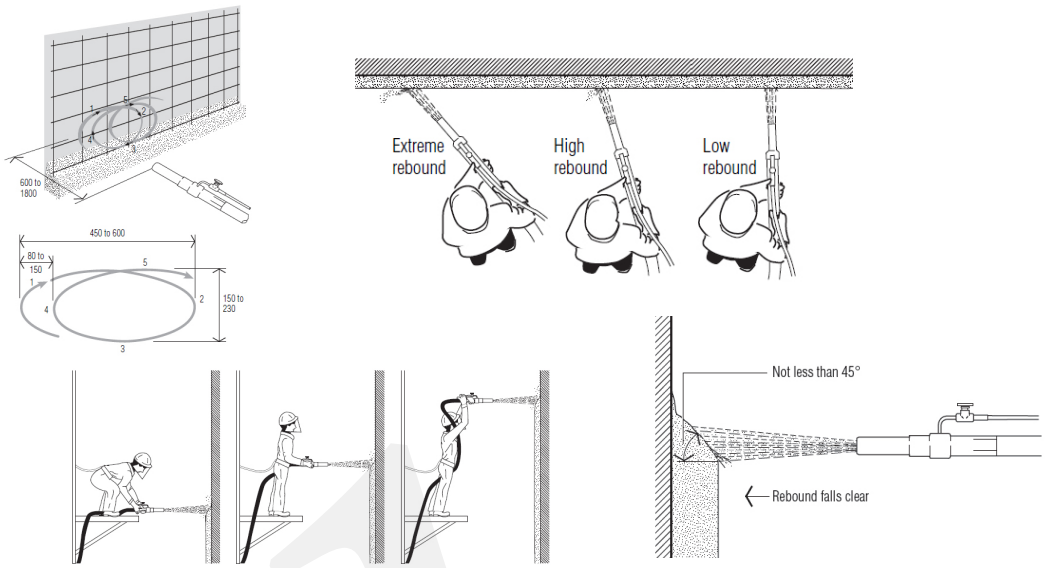



Fig. 10-22 Spraying (shotcreting) process.



Fig. 10-23 Spraying at bottom, mid-height and on inclined surfaces.

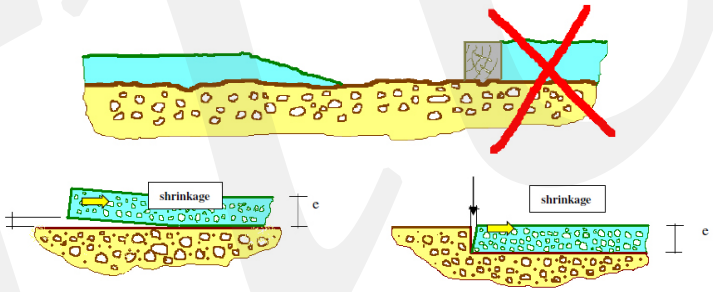



Fig. 10-24 Improper arrangements.

## 10.5.3 Finishing

The projection principle makes it possible to obtain a layer of concrete whose thickness and behaviour vary with the technique used. It is quite common to spray a finishing thin layer (around 1-2 cm) for aesthetic or safety purposes (Figs. 10-25 to 10-27). The following finishing are possible:

- Rough as sprayed
- Scratched
  - Spray evenly a thin topcoat layer of fine sprayed concrete (2 cm)
  - Mark with a ruler the plusses and minuses in relation to the expected profile
  - Fill the minuses by spraying locally and scratch the plusses

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- Ruled evenly
    - Same process as for scratched except rule off the surface at the end.
- Smoothed
    - Same process as for scratched except smooth off the surface with a trowel



Fig. 10-25 Reprofiling of the shotcrete.



Fig. 10-26 Spray of topcoat.







Fig. 10-27 Rough or smooth finishing after UHPFR shotcreting.

## 10.5.4 Curing

In order to keep the concrete moisture for the proper hydration of the 1st few centimetres and to avoid drying shrinkage cracking, the surface must be watered considerably between each layer of sprayed concrete using a low- pressure jet (or sprinkling water or water mist).

- For the intermediate layers: cure should be done two (2) times a day for at least four (4) days or until the next layer is sprayed.
- For the last layer: cure must be carried out during a period adapted to the formula used and to the atmospheric conditions (between 24 and 72h).

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## 10.6 Quality control

### 10.6.1 Quality control of materials

In principle, the shotcrete materials suppliers should therefore maintain a continuous production control system and perform material characterisation tests. Accordingly, all the relevant physical and mechanical characteristics should be provided and, whenever applicable, they should satisfy minimum required performance (see section 2.2 and 4.2).

Properties of hardened concrete according to EN 14487-1 should be considered (see section 4.5). For hardened mortars, at least the following properties, specified in EN 1504-3 (2006), should be provided by the supplier: R4 class, compressive strength at 28 days, modulus of elasticity, adhesion.

During materials reception, marking and labelling of materials should be checked. Visual inspection should also be performed. Materials should be handled, transported and stored according to manufacturers' specifications in order not to cause changes in their properties.

If materials delivered are not certified, test specimens of material should be prepared for laboratory testing.

### 10.6.2 Quality control before intervention

#### 10.6.2.1 Surface inspection & reinstatement

See the details in section 10.5.2.

#### 10.6.2.2 Materials

The materials should be delivered in their original package and stored in a dry place, protected from the sun light and from freezing, at temperatures between 5 ºC and 35 ºC. Packaging should mention the identification of products, namely, the product name, batch number, type of product and expiration date (Fig. 10-28).




Fig. 10-28 Examples of storage of premix materials.

#### 10.6.2.3 Environmental conditions

Prior to the application, temperature and humidity (ambient and concrete) should be monitored.

The use of mock-ups for shotcrete repair can also be required. This ensures that the shotcrete procedure and nozzlemen can achieve fully embedded reinforcing steel. Other parameters such as bond, air content, among others, can be evaluated prior to the approval for repair.

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### 10.6.3 Quality control during intervention

The following environmental conditions should be controlled prior to and during shotcrete execution on site: temperature (within the range of 5 to 35 ºC), relative humidity, dew point, wind and rain.

During execution, quality control consists of checking the layer thickness.

In order to know precisely the layer thickness to be sprayed, marks are placed every 2 metres with the use of a ruler (Fig. 10-29).

- Metal rod fixed in the substrate (apparent length corresponds to the required thickness)
- A rod with a washer placed at the intended thickness value
- Wooden cleats or HDPE pipes (for curved surfaces)



Fig. 10-29 Profile checking: ruler on HDPE pipes, fixing a HDPE pipe.

#### 10.6.3.1 Layer integrity

After concrete spraying, it is mandatory to preserve the adhesion between the substrate and the shotcrete, and to preserve the integrity inside the shotcrete layer. For this reason, no action on the shotcrete layer is possible as long as the concrete has not stiffened (Fig. 10-30).

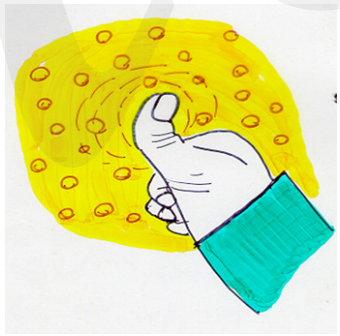


Fig. 10-30 No action on shotcrete layer as long as the layer is flexible.

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### 10.6.4 Quality control after intervention

After execution, the quality of the execution should be controlled by means of visual inspection and testing (Fig. 10-31). Inspection and test plan can comprise:

- Thickness inspection - gauges number and location to be agreed
- Hammer sounding test - 100% - 1 day after spraying
- Initial strength measurement (on request)
- Compression test on cores -for example 1/50 $m^3$ or 1/500$m^2$- 28 days after spraying
- Adhesion test (number and location to be agreed)

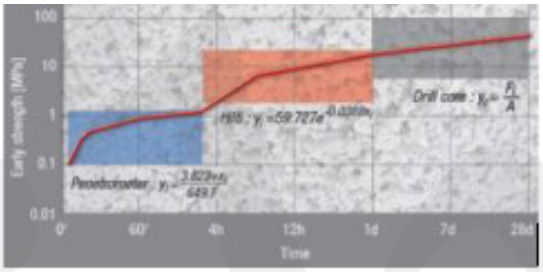



Fig. 10-31 Different methods of strength measurement.

Initial strength measurement: compressive strength of the initial strength development of fresh shotcrete up to 1.5 MPa has to be measured by needle penetration methods. Beyond this strength studs are used which are driven into the specimen surface. The EN 14488-2 "Testing sprayed concrete - Part 2: Compressive strength of young sprayed concrete" describes this procedure.

Penetrometer: in the case of accelerated concrete, the measurement must be made within 30 seconds of spraying (Fig. 10-32).



Fig. 10-32 Penetrometer equipment.

Stud driving method (Hilti): compressive strength between 3 and 20 MPa may be determined by threaded studs, which are driven into the shotcrete surface (Fig. 10-33).

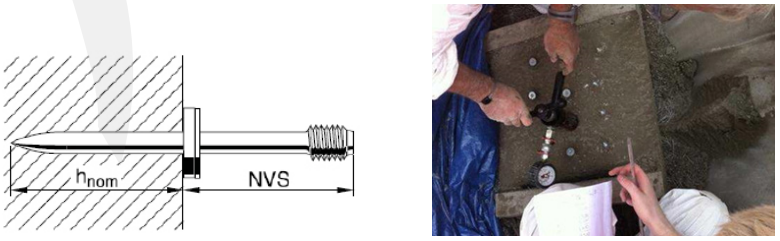



Fig. 10-33 Stud driving method.

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### 10.6.4.1 Compression test

Specimens are carried out on cores or cubes taken from a test panel (boxes)

- Core dimensions : d60 x L120 mm (example)
- Box open on the sides to allow rebounds to escape
- Protect immediately against moisture loss
- Curing shall continue under site conditions for at least 7 days

Any movement and vibration of the samples should be avoided as much as possible during the first 12 hours. Therefore, the specimens have to be handled carefully. Compressive strength depends on the project requirement and shotcrete is selected to meet the specification (Fig. 10-34) (not less than 45 MPa on cylinder typically).



Fig. 10-34 Compression test - Spraying in boxes, sample coring, ends preparation and test.



Fig. 10-35 Coring + pull out test on site.

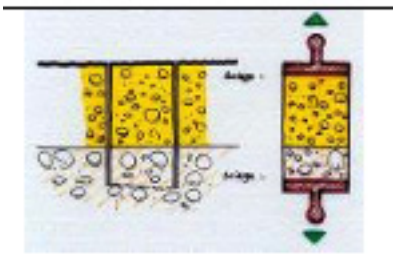


Fig. 10-36 Coring + tensile test at laboratory.

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Bond strength of sprayed concrete to substrate concrete (Figs. 10-35 and 10-36) (measured on the structure):

- Contractual value: at 28 days on at least 3 values
- Typical value: 1MPa at 7 days, 2MPa at 28 days

It is determined during the suitability test, where test specimens are taken from boxes prepared and sprayed on site with the equipment intended to be used for the project (Fig. 10-37).



Fig. 10-37 Evaluation of UHPFR Shotcrete on site (rheology).

## 10.7 Monitoring and maintenance

After execution, an inspection and maintenance plan should be implemented, indicating the periodicity and type of actions to be performed. Depending on the importance of the infrastructure, such plan can include periodic observation and maintenance, comprising (i) visual observation, (ii) tests on the quality of the bonding, and (iii) structural load tests.

## 10.8 Case study

| | |
|---|---|
| Project Title: | Repair of the docks of a shipyard |
| Location: | Estuary of the Sado river, Setúbal, Portugal |
| Client: | Setenave |
| Designer: | A2P CONSULT, Estudos e Projectos, Lda. |
| Constractor: | Jet-Beton, Lda.; STAP, Lda. |

### 10.8.1 Description of the structure

The Mitrena shipyard shown in Fig. 10-38 has three main docks designated as docks 20, 21 and 22. The shipyard was built in 1973/75. In 2000, three more docks were built to increase the shipyard's capacity for ship repair. The repair works analysed in this document are related to the docks 20 and 21.

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Fig. 10-38 Aerial view of the shipyard.

Dock 20 is a platform initially conceived for shipbuilding, consisting of walls with a thickness of 0.70 m at the top and 1.20 m at the bottom, Fig. 10-39. The walls are connected to a bottom slab which has a variable thickness from 1.20 m to 0.60 m. The dock has dimensions in plan of 420 m x 75 m and height of 8.0 m in a first section of 270 m and 10.5 m in the remaining part.

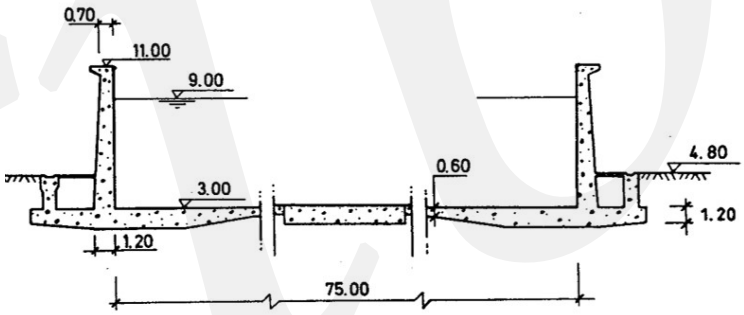



Fig. 10-39 Dock 20: transversal section (dimensions in m).

Dock 21, designed for ship repair, has 18.0 m high side walls, and has dimensions in plan of 450 m x 75 m. The bottom slab has a variable thickness from 1.80 m to 1.10 m, being anchored to the ground by means of cast-in-place reinforced concrete piles. Above the surrounding terrain, the lateral walls have a geometry identical to that of dock 20. In the buried area the structural solution consists of walls 0.30 m thick supported on large buttresses spaced 4.0 m apart, Fig. 10-40.

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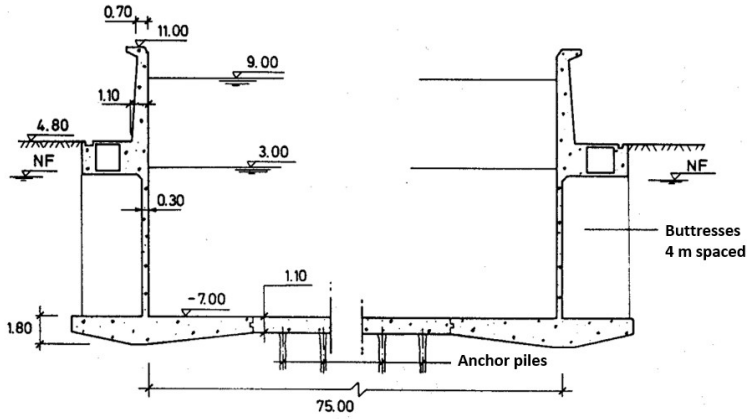



Fig. 10-40 Dock 21: transversal section (dimensions in m).

These two docks are aligned along the longitudinal axis having a total length of 876 m. Dock 21 works as a lock for dock 20.

Dock 22, which is also intended for ship repair, is located next to dock 21 and has dimensions in plan of 55 m x 350 m. The walls of this dock are 12.0 m high, and its top is at ground level.

The docks, built in reinforced concrete, are in use since 1975. The concrete composition had the following main characteristics:

- Cement content: 300 kg/m$^3$
- Water-cement ratio W/C: 0.7
- Cement type: CEM I
- Concrete strength class: $f_{ck,cube}$ = 22.5 MPa

The specifications for the concrete cover were:

- Walls: 4.0 cm
- Bottom slabs: 6.0 cm

## 10.8.2 Inspection highlights

The assessment of the deterioration of the docks showed the main aspects identified below.

The concrete cover measured on the walls showed, in general, values from 2 to 6 cm.

Tests carried out on concrete samples extracted from the walls and bottom slab showed the following properties:

- Characteristic compressive cube strength of concrete:
  walls: $f_{ck}$ = 17.6 MPa
  bottom slabs: $f_{ck}$ = 20.6 MPa
- Water permeability coefficient: walls: $k_w$ = 14 x 10-11 m/s
  bottom slabs: $k_w$ = 0.6 x 10-11 m/s
- Porosity accessible to water: walls: $p$ = 17.8 %
  bottom slabs: $p$ = 16.4 %

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These properties show that the concrete has an inadequate quality for structures exposed to the marine environment. Under these conditions, the chlorides are transported into the concrete at a high rate, leading to a rapid depassivation of the reinforcement.

In structures with low quality concrete, the fast transport mechanisms, permeation and absorption, prevail over the slow transport mechanism, diffusion, and the duration of the initiation phase of the corrosion mechanism is practically negligible.

Fig. 10-41 shows the chloride profiles measured in 1980 (5 to 6 years after construction) on the side walls of dock 20 at different heights. As can be seen, the contamination of concrete by chlorides was very high after a short period of exposure.

