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“Repair and Maintenance of Concrete Structures”

NORECON workshop:

Decision and Requirements
for Repair

3 – 4 April 2003

Lund Technical University
Division of Building Materials
John Ericssons väg 1, Lund,
Sweden

**a Nordic network on concrete research
and development**



Preface

This report presents summaries of 8, out of 20, lectures which will be given at the Norecon workshop in Lund 3rd – 4th April 2003. The workshop program and the list of the participants are also included.

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*Norecon workshop
Lund Sweden*

*Repair of concrete in Iceland
with
IMUR-CLADDING SYSTEM
and
Rough casting*

by

Civ. eng. Oddur G. Hjaltason

April 2003

Historical background

To day, concrete is the most popular material used in external walls in Iceland, and has been so for the last six or seven decades. The most common damage in external concrete walls is due to frost reaction in hardened concrete. In the recent years we have also discovered another big problem, which is corrosion of embedded steel in concrete, due to lack of concrete cover and use of concrete with too low cement content. Leakage through cracks in our walls is also a big problem. The most common factors causing these cracks are shrinkage, change of temperature, movement of moisture and too little reinforcement.

In the early stages there was no insulation on external walls, but in the early 40's lightweight concrete and cork insulation was used on the inner side in some buildings. The general interest for insulation of outside walls in Iceland came with the crisis caused by damaged concrete of external walls in the late 1970's, mostly due to frost and alkali reaction and also as a result of widespread leakage through cracks, in external walls, which had little or none reinforcement. Insulation on the outside of external walls was first used in the late 1970's, with some cladding systems imported from Europe, mainly Sweden, Norway and Germany.

The high cost of insulation and cladding external walls meant that builders kept to the former ways of insulating these walls on the inside. During this period, a cladding system, made in Germany, of acrylic-based plaster mix, on polystyrene insulation became popular for repairing frost-damaged walls. This system's popularity was due to lower costs, compared to cladding systems with air-spaced panels.

In the beginning of 1980's, the Building Research Institute of Iceland decided to perform some experiments with various types of cladding, due to widespread frost- and alkali damaged concrete walls, during the two previous decades. Following these studies, a strong interest arose in developing and preparing an Icelandic insulating system that would be competitive with foreign production. The first developmental step in this direction was taken in the Southeast Icelandic town of Höfn in Hornafjörður, during 1986. In 1987 the first application of cement based cladding system was made in this same town. To day we have two similar systems manufactured here in Iceland.

Icelandic cladding system for insulation of external walls

The manufacturer of IMUR-cladding system faced some several challenges in the early stages of the product. Marketing was inadequate, defects emerged in the cement facades and there were no specifications for the system. However, in the early stages of the system, a field investigation indicated that the product was an attractive sales item, since it was suitable for re-insulation of concrete buildings, especially those where appearance changes, if any, was to be minimised, and in many cases, the product proved reasonably successful. In light of this, the manufacturer decided to continue developing and marketing the product.

The system was first designed and used 1987, by an architect and civil engineer, Árni Kjartansson, and a craftsman, Sveinn Sighvatsson, with a help from an architect and civil engineer, Björn Marteinnsson. In the beginning the main purpose of the product was to repair damaged concrete facades in small buildings, mainly due to frost damages, by re-insulation of the outdoor facades of apartment buildings.

The system is based on a rockwool insulation, and a two-layered wall of a fiberised plaster, which is reinforced by a galvanised steel net. In the early stages of the system, the finish of the walls was usually two layers of a plastic paint. The load and bearing capacity of the system is made of fastening bolts, approx. 7-8 pieces per square meter, which is attached to the steel net.

In early stages of the system, many problems occurred in the fiberised plaster walls of the system, mainly cracks due to shrinkage, thermal expansion and contraction and changes in moisture, in the plaster walls. There were also some other problems in the system, due to the workmanship of the craftsmen.