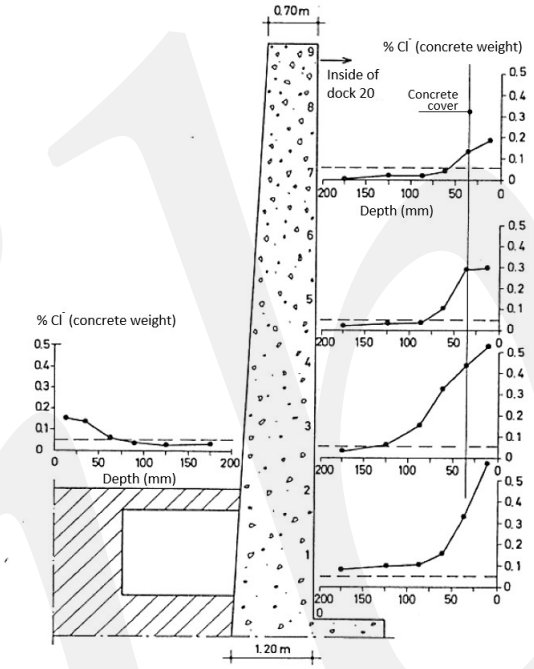



Fig. 10-41 Chloride profiles measured in the walls of dock 20.

Fig. 10-42 shows the chloride profiles measured in 1991 on the side walls of dock 21 at different heights. In these walls the chloride penetration was slower than in the walls of the dock 20.

The progression of the deterioration, the propagation phase, will depend on the rate at which the corrosion of the reinforcement develops. In this case, the main parameter that controls the corrosion rate is the cathodic process, that is, the availability of oxygen at the level of the reinforcement to sustain the cathodic reaction.

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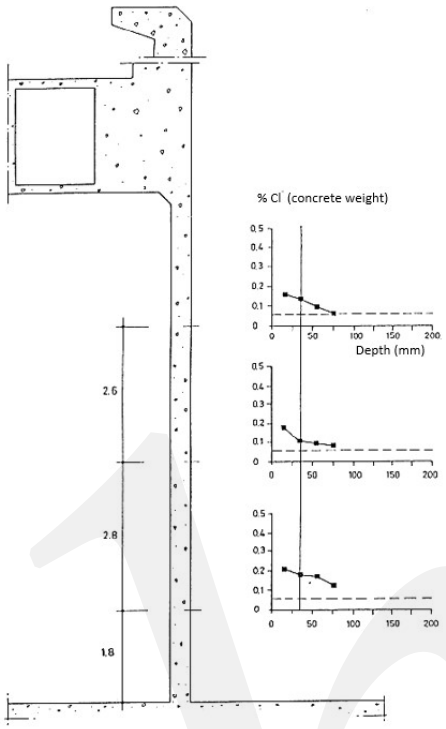



Fig. 10-42 Chloride profiles measured in the walls of dock 21.

The rate of the deterioration observed on the docks walls was determined by this factor. In elements with surfaces subjected to wetting/drying cycles with long intervals, high corrosion rates have developed because of the easy access of oxygen to the reinforcement. In elements with surfaces subject to frequent wetting cycles, deterioration evolved more slowly since the high- water content of the concrete pores restricted oxygen access.

The first situation was observed in the walls of dock 20 and on the upper area of the walls of dock 21. The second situation was observed in the lower area of the walls of dock 21.

At the time, the periodicity of filling the dock 20 was about 6 months while the periodicity of filling of dock 21 was about 2 weeks.

In 1978, three to four years after the completion of the construction, the docks showed clear signs of deterioration with greater relevance in the walls of the dock 20. Fig. 10-43 and Fig. 10-44 illustrate the deterioration of the walls of the dock 20 and 21, respectively, before the 1991/92 repair intervention. As one can see the deterioration of the walls of dock 20 was much higher than the deterioration observed in the walls of dock 21.

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Fig. 10-43 Deterioration of the east wall of dock 20.

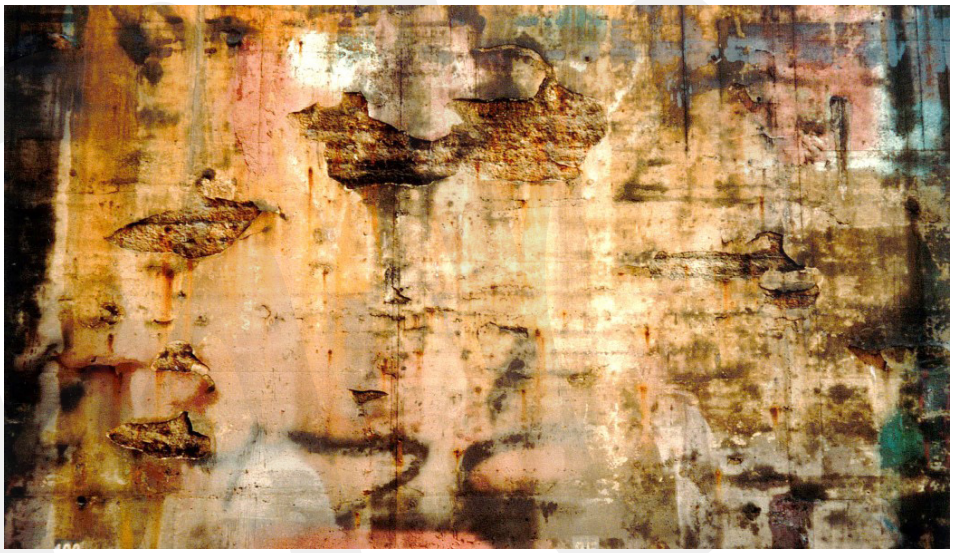


Fig. 10-44 Deterioration of the east wall of dock 21.

In 1984, a first repair intervention was carried out on the walls of dock 20 with an area of 3'600 $m^2$. In 1988, a second repair phase was carried out involving the upper part of dock 21 walls in an extension of 5'600 $m^2$. These two interventions involved different repair methodologies.

At dock 20, a repair was implemented by replacing the concrete surface layer surrounding the reinforcement with a new layer of cast- in-place concrete. At dock 21, a different methodology was used, the concrete surrounding the reinforcement was replaced by shotcrete.

In 1991/92, the third stage of the repair was carried out. It included the interior and exterior faces of the walls of dock 20 and 21, over a total area of circa 25'000 $m^2$. The repair methodology implemented was the replacement of the concrete surface layer surrounding the reinforcement with sprayed concrete using the dry process.

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### 10.8.3 Description of the intervention project (1989/90):

To carry out the repair, different intervention methodologies were considered. In view of the high contamination of the concrete by chlorides, there are few repair alternatives. In this case, solutions involving cathodic protection, or the replacement of the concrete contaminated with chlorides are the most effective. The cathodic protection solution was not accepted by the owner because he argues that it needs to work permanently and involves monitoring the system throughout the remaining service-life of the structure.

For the solution of replacement of the concrete contaminated with chlorides, two alternatives were considered:

Repairing areas with visible signs of corrosion deterioration, replacing the concrete with high chloride contamination, and applying surface protection to prevent chloride penetration.

Overall repair of the structure by fully replacing the concrete surrounding the reinforcement and applying surface protection.

Each of these alternatives involved different costs and offers different durability performance.

The first alternative consisted of repairing areas with concrete delamination, identifying and removing the concrete contaminated by chlorides to the required depth and replacing it with a high- quality concrete. In other areas where the chloride content was not high, a surface coating would be applied to prevent the additional chloride penetration and to avoid the depassivation of the reinforcement.

This alternative was presented from the technical point of view as a solution to consider and the cost would depend on the value taken as an acceptable limit for the concrete chloride content at the level of the reinforcement. This value would influence the durability of this solution.

However, several difficulties were presented:

- The tests carried out on the walls showed that chloride contamination was highly variable, making it difficult to demarcate areas with chloride content above a certain level.
- The need to carry out a large number of tests to determine areas subject to different types of intervention and the low reliability of this method to determine these zones.
- The durability of surface coatings in the docks is questionable, because the walls are subject to wear caused by equipment.
- In view of the difficulty of determiningg limits of areas with high levels of chlorides, the solution would require frequent maintenance as it would be expected that in some areas the corrosion process would develop.

The second alternative included removing all surface concrete and replacing it with new high-quality concrete. This solution was the one that, from the technical point of view, would guarantee greater durability, involving, however, a higher initial cost.

After evaluating the two intervention methodologies, the second one was selected.

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The main requirements defined for concrete were as follows:

- Minimum strength class: C40/50
- Maximum W/C: 0.40
- Cement content (kg/m$^3$): 350 < C < 400
- Maximum water penetration depth (ISO 7031): 10 mm
- Maximum porosity (RILEM recommendation – commission 25PEM): 15%
- Minimum bond strength (pull-off test): 1.0 MPa.
- Concrete curing: 12 days humid cure.

The use of microsilica as a mineral admixture was recommended. Cast-in-place and shotcrete solutions were allowed for the repair.

The shotcrete solution was the one that appeared to be the fastest to execute because it avoids formwork placement.

After analysing the various tender proposals for the repair of the docks, the proposal that presented a shotcrete solution was selected. The dry process was chosen because it offers high compaction and bond to the existing concrete.

### 10.8.4 Description of the intervention works (1991/92)

Before the beginning of the work, studies and tests of concrete compositions were carried out to achieve the requirements specified in the project.

The mixture composition that was used in the repair works had the following main characteristics:

- Cement content: 350 kg
- Microsilica: 15 kg
- Coarse aggregates ($D_{max}$= 9.6mm): 2'003 kg
- (Water: 145 l)

Since the dry process was used, the amount of water could only be controlled indirectly.

It should be noted that the mixture composition that showed the best performance was the one that did not contain fine aggregates.

Results of the concrete properties:

- Characteristic compressive strength of concrete: $f_{ck}$= 57.5 MPa
- Water penetration depth (ISO 7031): < 19 mm
- Porosity (RILEM recommendation – commission 25PEM): 13.2%

Furthers studies for the characterization of the concrete properties showed the following properties:

- Chloride diffusion coef. at 28 days (NT Build 492): D < 8 x $10^{-12}$ m$^2$/s
- Water absorption: < 8 mm after 4 hours
- Distribution of pore size measured by mercury intrusion porosimetry as showed in Fig. 10-45.

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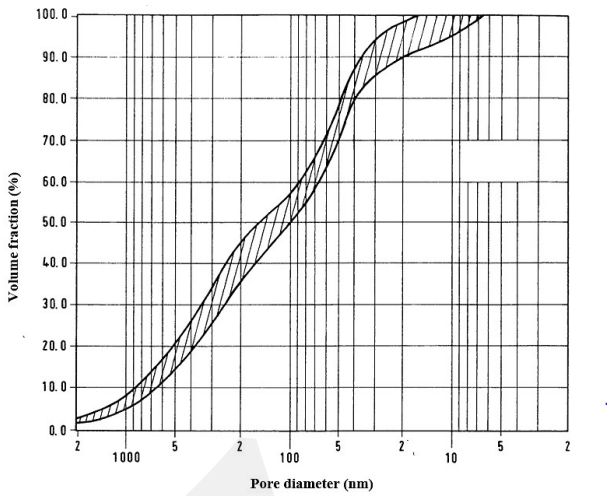



Fig. 10-45 Distribution of pore size measured in several samples.

The repair methodology implemented was as follows:

- Concrete removal to a minimum depth of 10 cm.
- Minimum concrete removal beyond the reinforcement layer: 2 cm.
- Maximum chloride content of the concrete not removed: 1% by weight of cement (0.125% by weight of concrete).
- Cleaning of reinforcement by abrasive blast.
- Replacement or supplementing of bars with a section loss greater than 20%.
- Preparation of concrete surface with projection of wet sand and removal of excess water with air jet.
- Placing the concrete ensuring a minimum cover of 4 cm.
- Wet curing of the concrete for a period of 7 to 14 days depending on the temperature and humidity conditions of the environment.

For the removal of the surface concrete, jack hammers were used because at the time of the repair hydro-demolition equipment was not available, Fig 10-46.



Fig. 10-46 Concrete removal with jack hammers.

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This method of concrete removal would come to be used later in 2000 for the repair of dock 22, Fig.10-47.



Fig. 10-47 Concrete removal using hydro demolition equipment.

The reinforcement cleaning and the preparation of the concrete surface with the projection of wet sand was carried out with the same equipment used for the concrete projection. This method is very effective for the surface cleaning and quick to perform.

Fig. 10-48 shows the reinforcement bars and the concrete surface after surface preparation has been carried out.



Fig. 10-48 Reinforcement bars and concrete surface after preparation.

Fig. 10-49 illustrates the placement of concrete by spraying in dock 20. Fig. 10-50 shows dock 21 after the repair of the walls.



Fig. 10-49 Concrete placement.

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Fig. 10-50 Dock 21 after the repair of the side walls.

The pull-off tests carried out to measure the bond of the shotcrete to the existing concrete showed that the quality of the adhesion was very good. Failure between the two concretes was never obtained in the bond surface. Tensile strength values generally exceeded 1 MPa with all failures occurring in the old concrete.

The quality control of the repair work showed that the requirements defined in the technical specifications were met on site except for the concrete cover in some areas of the dock walls.

### 10.8.5 Monitoring

The effectiveness of the repair of the walls of docks 20 and 21 was monitored over a period of 5 years by measuring the penetration of chlorides in several areas of the dock walls.

Figs 10-51 and 10-52 show the chloride profiles measured in two of those areas of the dock walls 21 and 20, respectively, repaired with shotcrete. One of the zones corresponds to the East wall and the other corresponds to the West wall in each of the docks. It should be noted that, for the repair concrete, the measurements show that the penetration rate is higher in dock 21. The difference, compared to what happened with the original concrete, where the absorption transport mechanism prevailed, is that in the repair concrete penetration is controlled by the diffusion mechanism.

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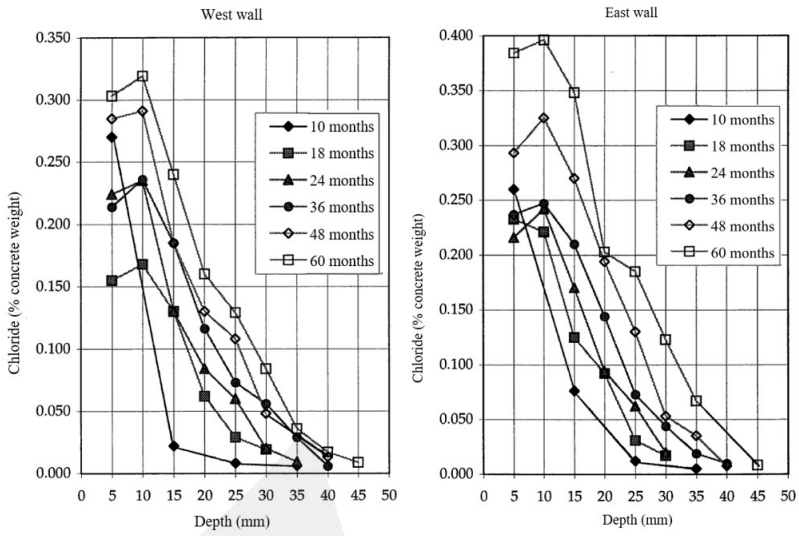



Fig. 10-51 Chloride profiles measured inside dock 21. West and East walls.

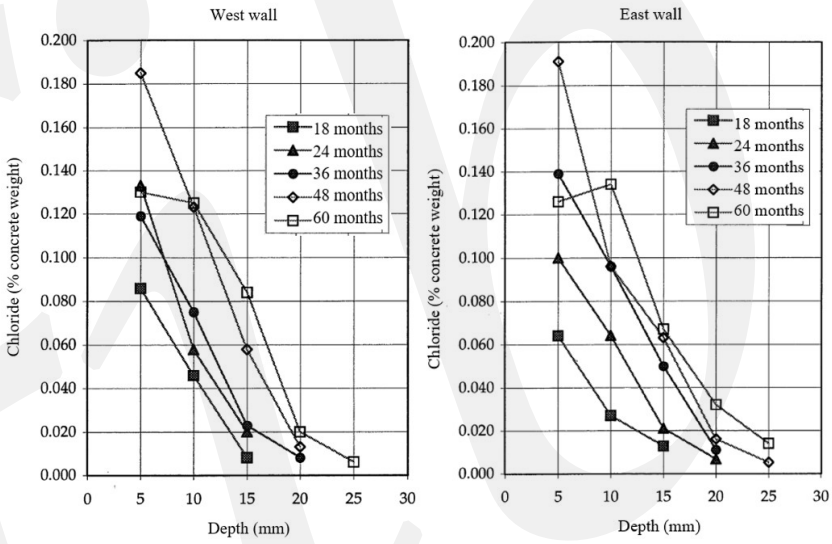



Fig. 10-52 Chloride profiles measured inside dock 20. West and East walls.

From the results of the monitoring, the model of the chloride penetration in the concrete was established for each of the docks based on Fick's 2nd law of diffusion. The defined diffusion model considers the variation in time of the diffusion coefficient for chlorides in concrete and the chloride content on the surface. Based on these variations' laws, the model that defines the penetration of the critical chloride content over time for each exposure situation was defined:

$$x_{cr} = 2\sqrt{D_1 \cdot t^{1-m}} \cdot erf^{-1}\left(1 - \frac{C_{cr}}{C_1 \cdot t^{n}}\right) \qquad 10\text{-}1$$

where

$x_{cr}$ – penetration depth of the critical chloride content after the exposure time $t$

$D_1$ – chloride diffusion coefficient. of concrete at one year of exposure

$t$ – exposure time

$C_{cr}$ – critical chloride content

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$C_1$ – chloride content on the concrete surface after one year of exposure

*m; n* – coefficients that reflect the evolution over time of the diffusion coefficient and the surface chloride content.

Considering the rebound losses of the sprayed concrete, which affects mainly the aggregates, the cement content of the concrete placed in the walls was estimated at about 500 kg/m$^3$. So, for the actual concrete, a critical chloride content of 0.08% of the concrete mass was considered corresponding to a value of 0.4% of the cement mass.