Our task was to re-design the system. The first phase was a field study and a report for a number of apartment buildings which already had been re-insulated with the initial system. This phase of the project started in February 1990 and was finished in April the same year. The second phase was a new design of the system due to the failure in the plaster walls. This work started in Mars 1990 and was finished in June the same year. In the third phase, a manual was written with drawings and specifications for the IMUR cladding system. The consulting engineering firm Línuhönnun performed this work. Our goal was to gain some experience of the new product before further decision of development were taken by the producer.

The second section of the project work was performed in 1996. This work, which consisted of an other field study, a report and a research of the systems characteristic, was performed by the consulting engineering firm Línuhönnun and the Building Research Institute of Iceland.

The third section consisted mainly of some work concerning development of the plaster, details, research of the load bearing capacity along with calculations and design of a product for larger buildings. This section was finished late in 2000.

WHAT HAVE WE LEARNED

1. **Repair of frost damaged concrete is possible when:**
 - a) Frost damage is mostly in the cement paste, thickness of 1-3 mm.
 - Rough casting in sement sand mix over repair.
 - or
 - Replace damaged cement paste with cement based plaster mix.
 - Use a water-repellent mixture (silane) and paint on the surface.
 - b) Frost damage is only in some parts of the concrete walls.

- Thickness of damaged concrete is between 10-40 mm. use cement based plaster mix, with low permeability
 - Thickness is over 40 mm. Use a concrete replacement mix with low permeability and 330-350 kg of cement/m³ with low v/c-ratio, ca. 0,45. Use reinforcement k10 c 150/250.
- c) Frost damage is widely spread over the walls. Use insulation, at least 30 mm thick, and cheat claddings on the outside surface.
2. **High cement content in plaster mixes (sement/sand) is better: (1:1 to 1:1,5)**
- a) Less water absorbing from rain in the concrete walls.
 - b) Plaster mixes on concrete surfaces. Higher frost resistance
 - c) Less damages, mainly frost and corrosion in the concrete.

Reykjavík
Mars 2003

Oddur C.G. Hjaltason

European Standards for Protection and Repair of Concrete Structures

by Magne Maage

Synopsis: The European Council decided in June 1985 that an open internal market should be effective from January 1993. This included also construction products, and the Construction Products Directive 89/106/EEC was agreed in 1989, including the six essential requirements for products to be used in construction works.

Since then the activity within European standardisation has been very high, organised through the organisation “Comité Européen de Normalisation” (CEN), established in 1961. The aim was to produce “harmonised” standards in order to fulfil the CPD requirements. An agreement between the “International Organization for Standardisation” (ISO) and CEN also exists.

Generally, standards for products, design, execution and test methods are produced for all aspects within the construction area.

This presentation will cover some general aspects of European Standardisation and focus on European Standards for “Products and Systems for Protection and Repair of Concrete Structures”.

The work on these standards started in 1989 and is not yet completed. The whole area will consist of six main product specification standards covering different applications, a standard for definitions, a standard for control during production of products, a standard for general principles for the use of products and systems, and a standard for site work. A great number of test methods are included in the total package of standards.

Keywords: Concrete, Europe, Protection, Repair, Standardisation

INTRODUCTION

The European Economic Community (EEC) was established in 1957 as a co-operation between the European Coal and Steel Union (established in 1951) and the European Atomic Energy Community (established in 1957). The EEC was established in the wake of World War II to promote the lasting reconciliation of France and Germany, to develop the economies of the member states into one large common market, and to try to develop a political union of the states of the western European countries. The EEC was also called the Common Market. From 1993, the name was the European Community, and from 1999 the most used name is the European Union (EU). The European Commission (EC) is the “government” of EU.

The European Free Trade Association (EFTA) was established in 1960 between the western European countries not being members of EEC. EFTA was an organisation to remove barriers to trade in industrial goods among themselves, but with each nation maintaining its own commercial policy toward countries outside EFTA.