This value can be considered conservative for the repair concrete as it is a high-quality concrete.

From the analysis of the chloride profiles measured on the walls, the following values were obtained for the parameters of the expression indicated above:

- Dock 21: $D_1 = 3.11 \times 10^{-12}$ m$^2$/s; C1 = 0.31 (% concrete weight); $m = 0.59$; $n = 0.33$
- Dock 20: $D_1 = 0.88 \times 10^{-12}$ m$^2$/s; C1 = 0.11%; $m = 0.42$; $n = 0.61$

Fig.s 10-53 and 10-54 show the evolution of the penetration of the critical chloride content over time in the internal faces of the walls of docks 20 and 21.

On the walls of dock 21, the modelling of chloride penetration predicted the depassivation of the reinforcement after 15 years considering 40 mm of concrete cover. This value of the initiation period may be underestimated because, as previously mentioned, the critical chloride content may be higher than that considered in the modelling.

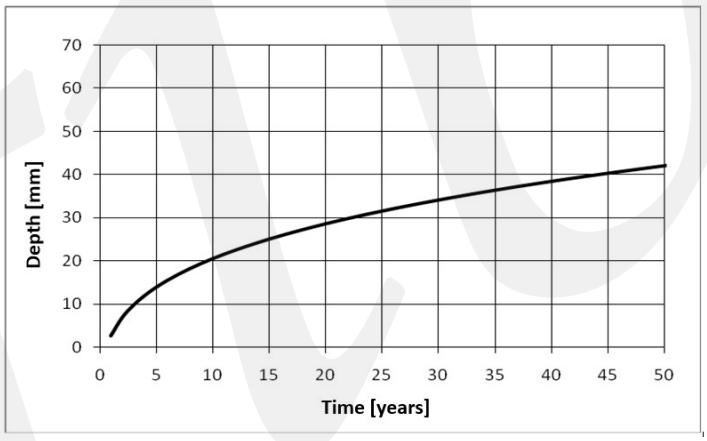



Fig. 10-53 Modelling of the critical chloride penetration in dock 20.

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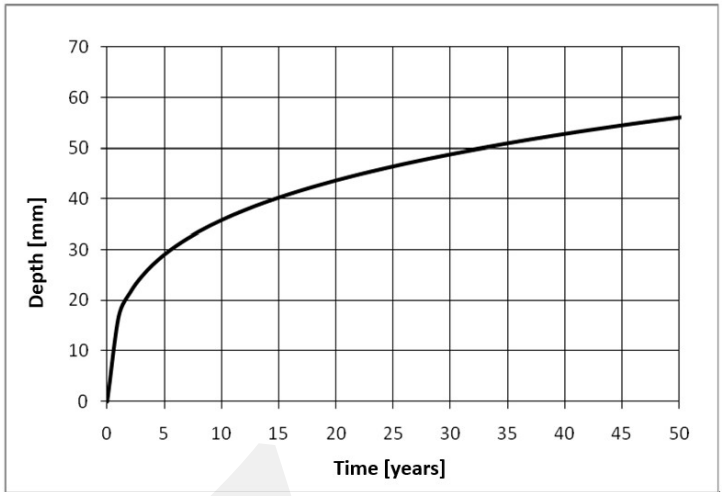



Fig. 10-54 Modelling of the critical chloride penetration in dock 21.

On the walls of dock 20, the modelling predicted the depassivation of the reinforcements after 44 years considering the same concrete cover. These periods of time correspond to the time of initiation of the deterioration process by reinforcement corrosion. The modelling also shows that small variations of the concrete cover correspond to high variations in the initiation period.

### 10.8.6 Description of maintenance operations:

After the repairs in 1990/91, the docks were subjected to other rehabilitation works. In 2009/2010, a new intervention was carried out at docks 20 and 21 involving the repair of the bottom slab of dock 20 and the repair of side walls of docks 20 and 21 in areas previously rehabilitated.

The intervention on the side walls involved the middle strip of the interior faces at dock 21 repaired in 1988 and the interior and exterior faces of the walls of the two docks repaired in 1991/92. Figs 10-55 and 10-56 illustrate the deterioration of the walls, where one can see local areas with concrete delamination.



Fig. 10-55 Dock 21 interior wall– local areas with concrete deterioration.

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Fig. 10-56 Dock 20 exterior wall – local areas with concrete deterioration.

The methodology for wall repair carried out in 2009/10 was identical to that carried out in 1991/92 using shotcrete with the same mixture composition.

The analysis of the behaviour of the walls in relation to deterioration showed. that in the areas affected by delamination due to corrosion, the phenomenon was essentially caused by a low cover of the reinforcement. This execution deficiency was observed mainly on the outer faces of the walls and on the inner face of the dock 20 walls, where in some zones there was a relatively low concrete cover of the order of 15 - 20 mm. For this level of cover, the modelling of the penetration of the critical chloride content defined in Fig. 10-54 indicates periods of initiation of the order of 10 years, a value that is in line with the observed deterioration.

Table 10-2 shows the delaminated areas and the rates of recurrence of deterioration observed on the walls of the docks previously repaired.

Table 10-2 – Delaminated areas and percentage of recurrence.

| Dock 21 | | | |
|---|---|---|---|
| Interior walls | Delaminated area [m$^2$] | Total area [m$^2$] | Recurrence [%] |
| East | 157 | 4'195 | 4 |
| West | 106 | 4'195 | 3 |
| Exterior walls | | | |
| East | 273 | 2'682 | 10 |
| West | 342 | 2'682 | 13 |

| Dock 20 | | | |
|---|---|---|---|
| Interior walls | Delaminated area [m$^2$] | Total area [m$^2$] | Recurrence [%] |
| East | 205 | 2'819 | 7 |
| West | 300 | 2'819 | 11 |
| Exterior walls | | | |
| East | 699 | 2'858 | 24 |
| West | 967 | 2'858 | 34 |

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In the wall areas of dock 21, repaired in 1991/92, i.e.,19 years after the repair, the deterioration is still very low, showing the good performance of the repair carried out. In these walls, there were no deficiencies in the reinforcement concrete cover as it was found in some zones of the dock 20 walls. The observed behaviour is in accordance with the initiation period foreseen by the modelling indicated in Fig.10-55. It is also worth noting that in this condition of exposure, characterised by frequent wetting cycles, the propagation period can be significant as illustrated by the very small area delaminated. In these exposure conditions, the high-water content of the concrete pores controls the corrosion rate by limiting the access of oxygen to the reinforcement.

Inspections carried out in 2020 show that the deterioration of the walls has progressed little after the 2009/10 intervention.

30 years after the repair of the dock walls with shotcrete, despite the concrete cover being 4cm and the very aggressive exposure environment, it can be concluded that the repair method using shotcrete presented a good performance.

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# 11. Repair of prestress tendons through reinjection of grout

## 11.1 Foreword

Installation of bonded tendons is often linked to difficulties, mainly related to cementitious grout injection. There are many cases where grouting was carried out inadequately/inappropriately. The primary causes for inadequate grouting are inexperienced staff and/or poorly defined quality control procedures for the grout injection process.

Problems with the grouting procedures/quality cause the structure's deterioration after several years of service. The presence of voids in the grout facilitates corrosion propagation, which weakens the cross section of the tendon. Consequently, cracks develop due to overall structural weakening and changes to the static behaviour of the structure.

## 11.2 Basics

### 11.2.1 When to adopt this method

Repair of prestressed tendons should be carried out following the detection of the defects in the ducts or deterioration of the tendons. Underestimating the deterioration associated with these defects can result in changes in structural behaviour and, potentially, in structural failure.

*Comments:*

*Before significant structural behaviour changes are perceptible, minor defects may be observed. These are likely to include rust stained water leakage from the structure adjacent to tensioned strands which potentially indicates voids and active corrosion process.*

Where defects associated with damage to post-tensioning are observed, a thorough inspection focussed on the post-tensioning (PT) system is required. This inspection should identify any concerns with the PT systems. Where voids are found adjacent to the strands, and where there is no significant damage to the strand, reinjection of grout may be recommended.

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This chapter was written by Carballo, Chrappa, Laco (main author), Sprinkel, and Zivanovic.

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### 11.2.2 Grout characteristics

Cement grout for injection of prestressing tendons should comply with national standards (e.g. EN 446); key considerations include stability, flowability (fluidity) and chemical composition. The recommended values are:

- Fluidity immediately after mixing: $9s < t_0 < 25s$
- Fluidity 30 minutes after mixing: $1{,}2t_0 \geq t_{30} \geq 0{,}8t_0$
- Volume Change at rest for 24 hours: $-1\% < dV < 5\%$
- Bleeding: <0.3% at 3h, no bleeding occurring at 24h.

The mechanical resistance (compressive strength) to be reached should not be less than 30 N/mm$^2$ at 28 days or 27 N/mm$^2$ at 7 days if it is proposed to estimate the likely 28 days strength at 7 days.

*Comments:*

*It is vital to achieve the resistance identical to that of the concrete structure. The exact values may be prescribed by national standards, annexes or guidelines.*

Grouting of tendon ducts must be done with particular care to ensure they are completely filled, as residual pockets of water may otherwise remain in the ducts, where its expansion upon freezing can cause further cracking in the structure. Therefore, the specification needs to state that no bleeding is acceptable.

Cement used for grout should comprise ordinary Portland cement, where the resistance grade should be defined in the specification and be in accordance with the relevant standards.

Good practice is to minimise water/cement ratio for the grout. Various standards and guidelines recommend that the water-cement (w/c) ratio for cementitious grout should vary between 0.38 to 0.42. However, w/c ratios near the upper boundary may cause deterioration of the grout, primarily due to insufficient swelling of the grouting material during the hydration process. Therefore, it is recommended that the w/c ratio does not exceed 0.4. Water reducing agents may be used to enhance workability and compaction.

Occasionally, cement for prestressing grout is specified in accordance with fib Guide to good practice “Grouting of tendons in prestressed concrete”. However, more commonly specifications are produced in accordance with national standards.

Admixtures should comply with the requirements of national standards.

In order to protect the prestressing steel from corrosion, specific requirements (limitations) regarding chlorides, sulphates and other substances added to the mix water and cement should be defined.

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*Comments:*

*European and PTI (Post-Tensioning Institute, Michigan, USA) specifications are generally consistent with only minor discrepancies in their recommendations regarding maximum levels of corrosive components, e.g. maximum chlorides levels are set out as 0.1% by weight of cement / binder for European standards and 0.08% by PTI. It should however be noted that PTI defines several quality levels for cement grout, which are dependent upon environmental exposure and the batching process employed for the grout (pre-bagged ready mixed or batched on site).*

*Acceptance criteria for volume change are more onerous.*

- *PTI permits a volume change of 0.1% at 24hr; whereas*
- *European Assessment Document (EAD 160027-00-0301) specifies a volume change limit of 0.3%; and*
- *European standards (EN 445) specifies a volume change limit of 1%.*

*The recommended testing procedure for cementitious grout requires a 0.5 m high wick induced tube test, which may be less accurate at highlighting bleeding and volume change.*

### 11.2.3 Techniques

The grouting procedure depends on the tendon layout. The duct fill rate should be between 5m/min and 15m/min.

*Comments:*

*The common practice in Europe for the duct fill rate range is between 4 m/min and 18 m/min.*

The grout vents (Fig. 11-1) should be positioned based on the tendon layout and the nature of the cement grout. Nationally and internationally published documents may specify vent locations. If thixotropic grout or vacuum assisted grouting methods are used, vents are not required.

The aim of the grouting operation is to completely fill the tendon ducts, ensuring the strands to be fully embedded to avoid any attack from the infiltration of corrosive agents. When the attack by an aggressive environment (e.g. high chloride, sulphate or sulphur content) has already started, an intervention through reinjection of grout should prevent further deterioration.

The strands must be located in a passivating environment or in an environment with a low content of aggressive ions ($C^{l-}$, $SO^{2-}$, $S^{2-}$). If the environment contains a significant level of aggressive ions, mitigating measures must be carried out prior to the reinjection.

For bonded prestressing tendons, the hardened cement grout must also demonstrate sufficient mechanical properties to ensure that bonding between the strands and the concrete structure occurs.

The details concerning the final protection of the prestressing strands must be defined at the project design stage in order to determine the position of the upper vents and the lower purging vents.

When the tendons are horizontally marginally deviated, it is necessary to consider vents at least every 40 m.

In the prestressing tendon layouts, presented below, the vent positions are located to avoid the presence of stagnant water in the ducts and the formation of air pockets during the grout injection.

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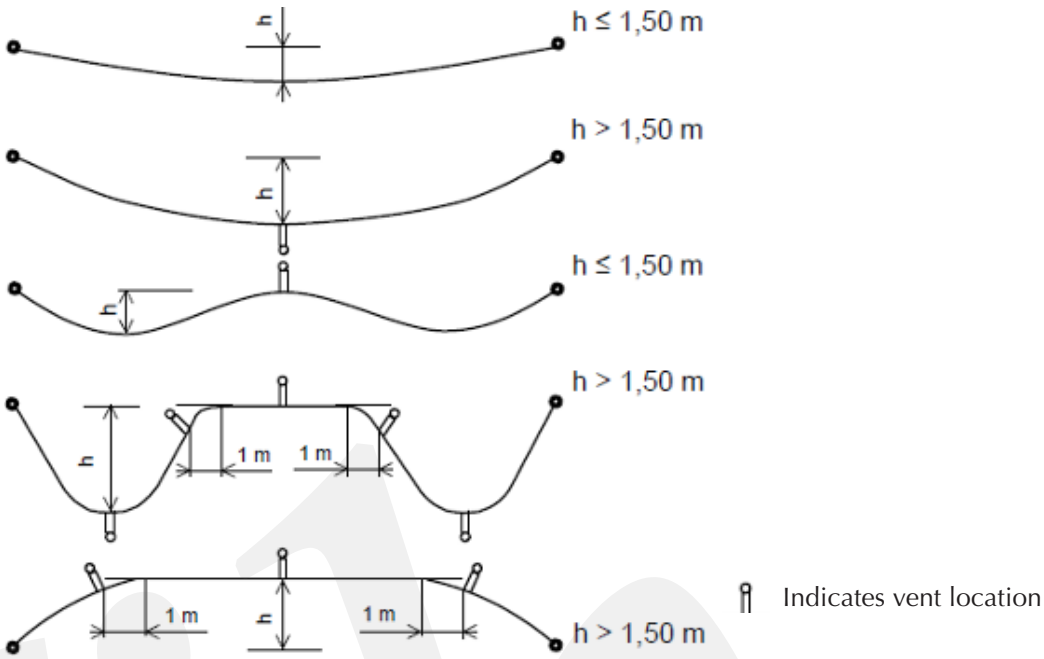



Fig. 11-1 Grout vents location at various geometry.

## 11.2.4 Grout behaviour principles during and after grouting

The grout progression inside a duct with a generally deviated profile was identical to the behaviour achieved in many model tests undertaken at a scale of 1:1. The observations from the test suggest:

- grout progression velocity ranges for each type of tendon.
- the position of nozzles, purging or injection vents to avoid or to eliminate trapped water or air pockets by means of grout re-injection.

Furthermore, it is a well-known fact that water from chemically unbalanced grout tends to gather at the high points of the ducts; thereby constituting a risk for the concrete containing the experiencing bursting forces due to the formation of ice or, alternatively, the creation of a vacuum if water has been reabsorbed by the grout.

Entrapment of air in voids located behind the grout progression front during injection, which happens mainly on the legs of the ducts sloping downward relative the advancing grout front, also constitutes a risk of corrosion.

*Comments:*

*To avoid the grouting defects, special grouts and techniques have been developed for each type of tendon: horizontal, vertical, beams with significant horizontal deviations.*

To achieve adequate grouting of prestressing tendons, it should also be considered that any non-homogeneous grout, in contact with the strands, causes migration of water by capillary flow and filtration through the interstices between the strand wires. The volume of segregated water is proportional to the difference in pressure existing between the upper and lower points of the duct. The most critical case concerns highly deviated or vertical tendons where water accumulates at the upper surface of the grout.

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### 11.2.5 Grouting devices and other required equipment

The required equipment for the execution of vacuum reinjection is as follows:

- Vacuum pump;
- Volume meter;
- Pressure gauge;
- Mixing and injection pump unit;
- Air compressor.

All devices must be certified, maintained and regularly tested.

## 11.3 Stakeholders' roles and qualifications

### 11.3.1 Owner

The structure's owner should provide all relevant available information from previous interventions, assessments, original design project and construction records.

The owner may specify additional requirements for service life of the strengthening elements, minimum or increased structural performance, durability of the strengthening element, warranties, stipulations not to affect the service of the structure during interventions, etc. The owner is also responsible for quality control measures, inspections, monitoring and maintenance.

### 11.3.2 Designer

An overall understanding of the structural behaviour is essential to ensure an adequate design can be carried out and a repair procedure proposed.

A designer should possess an adequate understanding and experience of performing structural assessments of post-tensioned concrete structures and bridges. In particular, the designer must understand the difference between static behaviour of bonded and unbonded tendons.

A designer should be able to develop a structural analysis in combination with section resistance calculations to determine the adequacy of the structure. This should consider bonding conditions and positions of the prestressing.