The combined area of EU and EFTA is today called the European Economic Area (EEA).

Already in 1961, the European organisation for standardisation “Comité Européen de Normalisation” (CEN) was established as a non-profit making co-operation between European countries (1). CEN got its present location in Brussels in 1975. In 1984/85, EEC/EFTA entered an agreement with CEN/CENELEC for preparation of harmonised standards in order to have standards that could complement the Directives, such as the Construction Product Directive (CPD). This was called the “New approach”, where the main intention was to transfer technical requirements from national regulations into European Standards. The original mission of CEN had been to promote voluntary technical harmonization in Europe, and to avoid differences between EEC and EFTA in technical requirements for products. The philosophy is that harmonization diminishes trade barriers, promotes safety, allows interoperability of products, systems and services, and promote common technical understanding.

The national standards bodies of the following countries are today (spring 2003) members of CEN: Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Luxembourg, Malta, The Netherlands, Norway, Portugal, Slovakia, Spain, Sweden, Switzerland, United Kingdom. Corresponding organizations are today the standardization bodies of Egypt, South Africa, Ukraine and Yugoslavia.

The activity within CEN was very low for many years, but in June 1985, the European Council decided that an open internal market within EEC should be effective from January 1993. This opened the need for European standards since all details could not be regulated by laws (directives). CEN was given the task of developing standards and all member states in CEN and EEA got the same rights and duties.

The “International Organization for Standardization” (ISO) was established in 1947 in Geneva. ISO has member states from all over the world while CEN has member states from Europe only. Standards developed by ISO may be used within a country depending on national decision, but member states may also have national standards for the same area in conflict with an ISO standard. This is not accepted for CEN standards. When a CEN standard has got a positive vote from enough member states, all CEN member states have to withdraw conflicting national standards for the same area within a certain time and adopt the CEN standard.

Most of the CEN member states are also members of ISO, and a co-operation between the two organisations was profitable for both parties. An agreement on exchanging of technical information between ISO and CEN (called the Lisbon Agreement) was active from January 1989. Subsequently, an agreement on technical co-operation between ISO and CEN (called the Vienna Agreement) was active from June 1991.

Parts of this paper are more or less reproduced from (2).

CONSTRUCTION PRODUCT DIRECTIVE (CPD)

The European Council decision from June 1985 on the open internal market, stated that the construction sector was one of the sectors where particular emphasis should be placed. This resulted in the particular “Directive 89/106/EEC Construction Products” (CPD) in 1989 (3). The CPD apply to “construction products”, meaning “...any product which is produced for incorporation in a permanent manner in construction works.” However, it is stated that the “member states are responsible for ensuring that buildings and civil engineering works on their territory are designed and executed in a way that does not endanger the safety of persons, domestic animals and property, while respecting other essential requirements in the interests of general well-being.” This is because different countries have different laws, safety requirements etc.

The CPD includes the important definitions of the six essential requirements (ER) on safety and other aspects, which are important for the general well-being. The six essential requirements are shown in Table 1 in bold.

Harmonised Standards (hEN) may be produced based on a mandate from the European Commission and the essential requirements. This leads to so-called CE marking of the product, which means that the product must be allowed free trade and free use for its intended purpose throughout the EEA.

(CE marking may also be based on European Technical Approval (ETA) for products for which no European Standard exists, normally products without long experience in a market. This route to CE marking will not be followed in this paper.)

EC MANDATE

The European Commission (EC) issues "mandates" to CEN, for standardisation work on harmonised standards, in agreement with the European Free Trade Association. A mandate is a political request, as agreed upon by the Member States of EU and EFTA, in support of

legislative work or an industrial policy from EC and EFTA. Product mandates lead to the development of "Harmonised Standards" in support of the "Essential Requirements" which allow the CE marking of the products.