In a situation where reinjection of the grout cannot solve the structural / durability problem, a combined intervention utilizing reinjection and other repair/strengthening methods, e.g. tendons replacement, can be adopted. This is particularly effective when heavy tendon deterioration is observed. However, the combined intervention requires careful works sequencing to prevent undesired unfavourable / adverse effects on the structure

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### 11.3.3 Contractor

Only a contractor certified to carry out post-tensioning installation works is permitted to make interventions upon a functional prestressing system. The contractor should be aware of the importance of the prestressing system within the structure and its importance to the integrity of the structure.

The contractor should have available all devices required for the tendon repair and is responsible for all aspects of their operability. The contractor is liable for the compliance of materials used with the requirements of the design / specification. The applied materials must comply with the required standards and specifications. Where the applied material does not comply with the design, a material approval request should be issued to the designer.

## 11.4 Design

### 11.4.1 Assessment of existing conditions of grouted tendons

Where there are areas of concern, the scale and severity of the defects should be identified by the engineer in charge, who should also decide on the appropriate repair method.

Existing conditions of the structure must be considered during this process. As part of this process, it is necessary to estimate the severity of the damage and estimate whether permanent structural changes are likely to have occurred. The structure manifests permanent changes through cracking of the affected area, deterioration of other structural elements such as ducts, strands or anchors, and/or through excessive deflections. Once this has been identified, the repair scope and other interventions should then be developed and specified.

Designers should also determine the criteria and set out the parameters for the replacement of a tendon. Replacement will be dependent on the geometry and other conditions of the structure including:

- Extent of grouting in the tendon;
- Accessibility to the affected tendon;
- Presence of additional ducts on the structure;
- Impacts of tendon replacement on the operation of the structure.

### 11.4.2 Service life

The envisaged design service life of the structure is applicable for all non-replaceable structural elements and therefore applies to built-in post-tensioning (PT) systems.

PT systems have been developed over many years to make them reliable and durable. Over time, grouting systems have become better developed and have thorough on-site supervision during construction. Thus, older structures have a greater probability of containing PT ducts which have not been effectively grouted. However, defects in grout in new structures can also be observed.

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The structure should achieve the designed service life without any further significant intervention to the PT elements once successful repair of grouting has been carried out. Achieving the full-service life of the structure is then dependent on regular inspection and maintenance procedures. Where any deterioration of the structure is identified, it should be repaired as required.

### 11.4.3 Reliability requirements

Full reliability level of the structure is envisaged after a successful repair operation. However, in some instances, there will be a decrease in the structure's reliability after repairs to the structure. This may be due to corrosion in a strand which has not been repaired or other damage to the structure which were not rectified during the repair grouting. The design engineer should then be able to determine any possible decrease to the structure's reliability.

### 11.4.4 Relevant references

Relevant documents applicable to post-tensioning system material and testing requirements are listed next.

*Comments:*

*These documents provide recommendations only; contractors and designers should follow national standards for design and execution of repairs where these are applicable.*

- European Committee for Standardization (CEN), "Grout for prestressing tendons – Test methods", EN445, 2007, Brussels, Belgium.
- European Committee for Standardization (CEN), "Grout for prestressing tendons – Grouting procedures", EN446, 2007, Brussels, Belgium.
- European Committee for Standardization (CEN), "Grout for prestressing tendons – Specification for common grout", EN447, 1996, Brussels, Belgium.
- *fib* Bulletin No. 33, *Durability of post-tensioning tendons*, 2005, 79 pp, ISBN 2-88394-073-8.
- *fib* Bulletin No. 20, *Grouting of tendons in prestressed concrete*, 52 pp, 2002, ISBN 978-2-88394-060-4
- European Organization for Technical Assessment (EOTA), "Post-tensioning kits for prestressing of structures." ETAG 013, 2002, Brussels, Belgium.

*Comments:*

*ETAG 013 was superseded by EAD 160004-00-0301 and EAD 160027-00-0301*

- European Assessment Document „Post-tensioning kits for prestressing of structures", EAD 160004-00-0301, 2016, Brussels, Belgium.
- European Assessment Document „Special filling products for post-tensioning kits", EAD 160027-00-0301, 2016, Brussels, Belgium.
- NCHRP 848, Inspection Guidelines for Bridge Post- Tensioning and Stay Cable Systems Using NDE Methods, 2017, 244 pp, ISBN 9780309446327 0309446325, Washington, DC.
- AASHTO Manual for Bridges Evaluation, 3rd edition, 2018, Washington, DC.

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### 11.4.5 Design assumption

Full bond between strands and embedding grout can generally be assumed after a successful grout repair. Full bond allows prestressing strand utilisation up to its yield strength at ULS. The challenge is in estimating the remaining prestressing force within the affected strands.

Where the direct force is not measured on site, the estimate of the prestressing force will be dependent on the predicted structural behaviour from the analysis model, the condition of the structure and the experience of the design engineer. The decrease in the tendon force can be estimated based on the expired service life of the structure taking into account an estimation of the immediate and long-term prestressing losses based on the classification of the installed strand type.

The designer should consider possible cross-section loss due to previously identified pitting corrosion of the strands in the areas affected by voids. The presence of cracking and/or excessive deflections, identified prior to the intervention, will also provide an indication of the stressing state within the strands. Where cracks have developed, it may be necessary to re-assess the structure considering section properties for a cracked cross-section.

### 11.4.6 Design procedure

Where the defects in prestressing ducts are revealed, an assessment of the structural integrity should be carried out. The assessment must initially consider the remaining prestressing force in all tendons. Tendon cross-section loss due to pitting and surface corrosion must then be estimated. Strands weakened by pitting corrosion should not be considered in the cross-section resistance calculations. Pitting corrosion can also be initiated between particular wires in the strands. Therefore, it is prudent that a strand attacked by pitting corrosion is neglected from the resistance calculations. When it is expected that the pitting corrosion is present, but not discovered, the designer should perform a sensitivity study to assess the potential influence of the pitting corrosion in relation to the number of intact strands and then re-evaluate the section resistance.

Croos-section resistance should be calculated allowing for potential cracking in all affected areas and other critical locations. The sections where voids are located within the PT ducts should be calculated assuming that the sections are prestressed with unbonded tendons, rather than assuming fully bonded strands. Where both grouted and non-grouted tendons are located within the same cross-section, a sensitivity analysis should be carried out based on the assumption of both fully bonded and fully unbonded tendons. The most onerous section resistance should be assumed as governing design situation.

Re-assessment of the structure should be carried out with the assumption of successful intervention and fully bonded condition. Strands affected by pitting corrosion and significant cross-section loss due to surface corrosion, which were not replaced, should be excluded from the resistance calculations, even though full strand embedment has been achieved. Once the structural adequacy of the section has been assessed, the void repair method should be developed.

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### 11.4.7 Supporting documents

The following documents should be available for the structural assessment and repair design.

- Historical construction and prestressing records from contractor;
- Previous similar design projects including drawings and calculations;
- Inspection and/or monitoring reports;
- Previous intervention records (repairs, strengthening, assessments);
- Repair technological process report;
- Relevant standards.

## 11.5 Execution

### 11.5.1 Preparatory works

Although different approaches can be adopted towards increasing or reaching the originally designed stressing forces (e.g. addition of external tendons or replacement of strands in ducts with unbonded tendons or in cable stayed structures), it is usually preferred to reinject the grout due to cost efficiency, if the required safety level can be achieved. Common grout injection practice is vacuum grouting.

*Comments:*

*Alternatively, anticorrosive agents and flexible fillers can be used. However, their utilisation is limited due to a lack of thorough research. Guidance provided in EAD 160004-00-0301 and EAD 160027-00- 0301*

Once all voids are identified, any inadequate/delaminated grout must be removed. Hydro-demolition is not generally recommended for the removal of the delaminated grout. Pumping dry air through the tendon can dry the delaminated grout and remove moisture thereby limiting corrosion process.

The most critical point in the intervention operation is air tightness of the system itself along with the tightness of the equipment and its accessories (hoses, valves, fittings). Therefore, multiple tests are required to ensure their adequacy, before the reinjection process can be initiated.

The success of vacuum grouting is dependent on maintaining vacuum in the sheath. The duct must be properly sealed and free from water. External air must be prevented from entering the sheath with the grout.

The following phases of work should be carried out with care:

- Installing and sealing the caps;
- Removing the remaining oil (used for strand temporary protection) in the sheath by blowing air;
- Sealing test with compressed air;
- Vacuum holding test to be done prior to grouting operations.

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### 11.5.2 System trials

Firstly, the tightness of the system and equipment needs to be checked and the condition recorded within the protocol. All places that have been considered as a potential source of leakage should be sealed or insulated. A standard 21 mm plastic ducts should be used for the connections between the valves, air compressor or mixing and grouting pump unit. The joints need to be sealed to avoid leakage of compressed air or injection grout. Any observation from the grouting team should be recorded to retain as much information as possible on the stressing system for use in future repair/maintenance activities.

*Comments:*

*Vacuum holding test*

*The aim of this test is to check the equipment on site. The vacuum pump and its devices are installed at the extremity of the tendon. At the other extremity, a valve is fitted to the cap. Before activating the vacuum pump, the valve on the other extremity must be closed to seal the sheath.*

*When the vacuum is held in the sheath, the rising pressure must be monitored. If fast increase in pressure occurs, the leak must be sealed and the test redone. If the proper holding of the vacuum cannot be achieved, the entire filling procedure of the duct cannot be ensured.*

### 11.5.3 Execution procedure

Where vacuum assisted grouting of prestressing tendons is to be carried out, it is necessary to clean all the affected zone prior to injection to ensure there is no remaining water present and to improve the overall effectiveness of the grouting operations.

The reinjection procedure is then applied in the following stages.

1. Verification of the airtightness of the system using compressed air. Deteriorated materials should be removed and reprofiled, cracks should be repaired using injection with fast set epoxy material.
2. Calculation of the void volume – based on the volume of compressed air.
3. Vacuum reinjection – the volume of supplied grouting material has to be greater than the measured volume of the void during the tests.

The vacuum is applied to the valve fitted at the extremity linked with the pump by sealing the valve. The vacuum pump is activated and the vacuum applied to the system. Once, the vacuum is created in the void, the grouting mortar can be introduced into the void by switching the three-way valve. The grout is then pumped through the grouting hose to avoid capturing air in the hose. When the grout flow through the transparent hose is observed, the vacuum pump is stopped. Then the valve on cap near to the vacuum pump is closed and the pressure in the sheath rises.

*Comments:*

*The recommendation is to mininise the residual pressure as much as possible. If the values of vacuum residual pressure is between 50 to 100 mbar and are stable, it is possible to continue with the injection works. The pressure of the grouting operation should then not exceed a maximum pressure of 15 bar. It is recommended to maintain the grouting pressure for an appropriate time after the voids are filled. The recommended values are 5 - 6 bars to be held for 60 - 90 seconds after completion.*

*Grout production and storage*

*The cement grout is usually produced using a combination of water, cement and specific admixtures. The grout is mixed in a mixing plant and then stored in the remixing plant before injection begins. The total volume of required infill material should include sufficient additional grout which will be retained in the hoses during the grouting operation and also to flush loose original material after grouting operations have been completed.*

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### 11.5.4 Finishing

Once the pressure in the void is maintained at the desired level without significant variation, the injection valves are to be closed to prevent leakage of the injection mortar. The plastic hoses can be removed after 36 hours. At this time, a visual control must be completed by the grouting team and supervisor. This visual control must be recorded in a protocol.

## 11.6 Quality control

### 11.6.1 Quality control of materials

Injection mortar is a homogeneous mixture of cement, water and additives. The injection mortar must comply with national requirements and standards. Every contractor executing this kind of works should use a certified mix design for the injection grout. The mix design should include the type of cement, the water/cement ratio and the definition of the additive(s) used. All the content should be in accordance with a valid declaration of conformity issued by the producer.

### 11.6.2 Quality control during intervention

During the process of grouting, several material and grouting system tests must be executed in accordance with national standards (e.g. EN 445 and 446). Evaluation of these tests is done in accordance with relevant standard (e.g. EN 447). The tests are equivalent to the standard grouting procedures for new tendon grouting and must follow an agreed quality control plan.

Attributes to be tested include, but are not limited to, fluidity, separation, volume change and compressive strength.

## 11.7 Monitoring and maintenance

### 11.7.1 Monitoring

Common monitoring methods involve routine inspections consisting of visual observations and identifying signs of tendon destress. Identified cracks and defects should be documented. Sensors embedded in the structure during construction are not reliable for the whole life of the structure and therefore should not be used as main monitoring system throughout the whole life of the structure.

Sounding of a sample of the duct with a hammer can be adopted as one of the initial investigation methods. This method can locate delaminated grout or concrete in the duct. Sounding is done by striking the duct surface with a steel hammer. The hammer produces a clear ring when used on sound concrete and a dull response on deteriorated grout/concrete. However, this method is ineffective for ducts with flexible fillers.

*Comments:*

*An increasingly popular health monitoring method is acoustic emission monitoring, which is able to detect changes (such as strand failure) in the internal structure of the solid elements. The failure of a tensioned strand results in the rapid release of strain energy held in the strand, and the rapid release of energy creates a shock*

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*wave which propagates through the structure. The released energy is transformed into elastic waves which can be detected by acoustic emission sensors. The frequency of occurrence of such waves indicates how frequently strands are failing. Triangulation techniques allow the location of the strands failures to be determined indicating where material/element damage has occurred.*

Where defects in tendons are identified, a bore scope inspection of the duct's voids can be undertaken to identify corrosion on the strands. Magnetic flux testing can be utilized for strand section loss investigation. However, magnetic flux testing is limited and there are parts of the tendons which cannot be accessed. These are mainly located at deviators and diaphragms. It should be noted that the deviators and diaphragms are locations where voids have a significant probability of occurring, higher than elsewhere along the tendon.

### 11.7.2 Maintenance

The repaired prestressing tendons should be regularly checked and signs of corrosion should be analysed by an experienced engineer. Water ingress into ducts must be prevented. If the prestressing ducts are damaged and tendon strands or grout are intact, a local repair by wrapping is acceptable.

Where voids are identified and considered to be less severe, reliable grouting materials should be considered for the maintenance and repair operations. These include flexible filler utilisation instead of cement grout.

*Comments:*

*The alternative filling materials may have different mechanical properties compared to the original cementitious grout. Hence, their effective mutual interaction can be disputed and needs to be considered carefully.*

### 11.7.3 Post-intervention documentation

The owner of the structure should consider the use of an asset management plan. This should include available monitoring options. Acoustic emission monitoring is considered to be an effective method to highlight early problems in the structure and protect asset performance, particularly where prestressing tendons have been identified as areas of concern.

All other documents from the prestressing tendon repairing operation, including repairing project, design and execution records, monitoring records and inspection reports, should be archived for further reference in the Structure service-life file.

## 11.8 Case study

| | |
|---|---|
| Project Title: | Virginia Department of Transportation. Experience with Grouted Tendons in the Varina Enon Precast Segmental Bridge |
| Location: | Richmond, Virginia |
| Client: | Virginia Department of Transportation (VDOT) |
| Designer: | Figg Bridge Engineers (Bridge Design) |
| Contractor: | S. J. Groves Construction (Bridge Construction) |

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## 11.8.1 Description of the structure

The Varina Enon Bridge (Fig. 11-2) crosses the James River 20 km south of Richmond, Virginia. The bridge consists of parallel precast box girder superstructures which contain 480 tendons with grouted post-tensioned strands as well as 26 stay cables supporting the centre spans from two towers.



Fig. 11-2 Varina Enon Bridge, Richmond, VA.

The bridge features the world's first use of precast concrete delta frames for construction of its 192 m cable-stayed main span. The approach spans, 45.7 m typical length, were constructed span-by- span up to the main pylon, permitting the main span to be constructed in unidirectional cantilever. Precast concrete segments were used for the piers, bridge superstructure, cable towers and delta frames. Its 91.4 m tall single pylons and single plane of stays support the main span with 44.8 m of vertical navigation clearance. Figs. 11-3 through 11-5.

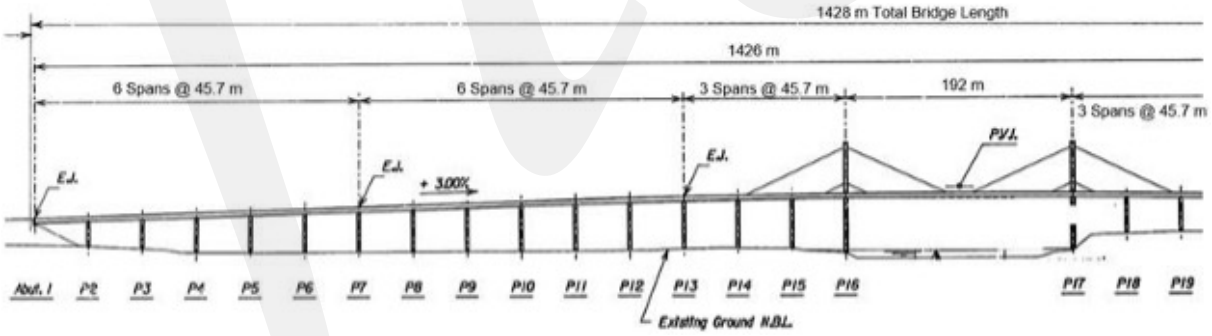



Fig. 11-3 Partial Bridge Elevation (Typical Approach Spans Units and Main Span Unit)

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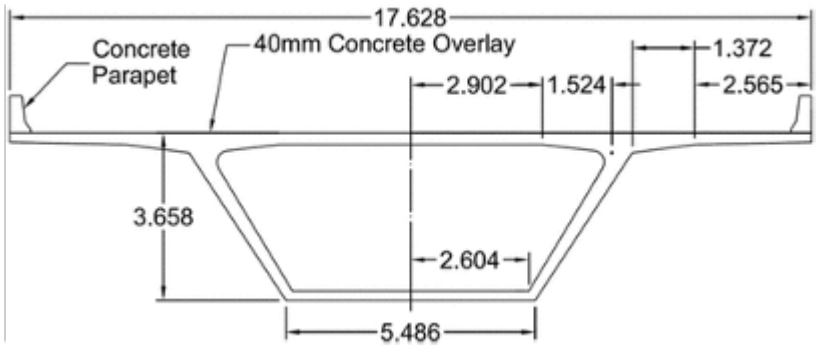



Fig. 11-4 Typical Approach Span Box Girder Dimensions.