Products and systems for the protection and repair of concrete structures are included in the mandate M/128 "Products Related to Concrete, Mortar and Grout", given to CEN (4). Table 2 shows the system of attestation of conformity for these types of products.

The mandate M/128 also lists the performance characteristics under each essential requirement (ER) that have to be addressed in a harmonised standard. Not all the listed performance characteristics are necessary in all standards, depending on intended use of the product. Table 1 lists the performance characteristics under each ER.

ATTESTATION OF CONFORMITY

Products have to fulfil a system of attestation of conformity, depending on intended use of the product, resulting in different degrees of third party intervention in attestation of conformity. However, the producer is fully responsible for the attestation that products are in conformity with the requirements of a technical specification, independent of the involvement of a third party. The CE numbering system for conformity attestation is 1+, 1, 2+, 2, 3 and 4 (5):

System 4 has no compulsory intervention of a third party in attestation of conformity. The producer is responsible for attestation of conformity with the requirements of the technical specification for the product. However, the producer is free to having the necessary tests done by outside laboratories.

System 3 means that responsibility for the Initial Type Test, ITT, is given to a third party rather than to the producer. All other responsibilities fall on the producer. An ITT is neither an assessment of the fitness for the use of the product nor an assessment of its conformity with a technical specification. The ITT is rather a determination of the performance of a product, on the basis of tests or other procedures described in the technical specifications. Responsibility for sampling the products to be tested, in accordance with the rules laid down in the technical specification, lies with the producer.

System 2 and 2+ means that the responsibility for the certification of Factory Production Control, FPC, (initial inspection alone for system 2 and continuous surveillance for system 2+) is given to a third party. FPC is the permanent internal control of production exercised by the producer. The third party shall confirm that the FPC is in conformity with the requirements of the technical specification and the CPD. Certification of the FPC does not involve assessment of the overall conformity of a product with technical specification, this remains the responsibility of the producer.

System 1 and 1+ means that the responsibility for the certification of the conformity of the product is given to a third party. Normally, the individual tasks required to enable product certification to take place are carried out by various parties (e.g. producer, certification body, inspection body, approved laboratories). The certification body is responsible for assembling all of the relevant information, verifying that tasks have been carried out according to the technical specification and assessing and certifying the conformity of the product. Responsibility for product sampling lies with the certification body, but may be delegated to an inspection body.

For products and systems for the protection and repair of concrete structures, the following systems will be used, see Table 2:

- System 4 for uses with low performance requirements in buildings and civil engineering works.
- System 2+ for other uses in buildings and civil engineering works.
- System 1, 3 and 4 for uses subjected to reaction to fire regulations.

THIRD PARTY, GROUP OF NOTIFIED BODIES AND MARKET CONTROL

A third party, mentioned under systems for attestation of conformity, is a body independent of the producer of products. It may be a technical control body, a certification body, an inspection body or an approved laboratory, often given the general name “Notified Body”. Such bodies are approved by the respective national building authorities for certain areas and may operate in other EU and EFTA countries after being notified in respective country. Notified bodies have the authority to carry out the type of inspection/control mentioned in the systems for attestation of conformity, which they are approved for. In order to level the activities for all notified bodies in all countries, they have the duty to follow the “Position Papers” from the Group of Notified Bodies (6).

The Group of Notified Bodies is a co-ordination group of all Notified Bodies in the EU and EFTA countries operating within a mandated area. All Notified Bodies have the duty to be members of the group, and to follow the decisions taken by the group. One Group of Notified Bodies is established by EC for each mandated area. The scope is to agree on detailed understanding of different harmonised European standards (hEN) and on administrative routines. Such agreements are outlined in so called “Position Papers”, which are mandatory for all Notified Bodies (6).

Market Control is a body at a national level confirming that products with CE marking are fulfilling the requirements in the respective hEN. Each country decides the level of this control. Normally the control will be carried out as spot tests or when there is doubt over conformity (6).

PRODUCTS AND SYSTEMS FOR THE PROTECTION AND REPAIR OF CONCRETE STRUCTURES

The CEN Technical Committee 104 “Concrete and related products”, is dealing with:

- Concrete
- Execution of concrete structures
- Silica fume
- Fly ash
- Additions
- Mixing water
- Protection and repair of concrete structures

In this paper, only the last topic will be addressed. It is taken care of by the Sub Committee 8 (SC 8) under Technical Committee 104 (TC 104) within CEN. Professor Geoff Mays,

Cranfield University, UK, is European Convenor and the French Standardisation Organisation AFNOR is providing the Secretariat (7).

The system of standards is shown in Table 3. In addition, a great number of standards for test methods are developed.

Electrochemical methods like “re-alkalisation” of carbonated concrete, “chloride extraction” of chloride contaminated concrete and “cathodic protection” are covered by principles 7 and 10 respectively, see Table 4. Standards for these methods are drafted by Technical Committee TC 219:

- EN 12696-1 “Cathodic protection of steel in concrete. Part 1: Atmospherically exposed concrete” (Completed).
- prEN 14038-1 “Electrochemical re-alkalisation and chloride extraction treatments for reinforced concrete – Part 1: Re-alkalisation” (Under development)
- prEN 14038-2 “Electrochemical re-alkalisation and chloride extraction treatments for reinforced concrete – Part 2: Chloride extraction” (Not started yet)

A standard for sprayed concrete, prEN 14487-1 – “Sprayed concrete – Part 1: Definitions, specifications and conformity”, is drafted by TC 104. It passed the stage of CEN enquiry by the end of 2002.

EN 1504 - Part 1: " General scope and definitions"

This standard defines types of products and systems for repair, for use in maintenance and protection, restoration and strengthening of concrete structures. The definitions are divided into the following groups:

- General (products, systems, technology and performance).
- Main categories of products and systems.
- Main chemical types and constituents of protection and repair products and systems.

ENV 1504 – Part 9: "General principles for use of products"

This standard includes the following items:

- The need for inspection, testing and assessment before, during and after repair.
- Protection from and repair of defects caused by the influence of certain environments and chemical substances.
- The repair of defects from such causes as mechanical damage, differential settlement, loading, biological attack, inadequate construction or the use of unsuitable construction materials.
- Protection and repair in order to decrease the progress of alkali-silica reaction.
- Meeting the required structural capacity in repair by:
 - replacement or addition of embedded or external reinforcement.
 - filling of external voids between elements to ensure structural continuity.
- Meeting the required structural capacity by replacement or addition of concrete.

- Waterproofing as an integral part of protection and repair.
- Protection and repair of pavements, runways, hard standings and floors, as an integral part of protection and repair.
- Methods of protection and repair including:
 - treating cracks
 - restoring passivity to reinforcement.
 - reducing the rate of corrosion of reinforcement by limiting moisture content.
 - reducing the rate of corrosion of reinforcement by electrochemical methods.
 - controlling corrosion of reinforcement with coatings.

Minimum requirements before protection and repair are given. However, this is not a guide for inspection and assessing the condition of the concrete structure before, during or after repair. Additional works to meet the essential requirements in the CPD and regulations or provisions valid in the place of use shall be specified.

The following options shall be taken into account in deciding the appropriate actions to meet the future requirements for the life of the structure:

- Do nothing for a certain time.
- Re-analysis of structural capacity, possibly leading to downgrading of the function of the concrete structure.
- Prevention or reduction of future deterioration, without improvement of the concrete structure.
- Improving, strengthening or refurbishment of all or part of the concrete structure.
- Reconstruction of part or all of the concrete structure.
- Demolition of all or part of the concrete structure.