Fig. 11-5 Typical Approach Span Internal View.

Completed in 1990 and built using state-of-the-art techniques, it was expected to provide relative maintenance-free performance for more than 100 years.

### 11.8.2 Inspection highlights

The issues that have arisen with the Varina Enon Bridge and other similar structures are complex and there is no simple repair or preservation solution.

Despite the longevity intended by its designers, the corrosion protection of the post-tensioning tendons of the bridge is suspected to have been compromised both as a result of poor grouting practice as well as the formation of voids which typically form at the high ends of draped tendons.

Virginia Department of Transportation (VDOT) has been keeping a close eye on the 28-year-old cable stayed bridge since 2001, when failures in Florida's Sunshine Skyway Bridge, the USA's first major cable-stayed bridge, prompted VDOT to carry out an in-depth inspection of the bridge. In 2001 and 2002 VDOT inspected external tendons for voids in grout, flaws in ducts, and corrosion of strands and anchors (visual, wire probes, sounding, and borescope).

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However, the true scale of the problem first came to light in 2007, when a biennial inspection found an external tendon had failed, raising concerns about the condition of the rest of the tendons and leading to emergency funding from the Federal Highway Administration (FHWA). Fig. 11-6.



Fig. 11-6 Failed Draped Tendon which had been Vacuum Grouted in 2004.

The problem did not come as a complete surprise, as voids in the external tendons had been vacuum grouted and vent tubes sealed in contracts awarded in 2003 and 2004. Borescope inspections in 2001 and 2002 through vent tubes in draped external tendons had revealed voids in the tendons adjacent to the anchor plates. Figs. 11-7 and 11-8. Although traces of grout on the strands and walls of the vent tubes indicated these voids had been filled with grout, bleeding and segregation of the water-cement (w/c < 0.42) used to grout the tendons resulted in voids in the draped tendons. In the 28-span parallel structures the voids typically made up 4% of the length of the draped tendon.

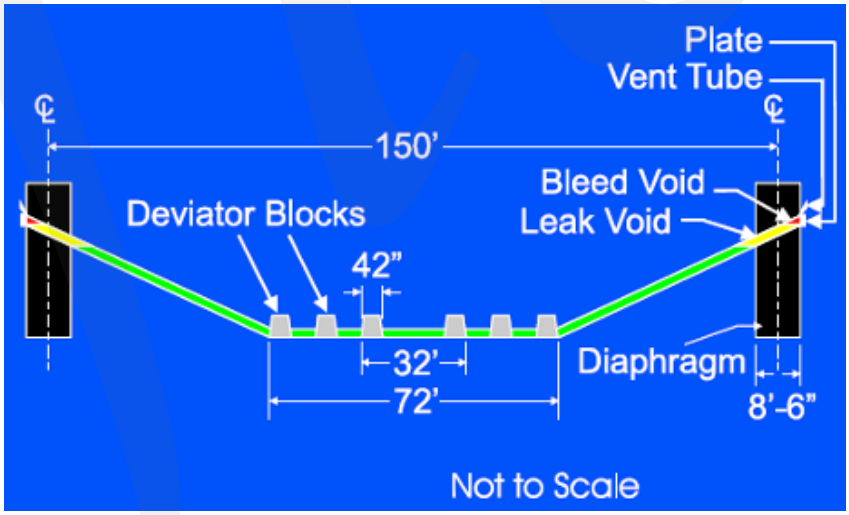



Fig. 11-7 Typical Approach Span External Tendon Layout.

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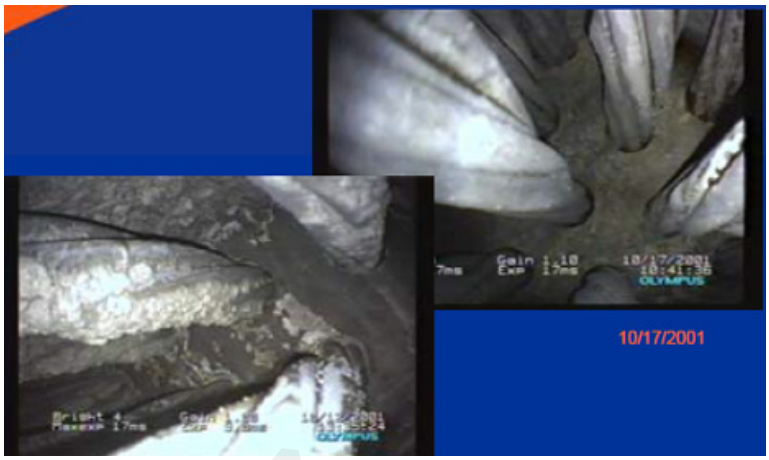



Fig. 11-8 2001 Inspection - Grout Void Adjacent to Anchor Plate.

While properly mixed grout is expected to leave voids at the high points of draped tendons, it was believed grout leaking from the tendons, coupled with incomplete grouting, contributed to additional void length. The grout used for construction of the bridge was ASTM Type II cement and water with a maximum w/c = 0.42 and an expansive admixture, Sika Intraplast N added @ 1% by weight of cement. Higher pumping pressure (allowed at the time of construction of this bridge) led to the separation of fines and bleed water from the grout mass and accumulated at the high points and other susceptible locations along the tendons. Significant changes were incorporated into grouting specifications in early 2001 with non-bleed pre-packaged grout to help prevent this issue.

FHWA emergency funding was used in 2007 to replace the failed external tendon, determine the condition of the other tendons, make needed repairs (including replacing another tendon which had corroding strands), and perform a comparison of the cost and feasibility of the available options for repairing and preserving the tendons.

The strands in the tendon had failed where they were surrounded by the low-quality grout, which had formed at the tendon's highest level, as a result of bleeding and segregation.

Typically, the poorest quality grout surrounds the strands near the anchor plates of draped tendons because these are often the highest point. Strands here are most likely to corrode, not only because of the low quality of the grout, but also because of its high sulphate content, a result of bleeding and segregation. The failed tendon had corroded strands in a 1.8-2.4 m section, with bleed-water grout where the failure occurred; strands in the rest of the 45.7 m length were not corroded.

Inspections in 2001 and 2007 also revealed that strands close to the 2.5 cm-diameter construction inspection holes in the plastic ducts had corroded slightly. Fig. 11-9. In one case, a visual inspection conducted in 2007 had revealed a floor drain plugged with grout that had leaked while a tendon was being grouted, allowing water from a nearby leaking joint to surround the tendon. Since water, acidic from pigeon droppings, would likely cause corrosion, the plastic duct was removed from the submerged section of tendon revealing accelerated corrosion of the strands. The tendon was replaced. As corroded and failed wires could not easily be identified, in the same year magnetic flux testing was used to find section loss in the tendons.

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Fig. 11-9 2001 Inspection – Hole in Duct (Rust on Strand and Void in Duct).

Although magnetic flux testing in 2007 identified section loss in another tendon, it did not find any in the remaining 479 tendons. Unfortunately, magnetic flux testing could not be used where the ducts went through the deviator blocks and end diaphragms, which is where voids are most likely to occur in draped tendons.

Visual inspection in 2007, however, did identify rust stains coming from a construction inspection hole in the duct of one tendon; subsequent removal of the plastic duct revealed two broken wires and a 5.3% section loss.

Twenty tendons were inspected in detail by removal of 0.6 m-long sections of duct; eighteen of these, because some corrosion had been seen in borescope pictures taken in 2003 and 2004; one, because magnetic flux measurements indicated section loss; and, another, because visual inspection of a construction inspection hole had identified rust.

These detailed inspections identified six different conditions associated with corrosion and possible reduced service life. These were: voids, no vacuum grout; voids, incomplete vacuum grout; drying shrinkage cracks in grout; minor corrosion; pitting corrosion and several broken wires; and vacuum-grouted tendon with broken wires.

The inspection also found that corroded strands and those containing broken wires were either not surrounded by grout or surrounded by low-quality grout which had segregated around the strands. No corrosion was found, however, in strands which were surrounded by grout of the quality specified for the construction and which had not been diluted by bleed water. Also, most strands in voids were found to have no corrosion after 11 years.

The grout in the tendons was found to have a high pH value, except in the vicinity of the failed tendon. This grout may have carbonated after tendon failure, as the actual date of the failure is unknown. The grout in two tendons with corroding strands and broken wires had high moisture content, where water in the grout mixture, that had not reacted with the cement, had promoted corrosion.

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### 11.8.3 Description of the intervention project

In 2003 and 2004, VDOT vacuum grouted about 55 percent of the high point voids, the ones in which there was access to the voided area. VDOT also sealed vent tubes and repaired PT ducts and duct/pipe sleeves. In 2007, VDOT replaced two tendons (1 failed, 1 with 3 broken wires).

Replacement of the two tendons was not without its challenges. Access to the inside of the bridges is via a 1 $m^2$ opening in the floor of the spans next to the abutments. The opening is approximately 7 m above the sloping riprap and all repair equipment and repair materials had to be taken through it.

As the failed tendon was already de-tensioned, it was easier to replace than the second tendon, which had to be de-tensioned. The second tendon had an anchor plate at a joint between continuous spans and access was not adequate for easy removal of the anchor plate. The replaced tendons had to be cut into short sections so they could be carried out.

An additional challenge in the implementation of repair, preservation and replacement techniques to address the deficiencies in the tendons is the lack of established recommendations.

Most bridge owners that have found voids in tendons have let contracts to vacuum grout the voids and this happened at Varina Enon. Fig. 11-10. The first contract was completed in just a few weeks because the quantity of void was significantly under estimated; after the second contract, it was estimated that the voids in 55% of tendons had been vacuum grouted, and the remainder could not be accessed because they were inside the 2.6 m end bulkheads.



Fig. 11-10 Vacuum Grouting of External Tendons.

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Vacuum grouting was attempted by the contractor at all locations; however, VDOT was not able to confirm its effectiveness. At some locations it was difficult to pull a vacuum because of leaks at the connections. But the tendon that subsequently failed had been vacuum grouted and the strand failed in a section with low-quality grout lying below the high-performance vacuum grout.

It is unclear whether the vacuum grout increased or decreased the time to failure of the tendon. Some argue that the difference in the properties of the grouts may have been to blame. Others argue that the strands could have been exposed to oxygen and moisture when the vacuum grouting was carried out, which accelerated the corrosion. Still others say that the vacuum grout provided enhanced protection for the strands and that they would have failed with or without the repair because of the corrosive condition of the bleed water grout surrounding them at the location of the failure. The value of vacuum grouting voids is not clear.

In 2015, at a cost of more than US$12’000 per tendon, four tendons were successfully injected with a hydrocarbon and silicon-based corrosion inhibitor, after connections were sealed with an epoxy paste or fibre-reinforced polymer. Air communication/tendon moisture testing was performed after the leaks had been sealed. The volume of material required to impregnate the tendons was much higher than expected and, in fact, significantly more than had been needed for tendons on similar bridges in Florida. In addition, the average relative humidity in the four tendons was almost 90%, again, significantly higher than relative humidity measured in Florida.

At approximately US$250'000 each (in 2007), tendon replacement is an expensive and time-consuming activity, but there are a number of options for Varina Enon.

They could be replaced with tendons containing corrosion- resistant strands such as stainless steel strands or carbon fibre composite cables. The total cost for this is estimated to be more than US$100 million.

Tendons could be dried and injected with corrosion inhibitor at a cost of approximately US$10'000 per tendon and total cost of US$4.8 million, but there could be ongoing costs to replace injected tendons if wires continued to break.

The most economical technology for identifying corrosion and providing information in respect to tendon replacement is acoustic emission monitoring. Each tendon in the Varina Enon Bridge has 19 seven-wire strands, 133 wires per tendon, and the estimated installation cost is US$360'000 to US$720'000 for one or two sensors per span respectively.

Identifying the most cost-effective tendon preservation repair for the Varina Enon Bridge is difficult because of the many variables, as well as the difficulty in quantifying the significance of each variable. However, only two out of 480 tendons have been replaced in 28 years and there have been no tendon failures since 2007. More years of observation will be needed to have a better understanding of the rate of corrosion.

Acoustic emission monitoring and timely replacement of tendons with broken wires is currently considered as the most effective strategy and a contract for the installation of such a system on the Varina Enon Bridge is due to be advertised soon. Acoustic emission monitoring will reveal which tendons need to be replaced by external post tensioning — or some other strategy — before they fail. Traffic congestion caused by the lane closures needed to replace failed tendons will be avoided.

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# 12. Injection to seal cracks in concrete

## 12.1 Forword

The aim of injection sealing of concrete cracks is to prevent the infiltration of substances promoting degradation (such as water and salts) into concrete, by filling cracks with injection materials, and thereby to improve the durability of the affected concrete structure. As one of the interventions to suppress the progression of the deterioration of concrete, injection to seal cracks in concrete can be applied as either a remedial maintenance or a preventive maintenance treatment.

*Comments:*

*Although the causes of cracking are various, cracks become entry points for substances promoting degradation, such as water and salt, which act to accelerate the deterioration of concrete structures. Therefore, injection to seal cracks in concrete prevents, or at least limits, the intrusion of substances which promote deterioration.*

*Injection to seal concrete cracks is a repair method which can be expected to enable some recovery of the durability of a deteriorated concrete structure, avoid corrosion of reinforcing bars, restore the bond strength between the reinforcing bars and concrete and so on. Thus, an improvement in both the structural performance and durability of the cracked concrete structure may be achieved by an effective synergy between the injection material and the cracked concrete providing a recovery of material structural properties and the simultaneous prevention of the ingress of external substances promoting degradation.*

## 12.2 Basics

### 12.2.1 When to adopt this method

The aim of injection for sealing concrete cracks is to prevent the intrusion of substances promoting deterioration. For this reason, it is effective for cases where preventing the entry of moisture (e.g. frost damage, ASR) is expected to slow or suppress the deterioration process.

*Comments:*

*For cracks caused by steel bars corrosion (originated by chloride ingress or carbonation), injection to seal cracks in concrete is not effective. Rust prevention and desalination should be considered.*

*Incidentally, if it is difficult to undertake a crack injection repair due to a high or very high density of cracks, wide cracks and so on, it may be necessary to consider an alternative repair method such as patch-repair.*

### 12.2.2 Materials and systems

Crack injection materials are broadly classified into resin-type and cement-type, and there are various products which have performance (flowability, filling property, and so on) corresponding to the state of the crack to be repaired (crack width, length, depth, wetting, presence or absence of active movement (opening and closing) and so on).

Therefore, when using a crack injection material, it is necessary to confirm whether the performance of the material satisfies the required performance after repair.

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This chapter was written by Kikuta (main author).

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*Comments:*

*Since crack injection materials have a large shrinkage rate during or after curing, they are often applied to a crack width of 1mm or less.*

*In principle, injection to seal concrete cracks is performed in an environment where there is no water supply to cracks to be repaired. If it is unavoidable to perform crack injection in an environment where the crack is supplied with water, injection materials with performance suitable for the environment must be used in order to prevent poor filling.*

#### 12.2.2.1 Resin-type injection material

The resin-type injection material generally used is an epoxy type, but other types, such as acrylic and polyurethane resins, are available. Before curing, viscosity (fluidity), pot life and curing time are indicators of the performance of the resin-type injection material. Whereas after curing, the important parameters are material characteristics (hard type / soft type), dimensional stability (curing shrinkage ratio, coefficient of thermal expansion), tensile strength and bond strength.

*Comments:*

- *Viscosity before curing tends to be affected by the environmental temperature. When this is high, the viscosity decreases (i.e. the material flows more easily) and the curing time becomes shorter. When the temperature is low, the viscosity increases (i.e. the material flows less easily) and the curing time becomes longer.*
- *Generally, resin-type materials are formulated for use in an execution environment of 5°C or higher. Since flowability decreases at low temperature, attention is required for use in a low temperature environment because it adversely affects the ability of the resin to properly fill the crack or void.*
- *Since the viscosity of the injection material and the setting time fluctuate depending on the execution temperature, it is necessary to understand the relationship between the crack width, the crack depth and the change in viscosity, in order to judge whether the properties of a particular injection material will be suitable for the intended crack sealing task / operation.*
- *For narrow cracks, a low viscosity resin is used in consideration of its better crack filling properties.*
- *In low temperature environments it is desirable to use a low viscosity resin, even when the crack width is wide, since resin viscosity increases at low temperatures.*
- *For wide cracks, a medium or higher viscosity resin is used in consideration of their crack filling property.*
- *If a low viscosity resin (more fluid resin) is used for wide cracks, the material tends to flow down inside the cracks, which causes deterioration of repair quality.*
- *Since resin-type materials do not mix with water, it is preferable to apply them to concrete in a dry state because their adhesive (bond) strength tends to decrease when the inside of the crack is wet. However, slight wetting (including moisture contained in concrete) does not adversely affect repair performance.*
- *For cracks which are in a largely wet state, a resin type suitable for underwater curing should be adopted.*
- *In general, when there is concern about the deterioration progression of cracks and active movement (opening and closing) due to load and the like, it is recommended to use a flexible type of resin which can accommodate future movement (opening and closing).*

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#### 12.2.2.2 Cement-type injection material

Cement-type injection materials are divided into:

- an ultrafine particle cement injection material in which ultrafine particle cement and an admixture material are mixed with water alone; and
- an ultrafine particle polymer cement injection material into which a polymer emulsion is mixed for use.

In general, a cement-type injection material has good adhesion even when the crack is in a wet state. It is also excellent in terms of fire resistance and heat resistance.

*Comments:*

- *Since the cement-type injection material cannot be expected to accommodate movement (opening and closing), it is not suitable for cracks where there will be future movement (i.e. an active crack).*
- *When the cement-type injection material is injected while the inside of the crack is dry, moisture is absorbed by the cement inside the crack, which may cause a phenomena (dry-out) of poor filling or poor curing.*
- *In principle, the cement-type injection material should be injected with the inside of the cracks wet to prevent the dry-out phenomenon. Therefore, if the inside of the crack is dry, water is injected in advance to make the inside of the crack wet, and then the injection material is injected.*
- *The shrinkage rate after curing, and adhesion to concrete are also important performance features, and the use of products containing polymers is now common.*
- *In cold snowy climates where frost damage in concrete is likely to occur, cement-type injection material can also experience frost damage, so it is preferable to use a polymer modified type of cement injection material.*
- *As cement-type injection material freezes below 0°C, it is recommended that it is only applied in an execution environment of 5°C or higher, unless special measures are to be employed.*
- *As the required execution environment temperature is 5°C or higher, and that the temperature of the concrete adjoining the crack does not easily rise, when curing in cold weather, it is necessary to control the concrete temperature during execution.*
- *Since cement-type injection material has low viscosity, the injection pressure inside the crack decreases easily, so if the crack width is wide, the injection material may flow out of the crack and produce a defective repair.*
- *Since the material is the same cement-type as concrete, the cracking inside is excellent in terms of density and solidity, but the tensile strength is low and no followability, so it cannot be applied to cracks occurring due to load or similar causes.*
- *For cracks that are constantly supplied with water, an injection material that has the performance to cure quickly should be used.*

### 12.2.3 Techniques & equipment

For the injection for sealing concrete cracks, an appropriate execution method should be selected from either the manual or the mechanical injection method or automatic low-pressure injection method.

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*Comments:*

*There are two types of injection for sealing concrete cracks: the manual or the mechanical injection method which can be injected at high pressure in a short time; and the automatic low-pressure injection method which injects over time with low pressure and low speed. Details on the two kinds of repair methods are provided below.*

1. *Manual or mechanical injection method*
   - *As shown in Fig. 12-1, install the injection tool such as an aluminium pipe into the cracks and use it as an injection port, and inject using a manual pump akin to a grease type pump.*
   - *It is not necessary to make any special tooling, as it is a simple injection method.*
   - *Various instruments, such as simple pipes and valves attached to pipes, dedicated injection instruments using pneumatic tanks and so on can be used for injection ports.*
   - *Injection is performed from one end of the fixed pipe. After checking the outflow of the injection material from the adjacent pipe, move the nozzle of the pump to the pipe from which the injection material flows out and inject more. This is repeated in the same order.*
   - *The mechanical injection method is an execution method injecting at high pressure using a comparatively large injection device, and it is often used for cracks with thick members and large crack depths. This is because high pressure is required to fully inject the crack injection material into cracks with large depth.*
   - *Although the execution procedure is almost the same as the manual type, the difference from manual type is that the injection operation into the pipe use a foot stepping type pump, an electric pump and so on without using human power, execution requires a skilled worker.*

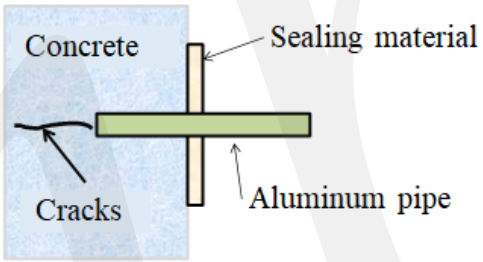
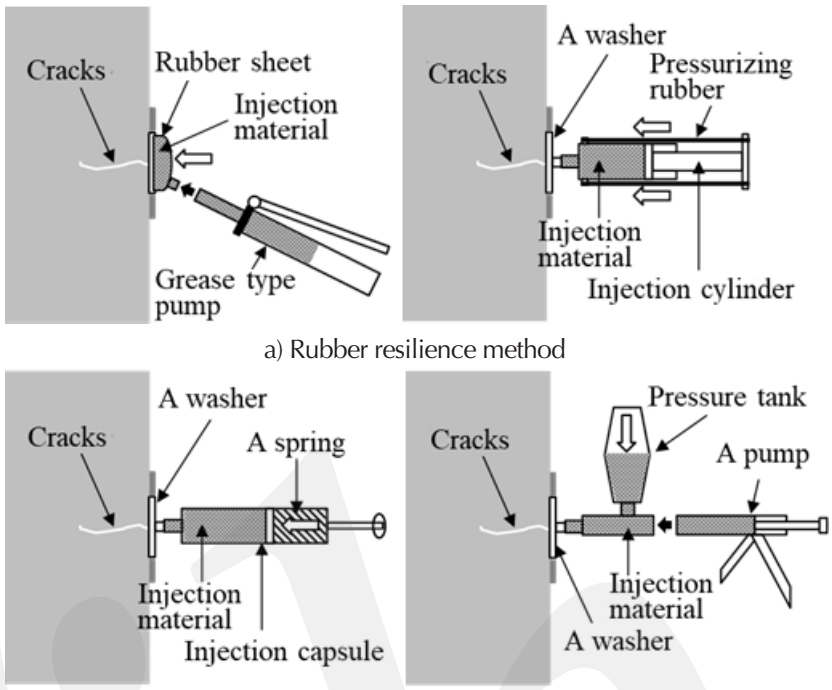



Fig. 12-1 Example of manual injection method.

*Comments:*

2. *Automatic low-pressure injection method*
   - *The injection device is available as various dedicated instruments, as shown in the explanatory Fig. 12-2. There is no major difference in injection performance.*
   - *The injection pressure is defined as pressurization of 0.4 N/mm$^2$ (4 Bar pressure) or less.*
   - *Once the repair material is loaded into the special equipment, no further manual input is then required, and the repair material is automatically injected into the crack by pressurization created by the special equipment. Leakage from the seal is small because it is low pressure. The injection operation is simple and repair material can be reliably and stably injected into cracks, so execution management is easy.*
   - *Since the dedicated equipment that holds the repair materials is formed of transparent material (plastic), the amount of repair material used can easily be seen with the naked eye.*
   - *Since the injection pressure is low pressure, cracking and lifting are not caused by injection process.*
   - *As the repair material is pressurized until it has hardened (fluidity disappears), it penetrates into fine cracks. In addition, voids due to suction do not occur.*
   - *With the automatic type low pressure injection method, the spacing between application (injection) points is 200 to 300mm, which can be wider than the installation interval employed in the manual type / mechanical installation method.*

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a) Rubber resilience method

b) Spring resilience method

c) Compressed air method

Fig. 12-2 Example of automatic low-pressure injection methods.

## 12.3 Stakeholders' roles and qualifications

### 12.3.1 Owner

It is necessary for the owner to formulate maintenance plans for structures and to perform routine inspections, regular inspections and so on based on the plans, and it is important to deal promptly when an abnormality is found.

Injection for sealing concrete cracks assumes/requires that the injection works are carried out before the usability / safety of the structure are significantly affected. For example, in the management of civil engineering road structures, it is necessary to prevent any hindrance to traffic flow or adverse effects upon safety, so it is necessary to implement crack injection in a manner that ensures the safety and security of road users.

*Comments:*

*Although it is difficult to check the internal state of cracks after injection, if leakage and precipitate occurs from the crack injection point after repair, repair effect has not been sufficiently obtained or re-deterioration has occurred. In case of discovering this, re-repair should be carried out.*

### 12.3.2 Designer

In crack repair, there is a possibility that premature re-deterioration may occur unless proper judgment is made on the degree and scale of cracks' deterioration, the cause of cracks and the situation in which the injection material is placed, and proper selection of the process method and material selection are carried out.

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The cracking injection method is a method of restoring the soundness of concrete by filling all cracks with an injection material. Therefore, the designer must specify the types and quantities of injected materials that can fill the target cracks, accurately ascertaining the cause of the cracks, the width, depth, length and progress of cracks. In doing so, the designer develops a repair plan that satisfies the required performance after the repair of the structure requested by the owner.

The designer must understand the methods and equipment which can be used to perform an injection to seal cracks in concrete, along with the different types of injection material, their properties and the precautions which need to be taken at the time of execution, as well as the performance the injected material will give after curing and so on, and formulate a design / specification taking these factors into account.

### 12.3.3 Contractor

After understanding the repair plan presented by the owner and designer, the contractor investigates the cracks to be treated before the remedial works commence and confirms the consistency between the present condition of the crack and the assumed design conditions. If consistency is achieved, injection is performed promptly using leakage-free injection material with appropriate temperature control under the specified conditions. At the time of execution, it is necessary to thoroughly understand the injection method, properties and points to be noted of the injection material, and to ensure a reliable injection is achieved which ensures quality.

### 12.3.4 User

For concrete structures, users are rarely involved in repairing cracks in concrete, but there is a possibility that the adequacy of the structure may be doubted if the concrete appears to be cracked. In addition, there are cases in which visual indications of deterioration, such as water leakage and efflorescence, occur even after the crack repair work is completed. If these adverse / unsightly visual indications are left unchanged, the appearance of the structure will deteriorate and may raise public concerns about the adequacy of the structure. In some cases, based upon the poor visual appearance of the structure, users will report their concerns about the adequacy of the structure direct to the owner. Users may also raise concerns if the structure appears to be experiencing excessive deformation.

Such reports are rare in civil engineering structures but, in the case of buildings, there are many cases where users request the owner to improve the building.

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## 12.4 Design

### 12.4.1 Assessment of existing conditions

In order to obtain necessary information to determine the applicability of injection for sealing cracks in concrete and performing repair design, investigate the items shown in Table 12-1.

Table 12-1 Survey items necessary for the injection to seal cracks in concrete

| | Applicability | Design |
|---|---|---|
| Causes of cracking | ○ | ○ |
| Crack width, length, depth | ○ | ○ |
| Presence of penetrating cracks | | ○ |
| Presence of active movement (opening and closing) | | ○ |
| Presence of moisture supply | | ○ |
| Presence or absence of crack blockage by precipitates | | ○ |
| Application temperature | | ○ |

### 12.4.2 Service life

The injection for sealing concrete cracks achieves a repairing effect by fully filling the cracks in the concrete. Therefore, in the case where the cracks are not fully filled, this potentially allows substances promoting degradation, such as water and salt, to penetrate into the concrete structure. Thus, a defective repair may not improve the durability after repair sufficiently to achieve the service life required for the concrete structure.

### 12.4.3 Reliability requirements

With injection to seal concrete cracks, the reliability level after repair depends on the selection of injection material. Therefore, the selection of the injection material must be made to satisfy the performance required by the owner, take account of the environment in which the structure is located and the state of the cracks.

*Comments:*

*For example, an underwater curing type resin material should be selected for wet cracks / those subject to water leakage flows. Similarly, a flexible type of resin should be selected for cracks where there will be future movement (opening and closing). In the case where injection will be carried out in a low temperature environment, materials with low viscosity (high fluidity) should be selected, and so on for other particular circumstances.*

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### 12.4.4 Codes and standards and other relevants references

- EN 1504-5: 2013, Products and systems for the protection and repair of concrete structures. Definitions, requirements, quality control and evaluation of conformity. Concrete injection
- ISO 16311-4:2014, Maintenance and repair of concrete structures
  Part 4: Execution of repairs and prevention.
- CS, TR69 Repair of concrete structures with reference to BS en 1504, Concrete Society, UK, 2009.
- ACI, 2007, 503.7-07, Specification for Crack Repair by Epoxy Injection, American Concrete Institute, Farmington Hills, Michigan.
- JSCE, 2018, Standard Specifications for Concrete Structures- 2018 Test Methods and Specifications, Japan Society of Civil Engineering, Tokyo, Japan.
- JCI, 2013, Practical Guideline for Investigation, Repair and Strengthening of Cracked Concrete Structures-2013, Japan Concrete Institute, Tokyo, Japan.

### 12.4.5 Design assumptions

The objective for the crack repair method is to suppress the deterioration of concrete by preventing and suppressing the ingress of substances promoting degradation (water, salt and so on) into cracks, also to secure the durability of affected concrete structures by protecting the passive film on embedded steel materials and reinforcing bars, etc.

The performances required for the crack injection material and its main quality standards are shown in Table 12-2.

*Comments:*

*For the injection sealing of cracks in concrete, it is a premise that all the crack injection material is charged and fills the target crack. In those circumstances, it is possible to satisfy the objectives of repair policies (1) and (2) in Table 12-2 by completely filling the crack with the crack injection material. If the amount of crack injection material is insufficient, or if it is not well injected, the cracks will not be completely filled, which may permit the ingress of substances promoting degradation and, in addition, corrosion of the reinforcing steel could occur.*

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Table 12-2 Relationship between repair policy and performance required for crack injection

| Performance required for crack injection material | Main quality standards | Repair policy | |
|---|---|---|---|
| | | (1) Prevention / suppression of the ingress of substances promoting degradation | (2) Suppression of steel corrosion |
| Fillability / Denseness | Viscosity, flowability, shrinkage rate | ○ | ○ |
| Materials sttrength | Compression, tension and flexural | ○ | ○ |
| Followability to crack movement (opening and closing) | Elongation rate | ○ | |
| Adhesion durability | Adhesive strength, adhesion durability | ○ | |
| Materials durability | Resistance to freezing damage and so on*1 | ○ | |

*1: In the case of cement-type injection material

*Comments:*

*In locations where there is a high / very high density of cracks, wide cracks and so on, it may be necessary to consider an alternative repair method, such as undertaking a patch-repair or reconstruction of part of the structural member concerned.*

## 12.4.6 Design procedure

The design procedure for implementing the injection for sealing cracks in concrete is as follows.

### 12.4.6.1 Selection of crack injection material

The selection of the crack injection material should be made so as to satisfy the required performance after the repair of the target concrete structure, grasp the relationship between the crack width and the fluidity and viscosity of the crack injection material, and then select products suitable for the temperature / exposure environment in which the structure is located and the local work (execution) environment.

Selected examples of crack injection materials are shown in Tables 12-3 and 12-4.

*Comments:*

*In the injection sealing of cracks in concrete, it is ideal when the crack injection material fills the full depth of the crack.*

*Because many resin products can tolerate some wetting, it is possible to apply them to concrete in a wet state and with a small degree of water seepage (water flow through the crack).*

*In the case of penetrating (through thickness) cracks, it is important to select a crack injection material which will be able to flow to the back side of the through crack.*

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Table 12-4 Selection example for cement-type crack injection material

| | No polymer | | With polymer | |
|---|---|---|---|---|
| | Non-penetrating cracks | Penetrating cracks | Non-penetrating cracks | Penetrating cracks |
| Surface crack width (mm) | 0.1~5.0 | 0.1~0.5 *1 | 0.1~5.0 | 0.1~1.0 *1 |
| Relationship between crack width and depth | Narrow/deep: ○<br>Wide/deep: × | Narrow/deep: ○<br>Wide/deep: × | Narrow/deep: ○<br>Wide/deep: × | Narrow/deep: ○<br>Wide/deep: × |
| Penetrating crack | ○ | Δ *1 | ○ | Δ *1 |
| Active movement | × | | ○2 | |
| Wet environment | | | | |
| Cold environment | Δ *3 | | ○*3 | |

○: Applicable, Δ: Partly applicable, ×: Not applicable
*1: There is a possibility of flowing down on the discharge surface
*2: Consideration is necessary for application to wet places all the time
*3: In principle it has freezing resistance and application in non-freezing environments

Table 12-3 Selection example for resin-type crack injection material

| | Rigid type | | | Soft type | | |
|---|---|---|---|---|---|---|
| | Ultra-low viscosity | Low viscosity | Medium viscosity | High viscosity | Low viscosity | Medium viscosity | High viscosity |
| Viscosity (mPa·s) | 250 or less<br>e.g. Very thin like oil for delicate machinery | 1'000 or less<br>e.g. Thicker like oilfor automobile engines or similar | 20'000 or less<br>e.g. Gel like consistency | – | 1'000 or less<br>e.g. thicker oil for automobile engine or similar | 20'000 or less<br>e.g. Gel like consistency | – |
| Surface crack width (mm) | 0.1~ | 0.2~1.0 | 0.5~5.0 | 1.0~5.0 | 0.2~1.0 | 0.5~5.0 | 1.0~5.0 |
| Relationship between crack width and depth | Narrow/deep: ○<br>Wide/deep: × | Narrow/deep: ○<br>Wide/deep: Δ | Narrow/deep : Δ<br>Wide/deep : ○ | Narrow/deep : ×<br>Wide/deep : ○ | Narrow/deep : ○<br>Wide/deep : Δ | Narrow/deep : Δ<br>Wide/deep : ○ | Narrow/deep : ×<br>Wide/deep : ○ |
| Penetrating crack | Δ *1 | ○*1 | ○ | ○ | ○*1 | ○ | ○ |
| Active movement | × | | | | Underwater curing type, Δ (For general use) Δ*2 ○ | | |
| Wet environment | | | ○ Underwater curing type, Δ (For general use) Δ*2 ○ | | | | |
| Cold environment | ○ | ○*3 | × | × | ○*3 | × | × |

○: Applicable, Δ: Partly applicable, ×: Not applicable
*1: At high temperature there is a possibility of flowing down on the discharge surface
*2: Injection materials for general use are applicable in case of little wetting
*3: Applicable when the curing conditions are appropriate

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#### 12.4.6.2 Estimation of the injection quantity

In principle, it is necessary to estimate the injection quantity by calculating, using the crack width, the crack length and the crack depth, assuming that the target crack is fully filled with the crack injection material. In the case of penetrating (through thickness) cracks, it is desirable to make the estimate of the injection quantity assuming that the crack depth corresponds to the member thickness.

*Comments:*

*There are ultrasonic methods for measuring crack depth, but it is difficult to reliably measure actual crack depth and, in some cases, the crack is internally branched (ie. breaking into a larger number of finer cracks). Therefore, at the design stage, it is necessary to estimate the injection volume taking account of the state of cracks, deterioration in the concrete and so on. However, since the actual amount of injected material is often large, it is desirable to specify assuming a slight overestimate of the crack depth. Also, regardless of the estimated resin quantity, it may be necessary to change the quantity according to the injection situation. It is important that during execution all the crack voids are filled in order to satisfy the required performance after the repair.*

#### 12.4.6.3 Selection of repair method and equipment

Select an appropriate repair method and equipment considering the depth of cracks, the extent of execution range and the characteristics of the crack injection material to be used, from the methods and equipment shown in Section 12.2.3.

*Comments:*

*The available injection methods are the low-pressure injection method and the high-pressure injection method. Most low-pressure injection methods are automatic, consisting of automatic injection from a syringe-like-cylinder employing the rubber resilience application principle, taking account of pressure change and so on. This automatic low-pressure injection method has recently become mainstream because comparatively it is easy to work and low cost, but a high-pressure injection method is used in cases where the crack depth is large or where a water-stop is necessary. High pressure injection methods are the manual type and the mechanical type, but the mechanical type is often used when the execution range is relatively wide.*

### 12.4.7 Supporting documents

The following documents have to be delivered to confirm that injection to seal cracks in concrete has been properly executed under an appropriate quality control regime using an injection material that satisfies the post-repair performance requirements for the concrete structure.

- Material warranties and material test documents provided by suppliers;
- Material safety data sheet (MSDS) provided by suppliers;
- Records concerning execution management, e.g. the injection amount, moisture, temperature, etc;
- Documents certifying that the industrial wastes generated during the crack repair were properly handled in compliance with the relevant laws and regulations.

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## 12.5 Execution

### 12.5.1 Preparatory works

#### 12.5.1.1 Preliminary condition survey

Before executing the crack repair method selected in the design / specification stage, understand the current condition of the cracks which are to be repaired and confirm the presence or absence of any variations.

*Comments:*

*If time has passed since the investigation of the concrete cracking, there is a possibility that cracking has progressed further. If repair is performed as originally designed in spite of large variations occurring, proper repair will not be possible which raises a concern about further deterioration taking place. When a change is confirmed by the preliminary execution investigation, it should be judged whether the repair method or repair material of the original design can be applied. In the case of large variations from the design assumptions, it is necessary to go back to the design / specification stage and to address the necessary changes in repair methods and materials.*

#### 12.5.1.2 Control of temperature in the execution environment

In principle, in order to install the injection material at an appropriate application temperature, execution at both low temperature and at high temperature should be avoided as much as possible.

*Comments:*

*In tropical and subtropical areas, it is difficult to cool the execution environment, so avoid using delayed set-type injection materials during the day.*

*In snowy cold climates, maintain the application temperature of the injection material by warming and also by heating the execution environment. Also, the temperature of the concrete should be controlled so that the injection material is not adversely influenced during injection by a low temperature of the concrete adjacent the crack.*

*As with the injection, prepare about 30 to 50cc of injection material at a time, leave it in the execution environment and measure the time the material takes to harden. At this time, the temperature change of the injection material should also be measured. In resin-based injection materials, since there is a temperature rise due to the exothermic chemical reaction during curing, it can be used to provide a measure of the progress of curing.*

*Fig. 12-3 is an example of how the locations employed for the investigation of the blocking depth (created by precipitates on the surface of the concrete) can be reused as resin injection points. Since crack voids are found in the cross-cut, it is practicable to inject the crack injection material at this location. It is possible to easily construct crack injection points, even for cracks where the surface precipitate has formed.*

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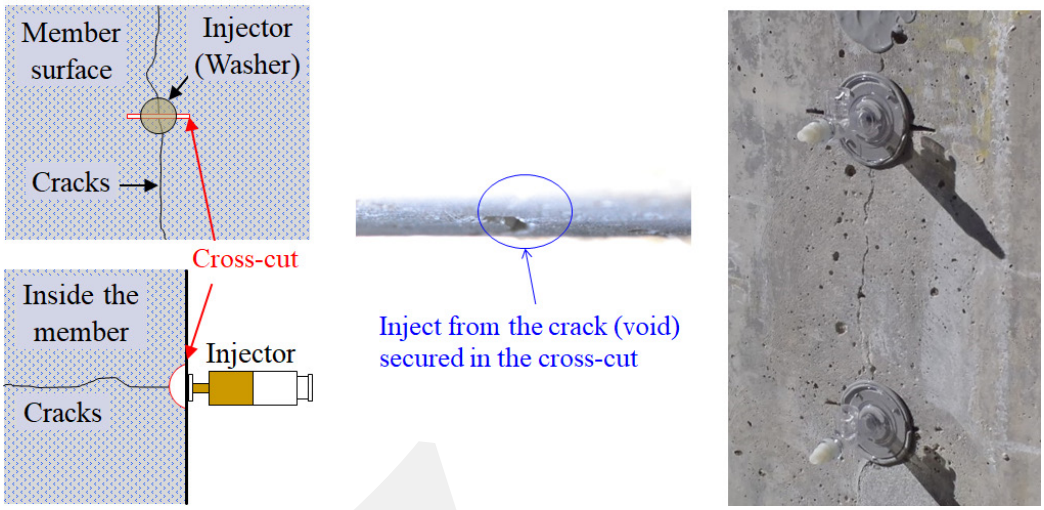



Fig. 12-3 Illustrative example of an injector reusing a cross-cut testing location.

#### 12.5.1.3 Curing time management of injection material

Since the viscosity of the injection material and the setting time change depending on the execution environment temperature and the concrete temperature, it is preferable to measure the curing time of the injection material before undertaking injection in the execution environment.

### 12.5.2 System trials

Regarding cracks with precipitates on the surface, carry out an investigation to enable securing of the injection port by partially removing the blocked portion of the surface and coping with the injection for sealing concrete cracks.

In addition to a general core sampling method, the investigation of the depth of blockage of the surface cracks by precipitates (e.g. calcium carbonate) involves, after removal of any deposits on the concrete surface, cutting incisions in the concrete surface, perpendicular to the line of the cracks (hereinafter referred to as cross-cuts) using an electric tool. The inside of the cross-cut is examined using a loupe or close-up digital photograph. There is a method of measuring the blocking depth by checking the voids in the cracks.

In this method, since the investigation site can be used as an injection port after the investigation, it is advisable to define the investigation positions considering the required injection port spacing.

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### 12.5.3 Execution procedure

The execution procedure is slightly different depending on the type of injection method selected, but the common standard execution procedure is as follows.

1. Cleaning the crack surface (pre-treatment);
2. Determination of injection inlet positions;
3. Installation of injection instruments and the like;
4. Application of crack sealant;
5. Confirmation of curing of injection material and the like and sealing material;
6. Agitation of crack filler;
7. Injection work;
8. Confirmation of curing of the crack injection material;
9. Removal of injection instruments and sealing materials;
10. Surface finish (as needed) and clean up.

### 12.5.4 Finishing

If necessary, polish the cracked part and perform a surface finishing operation to make the crack repair inconspicuous. In addition, in order to block the ingress of substances causing deterioration of concrete and steel, it is also possible to apply a surface coating to the entire concrete surface. This is performed after crack injection. These actions are often carried out in buildings to improve their appearance.

## 12.6 Quality control

### 12.6.1 Quality control of materials

Many of the commonly used crack injection materials are made to satisfy past standards, so it is necessary to confirm that they meet the current performance criteria before use.

Also, keep the crack injection material away from direct sunlight, fire, moisture, water and so on, as specified by the manufacturer.

### 12.6.2 Quality control before intervention

Confirm that the execution environment conforms to the design assumptions / intentions and manage the listed items to ensure consistency in the quality of the execution method and materials as specified in the design.

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*Comments:*

*Examples of main inspection items at the time of execution of the injection to seal cracks in concrete are shown below.*

1. *Concrete surface temperature;*
2. *Execution environment temperature;*
3. *Temperature of crack restoration material;*
4. *Surface moisture content of the concrete substrate.*

*Since the flow performance, viscosity and curing time of crack injection materials vary with temperature, the control of temperature during execution is very important. As well as controlling the temperature of the crack injection material itself, it is also important that the working environment temperature affecting the performance of the crack injection material and the temperature of the substrate concrete are also controlled at the time of execution.*

### 12.6.3 Quality control during intervention

As it is difficult to check the condition of the material injected to fill cracks in concrete after execution, the quality of the outcome of the operation has to be ensured by performing detailed and rigorous inspections in advance of undertaking the execution works. These inspections need to check the suitability of the arrangements made to perform the injection and also the nature / suitability of the crack injection material.

*Comments:*

*Examples of the main inspection items at the time of execution of the injection to seal cracks in concrete are shown below.*

1. *Confirmation of sealing material formulation and effect;*
2. *Confirmation of injector fixing;*
3. *Mixed amount of crack injection material;*
4. *Confirmation of filling by crack injection material, optional;*
5. *Curing time and effect confirmation of crack injection material.*

*Curing time management is important because the curing time of the resin-based injection material varies greatly depending on the temperature of the work environment.*

*For the automatic low-pressure injection method, the time available to undertake the injection and fill the crack to the required depth can be estimated from the curing time of the injection material. Also, after confirming that the flow of the crack injection material has stopped and hardened, the timing of removal of the injector pipes can also be decided*

### 12.6.4 Quality control after intervention

Quality may be judged by the amount of the crack injection material actually injected and the workmanship achieved after removing the seal.

*Comments:*

*The amount of the crack injection material required varies depending on crack condition and environmental conditions. Often, there are differences between estimated required (design) quantity and quantity of material actually injected. But the state of the crack and filled state of the cracked injection material can be inferred by recording and managing the amount injected at each injector location.*

*For example, when the injected quantity is much larger than the estimated design quantity, it is conceivable that crack injection material continues to discharge from the back of the penetrating (through thickness) crack.*

*When the injected quantity is much smaller than the estimated design quantity, it is conceivable that the crack injection material is not flowing satisfactorily (i.e. it is stagnant). As a countermeasure, it is possible to change the viscosity of the injection material or the injection pressure.*

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# 12.7 Monitoring and maintenance

## 12.7.1 Monitoring

### 12.7.1.1 General

To properly maintain and manage the concrete structure after the crack restoration work has been completed, a maintenance plan will need to be devised. Inspections should be carried out based on this plan, the soundness of the restored structure will be confirmed from the inspection results.

*Comments:*

*After execution, it is advisable to regularly inspect the structure and to take appropriate measures as soon as possible, if a change is found. Deformations not dealt with often progress and deform further, meaning that not only is the required performance after repair unsatisfied but also that the situation may be worse than before repair. Therefore, it is very important to periodically check the status of repair spots to promote maintenance work thereafter. Also, when a change is found early and countermeasures are taken, it may be sufficient to take relatively minor countermeasures, which is beneficial from the aspect of maintenance cost.*

### 12.7.1.2 Frequency of inspection

Regular inspections need to be carried out after execution of the crack restoration works. The first regular inspection will be conducted about one year after repair execution and will then be carried out at an appropriate frequency.

*Comments:*

*Changes which occur soon after cracks are repaired often occur in the first year after the completion of crack repair. In this case, as shown in Fig. 12-4, the most common variants are recurrence of leakage and precipitates from cracks, being many of them due to moisture flow / leakage through the cracks. Such early recurrence s are often seen when it was not possible to take measures to block moisture entering the cracks, or when it could not be improved by the crack repair method. Therefore, it is necessary to observe with special attention in routine inspection for one year from the completion of crack repair and to perform the first regular inspection in about one year later.*



Fig. 12-4 Leakage and precipitates 10 months after crack injection.

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*Comments:*

*Also, in cold snowy climates, changes often occur in the spring after the first winter season after repair execution. Therefore, it is desirable to perform routine inspections at this time emphatically and to judge the timing of initial inspection as necessary.*

#### 12.7.1.3 Inspection item and methods

- The check items and methods to be carried out after execution of the crack repair method should be set appropriately according to the type of concrete structure.
- The inspection method should be non-destructive, such as close visual inspection, crack width measurement, hammering test and so on.

*Comments:*

*In injection to seal cracks in concrete, examples of inspection items and inspection methods that focus on the state of surface crack repairs are shown in the Table 12-5.*

Table 12-5 Inspection item and method after execution of the injection to seal cracks in concrete

| Inspection items (Points to notice) | Inspection method |
|---|---|
| Surface crack width | Visual inspection, crack gauge |
| Water leak | Visual inspection |
| Precipitate | Visual inspection |
| Rust staining | Visual inspection |

### 12.7.2 Maintenance

Evaluate and judge the inspection results, understand the situation after repair, and reflect it in the maintenance plan. Also, consider what countermeasures to take if necessary.

*Comments:*

*When a change is confirmed by examination and it is judged that detailed inspection is necessary, this should be carried out. In the detailed inspection, it is necessary to examine the items carried out in routine inspections and regular inspections in more detail and, in some cases, to consider implementation of destructive tests such as core collection. Especially when water leakage or precipitate recurs, meaning that the performance recovered by repair has deteriorated again, it is desirable to confirm immediately.*

### 12.7.3 Post-intervention documentation

#### 12.7.3.1 Execution record

Record the repaired cracked spot and cracked condition (crack width, crack length, cause of deterioration), the method used, type and brand of crack injection material used, design quantity and run quantity, execution temperature and execution time.

*Comments:*

*Regarding the condition before the crack is repaired, in addition to the location of the crack, the crack width, the crack length, together with the repair drawing, record the cause of the cracking and deterioration factors together.*

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*In addition, the execution method (high pressure, low pressure, execution method name), the type of crack injection material used and the brands (resin-type, cement-type, material maker, product name), the design quantity and the quantity executed, the concrete surface temperature at the time of execution, execution environment temperature, outside air temperature, weather, hardening time of crack injection material are recorded.*

*These are items for checking the quality after repair, and also serve as countermeasure intelligence in the event of a change occurring before the next inspection of cracks.*

#### 12.7.3.2 Recording of examination

The inspection results are important information for maintenance management after execution, so everything is recorded regardless of pass / fail judgment and kept for the entire service period of the structure.

*Comments:*

*Records made at the time of restoration contain very important information for maintenance management conducted after crack repair, so it is necessary to keep all of them. Negative results of the pass / fail judgment, and the results of the inspection after the corrective measures are also recorded. Since there is a possibility that the repair may be carried out many times, the inspection record is always saved for the entire service period and it should be used as reference information for the next repair. In particular, since injection to seal cracks in concrete becomes invisible after installation, it is important to keep much inspection information.*

#### 12.7.3.3 Recording of inspection results

Record the results of inspections in an appropriate way and keep them for the entire period of structure use.

*Comments:*

*The recording of inspection results is very important for maintenance management, so it is necessary to record in an appropriate way according to the type and function of the concrete structure and to preserve the results for the entire period of structure use.*

## 12.8 Case study

| | |
|---|---|
| Project Title: | Repair of cracks with efflorescence |
| Location: | Hokkaido, Japan |
| Client: | Hokkaido Regional Development Bureau, Ministry of Land, Infrastructure, Transport and Tourism |
| Designer: | not available |
| Contractor: | not available |

### 12.8.1 Description of the structure:

The structure (Fig. 12-5) to which crack injection was applied is an inverted "T" abutment on a national highway bridge with a length of 755 m that had been built on the plains in a cold, snowy region 24 years ago.

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Fig. 12-5 The structure to which crack injection was applied.

## 12.8.2 Inspection highlights

### 12.8.2.1 Overview of investigation

The abutment of this bridge had cracks which had been generated by thermal stress at the time of construction. Since the surface crack widths were within the maximum permissible value (no more than 0.2 mm), the abutment was subject to extensive observation instead of immediate repair. Periodic inspection every 5 years after bridge completion revealed no significant issues, but efflorescence at cracks was confirmed at a regular inspection 20 years after construction, and the surface crack width was about 0.2 to 0.4 mm, so consideration for repair was judged to be necessary. Detailed investigations were conducted for 2 years in order to confirm the progress of cracking, and repair interventions were considered. Thereafter, to obtain necessary information for repair design, we investigated the state of the cracks (width, depth, whether cracks penetrate, presence of crack opening/closing/deformation) and the state of blockage of the cracked surface by efflorescence.

To investigate cracking progression, the surface crack width was measured with a crack gauge and cracking progression was checked by measurement every year during periodic inspections (Fig. 12-6) In addition, investigation of the state of blockage of the cracked surface by efflorescence was carried out by general core sampling and by a newly proposed method for confirming the blockage depth by making a cut perpendicular to the crack (hereinafter: “a crosscut”) with an electric tool and confirming the blockage of cracks inside the crosscut. Since crack investigation by the crosscut method has been implemented in only a few cases, core sampling investigation was partially conducted in order to compare the crosscut results to the core sampling results. (Fig. 12-7) In addition, since efflorescence from cracks had occurred, we investigated the presence of water supply.

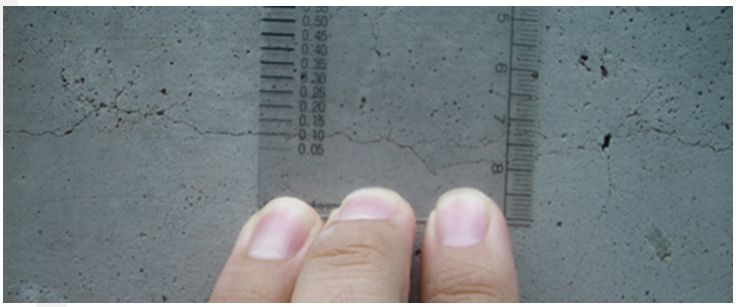


Fig. 12-6 Investigation of surface crack widening (for a crack without efflorescence).

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Investigation by crosscut



The blockage of cracks by efflorescence | Surface efflorescence removal | Cutting at a right angle to the crack (crosscut) | Observe the interior of cracks, and confirm blockage depth

Make an incision (blade thickness: 2 - 3mm; incision depth: 1 - 2 cm) at a right angle to the crack. Using a loupe or a close- up digital photo, visually confirm blockage of the crack inside the incision, and measure the depth of that blockage

Investigation by small-diameter core sampling



Using a loupe or a close-up digital photo, visually confirm blockage of the crack inside the incision and measure the blockage depth.

Fig. 12-7 Investigation of crack blockage depth due to efflorescence.

### 12.8.2.2 Investigation results

Two years of thorough crack width measurement found no significant changes in any cracks, but since leakage from cracks was confirmed at the time of rainfall, the cracks were expected to gradually open in the future.

Because the crack widths had stopped increasing shortly after the abutment was built and because there was no scaling of the concrete surface that would have been seen with frost damage deterioration nor was there the expansion cracking that would have been seen with ASR, it is considered that the crack progress factor confirmed 20 years after abutment completion was the freeze-thaw of moisture supplied from the back side of the penetrating cracks, which gradually caused the cracks to open further.

In the crack blockage investigation, the results of crack blockage depth determined by crosscut roughly agree with those determined by core sampling (Table 12-6).

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The main points of the investigation results are as follows.

1. The surface crack width was about 0.1 to 0.4 mm, and it is assumed that cracks had penetrated deeply into concrete section, as there was water leakage from the cracks following a period of rain.
2. Since 24 years had elapsed since this structure was completed, the cracks had stopped opening. Also, since it was not a part where deformation or deterioration would occur due to external force, it was presumed that the cracks had progressed due to the influence of moisture supplied to the inside of the crack.
3. Efflorescence was found in the vicinity of the surface, and the crack blockage depth was about 0 to 5 mm.
4. Since rust staining was found neither in the water nor in the efflorescence, it was presumed that the internal reinforcing bars had not yet corroded

Table 12-6 Investigation results of crack blockage due to efflorescence

| Surveypoint | Surfacecrack width (mm) | Maximum thickness of efflorescence(mm) | Blockage depth (mm from the surface) | |
|---|---|---|---|---|
| | | | Core side/ interior | Crosscut |
| 1 | 0.3 | 2 | 0 | - |
| 2 | 0.3 | 1 | 15 | - |
| 3 | 0.3 | 1 | 0 | - |
| 4 | 0.2 | 1 | 0 | - |
| 5 | 0.4 | 1 | 0 | - |
| 6 | 0.2 | 1 | 5 | - |
| 7 | 0.2 | 2 | 0 | - |
| 8 | 0.2 | 10 | 2 | - |
| 9 | 0.2 | 2 | 2 | 0 |
| 10 | 0.2 | 6 | 4 | 4 |
| 11 | 0.2 | 10 | 0 | 0 |
| 12 | 0.2 | 4 | 0 | 0 |
| 13 | 0.2 | 3 | 0 | 0 |
| 14 | 0.1 | 2 | 2 | 2 |

## 12.8.3 Description of the intervention project:

### 12.8.3.1 Design

#### *12.8.3.1.1 Setting of a repair policy and selection of a repair method*

For penetrating cracks that result from thermal stress, the blocking of the penetration of substances promoting deterioration (e.g., the suppression of infiltration by moisture) and the protection of reinforcing bars were set as the objectives of the repair policy, in order to prevent corrosion of the internal reinforcing bars.

Repair methods that can protect against crack-inducing factors of deterioration include surface impregnation, crack coating/filling, and crack injection. However, the objects in this case were penetrating cracks, and moisture penetration from the back was observed

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at the time of rain. Therefore, we applied "injection for sealing concrete cracks," which can protect the inner reinforcing bars by completely filling, not only the cracked surface, but also the full-depth of the crack through to the back surface. The infiltration of rainwater from the back surface during rain can also be prevented by using the crack injection method to completely fill the cracks.

The width of the surface crack that was to be repaired was within the range to which the crack injection method is applicable, and although there were parts where the surface was blocked by efflorescence, it was possible to reuse sites where blockage was investigated by using the crosscut as an inlet.

The repair work was carried out at an ambient temperature of around 10 °C, which is a temperature at which crack injection can be applied. In light of the above, the crack injection method was applied to the target cracks when the remedial measure was decided.

*12.8.3.1.2 Selection of materials*

Based on the status of cracks and the characteristics of each cracked material, the injection material was selected as follows.

- The outside air temperature was about 10 °C, which is within the application temperature of conventional injection materials, so such a material was selected.
- Since the back side of the penetrating cracks could not be sealed, a resin-type injection material was selected because there was the possibility that a cement injection material would have flowed out the back side, resulting in an incomplete filling of the crack.
- Since most of the cracks were 0.2 mm in width, a low-viscosity injection material was selected.
- Because the cracks that were to be repaired were those whose changes had resulted from external loads, an inexpensive rigid material was selected.

In light of the above, we selected a hard, low-viscosity epoxy resin- type injection material that is generally used.

*12.8.3.1.3 Determination of quantity*

The design quantity of injection material is determined based on crack width, length and depth obtained by detailed investigation such that cracks to be repaired are filled completely with the material. Since the target structure in this case had penetration cracks, the design quantity was designed with the crack depth taken to be the thickness of the member to be repaired.

### 12.8.3.2 Execution

*12.8.3.2.1 Preliminary condition survey*

If time passes after investigation and design, there is the possibility of the crack progressing, so it is necessary to confirm any changes by a preliminary condition survey.

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In this case, as a preliminary condition survey, we surveyed blockage by efflorescence and executed repair interventions.

*12.8.3.2.2 Selection of a crack injection method*

Crack injection methods include manual or mechanical injection methods and automatic low-pressure injection methods, and it is necessary to select the appropriate method based on conditions such as the crack situation, the execution environment, the extent of execution, and the characteristics of the cracking injection material.

Although the material thickness (crack depth) of this target structure is relatively large (circa 3.0m), the cracks were thermal stress cracks, the crack length was circa 2m, and the cracks were relatively linear, so we adopted an automatic low-pressure injection method that did not require a skilled worker. Instead, the method used dedicated instruments and easy-to-control injection volumes.

*12.8.3.2.3 Execution situation*

Execution was carried out according to the standard execution procedure shown below.

Standard execution procedure

1. Clean the cracked surface (pre-treatment);
2. Determine an inlet;
3. Install injection instruments and the like;
4. Apply the sealant;
5. Confirm the curing of the injection material and the like and of the sealing material;
6. Agitate the cracked filler;
7. Perform injection work;
8. Confirm the curing of the crack injection material;
9. Remove the injection instruments and sealing materials;
10. Finish the surface (as needed) and clean it up.

When creating injection ports, we started by removing precipitate from the surface of the cracks. The ports were then created by cross-cutting at intervals of about 200-300 mm, which is the standard injection interval for the automatic low-pressure injection method. Those intervals include the places where cross-cutting had been performed at the time of the detailed investigation. (Fig. 12-8.)

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| No. | Efflorescence | Inlet type | Surface crack width (mm) | Injectorinterval (cm) |
|---|---|---|---|---|
| A1 | absent | ordinary | 0.2 | 16 |
| | | | | 24 |
| A2 | absent | ordinary | 0.1 | |
| | | | | 25 |
| A3 | absent | ordinary | 0.3 | |
| | | | | 23 |
| A4 | absent | ordinary | 0.2 | |
| | | | | 25 |
| A5 | absent | ordinary | 0.2 | |
| | | | | 22 |
| A6 | present | crosscut | 0.2 | |
| | | | | 17 |
| A7 | present | crosscut | 0.3 | |
| | | | | 18 |
| A8 | present | crosscut | 0.1 | |
| | | | | 18 |
| A9 | present | small-diametercore | 0.05 | |
| | | | | 12 |

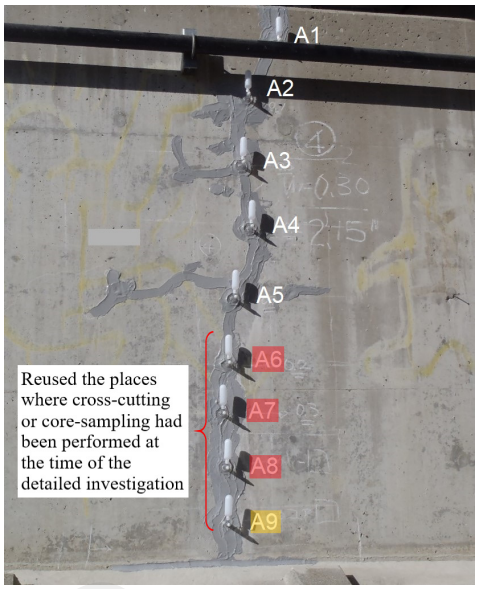



Fig. 12-8 Example of injection situation.

#### *12.8.3.2.4 Duality control*

Since water leakage occurred from cracks during rainfall, crack injection was carried out in fine weather (forecast of clear skies continuing).

In addition, to ensure the quality of the method and the materials determined in the design, the following items were managed.

- Concrete surface temperature;
- Execution environment temperature;
- Temperature before the mixing of the crack injection material;
- Bonding condition of the sealing material and injector;
- Mixed amount of crack injection material;
- Curing time after the mixing of the crack injection material.

#### *12.8.3.2.5 Keeping and disposal of materials*

The resin-type injection material is a combustible substance, so it was stored appropriately in accordance with the Material Safety Data Sheet (MSDS) provided by suppliers and the law. In addition, waste was treated as industrial waste in accordance with related laws, and certificate documents were submitted.

#### *12.8.3.2.6 Inspection*

Since it is difficult to confirm the internal condition of cracks after the crack injection method is executed, quality was secured by ensuring the appropriateness of the execution situation and the run quantity. The details and methods of inspection conducted at the time of execution and completion are shown in Table 12-7.

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Table 12-7 Details and methods of inspection

| Classification | Inspection details | Examination |
|---|---|---|
| Examinations at the time of execution | Materials used | Inspection of quality certificate and exam report provided by suppliers |
| | Concrete surface temperature | Temperature measurement by thermometer |
| | Execution environment temperature | Temperature measurement by thermometer |
| | Finished form and curing confirmation of sealing material | Examination of amount to be used; visual and palpation examination |
| | Confirmation that the injectors were properly mounted | Confirmation by sampling |
| | Mixing amount of crack injection material | Confirmation by weighing |
| | Filling confirmation of crack injection material | Confirmation of discharge of crack injection material |
| | Curing duration and curing confirmation of crack injection material | Confirmation by sampling for each stirring (Fig. 12-9) |
| Examinations on completion | Amount of crack injection material actually injected | Recording of injection amount for each injector, or record of usage |
| | Workmanship after removal of the seal | Visual inspection |

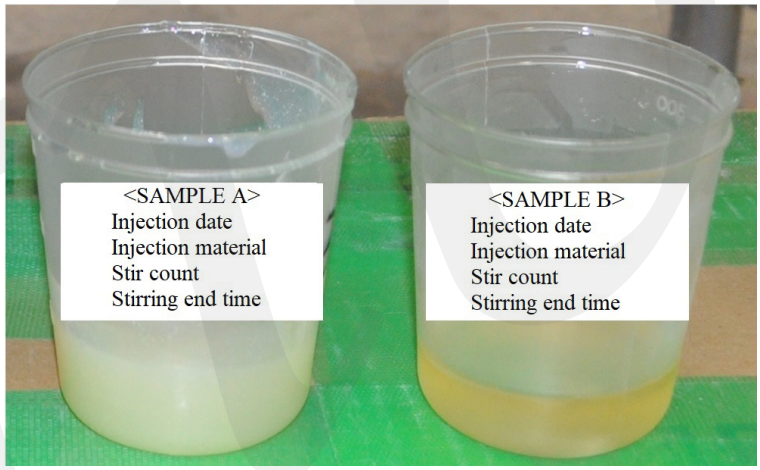



Fig. 12-9 Labelled reference samples for each stirring of injection material, used to check the in-situ curing time.

In addition, since a crack injection method that uses crosscuts had been applied in only a few previous cases, a small-diameter core was sampled at three points (P1, P2 & P3) after injection hardening, and the filling state of the injection material was confirmed. The conditions of small-diameter cores that were sampled are shown in Fig. 12-10, and in the range that can be confirmed visually, the injection material is seen to completely fill the cracks in all the cores.

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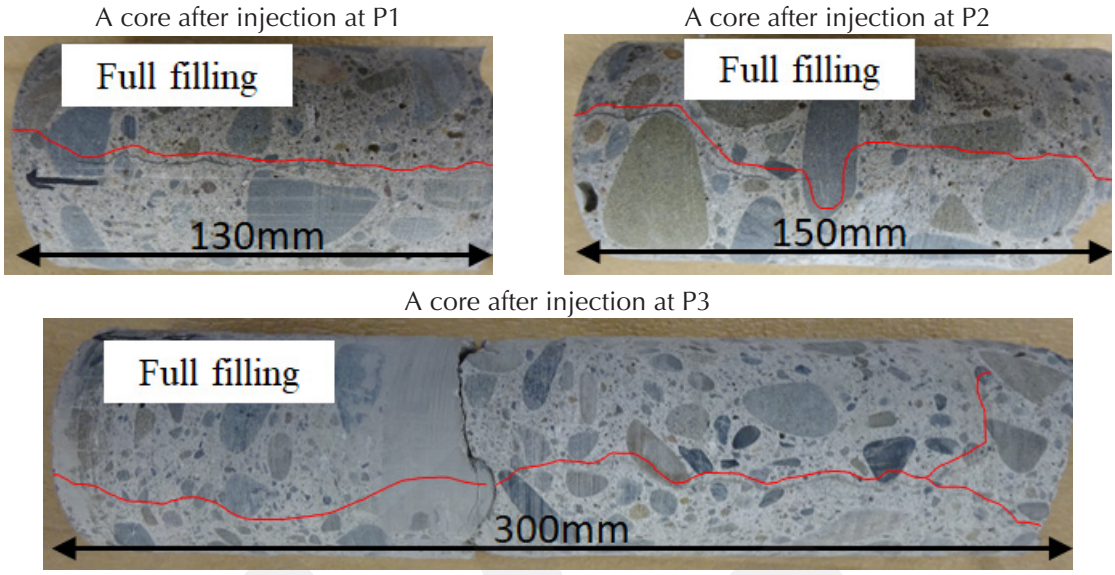



Fig. 12-10 Confirmation of filling of the crack injection material by small-diameter core sampling at Points P1, P2 & P3.

Since the inspection results are important information for maintenance after repair, all of them were recorded and it was decided to keep them during the service period of the structure.

*12.8.3.2.6 Records*

The following items were recorded as data to be used in interventions in the event that subsequent investigation found quality problems after repair or maintenance or that subsequent investigation found other abnormalities.

- The repaired crack location and the crack condition (crack width, crack length, cause of deterioration);
- The method used;
- The type and product name of crack-filling material used;
- The design quantity and run quantity;
- The execution temperature and execution time.

### 12.8.3.3 Investigation

When abnormalities occur early after cracks are repaired, they often occur in the first year after the completion of crack repair. Therefore, we investigated the repair intervention site one year after completion of the repair intervention. The investigation was carried out during rainfall to check for water leakage.

It confirmed that even during rainfall, there was no leakage of water from the cracked portion and that the generation of precipitates and the suppression of water infiltration were sustained.

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