Factors to be considered when choosing options are listed under the following headings:

- General, for example intended use, design life, cost, properties of the existing substrate, appearance of the repaired concrete structure etc.
- Health and safety, for example consequences of structural failure, impact of the repair operations on occupiers and users of the structure and on the public etc.
- Structural, for example the means by which loads will be carried during and after repair etc.
- Environmental, for example the need to protect the structure during and after the repair etc.

The basis for the choice of products and systems is founded in 11 principles of protection and repair. These are based on the chemical and physical laws, which allow prevention or stabilisation of the chemical or physical deterioration processes in the concrete or the electrochemical corrosion processes on the steel surface. These 11 principles, and associated methods of protection and repair, covered by the 1504 series of standards, are summarised in Table 4 for defects in concrete and for reinforcement corrosion, respectively. Methods making use of products and systems not covered by the 1504 series are not listed in Table 4, for example pre-stressing, replacing elements etc. For some of the listed methods, it is pointed out that this does not imply their approval, for example re-alkalisation of carbonated concrete by diffusion and applying inhibitors to the concrete.

This Part 9 of the 1504 series of standards may be called the consultants standard.

EN 1504 – Part 10: "Site application of products and systems and quality control of the works"

The scope of this standard includes the following items:

- The preparation of the concrete or reinforcement before application of products and systems.
- The minimum requirements as to environmental conditions for storage and application of products and systems.
- Controlling the quality of the repair work.

This standard is covering most of the methods included in Part 9, but leaving out methods covered by other standards, for example:

- Erecting external panels
- Sheltering or over-cladding
- Replacing elements
- Pre-stressing (post tensioning)

This Part 10 of the 1504 series of standards may be called the contractors standard.

EN 1504 – Part 8: "Quality control and evaluation of conformity"

The scope of this standard includes the following:

- Sampling during production of products and systems.
- Evaluation of conformity to the requirements given in the product specification standards in Parts 2 – 7 of the 1504 series for:
 - identification tests.
 - performance tests.
 - factory production control (FPC)
- Marking and labelling.

This Part 8 of the 1504 series of standards may be called the material producers standard, and it will be used in close association with the six product specification standards.

EN 1504 – Parts 2 to 7: Product specification standards (see Table 3)

These parts of the 1504-series are the product specification standards defined as harmonised standards. They are all built up in the same way, but covering different principles and methods for protection and repair, as described in Part 9, see also Table 4.

The main list of content of Parts 2 – 7 are shown in Table 5. Some of the chapters and Annexes will be commented.

Chapter 4, "**Performance characteristics for intended uses**", has a listing of performance characteristics for "all intended uses" and for "certain intended uses" for each method associated with Part 9 of the 1504 series.

For example, Table 6, reproduced from prEN 1504 – Part 4, lists the performance characteristics for structural bonding agents associated with the principle "Structural Strengthening", see Table 4, for each of the two repair methods "Bonded plate reinforcement" and "Bonded mortar or concrete". This Table includes more characteristics than necessary for fulfilling the essential requirements because necessary characteristics for the intended use shall be fulfilled.

Chapter 5 – Requirements (see Table 5) is divided into several sub-clauses:

"Identification requirements": An identification test is carried out to verify a required property of the product or system in terms of consistency of production. Test methods to be used are given, and the requirements are usually stated in terms of the property falling within a specified percentage of a declared value provided by the manufacturer.

For example, it is stated in prEN 1504 – Part 4 that "Ash content by direct calcinations" is one of the properties that shall be tested. The test method to be used is EN ISO 3451-1 "Plastics – Determination of ash – Part 1: General principles", and the requirement is that "Documented value $\pm 5\%$ or ± 1 percentage point of the total product, whichever is the greater."

"Performance requirements": A performance test is undertaken to verify directly a required property of the product or system in terms of its specified performance during application and use. Test methods to be used are given, and requirements may be stated in terms of a threshold value, a declared value or as a pass/fail criteria.

For example, it is stated in prEN 1504 – Part 4 that "Shear strength" is one of the properties that shall be tested (see Table 6). The test method to be used is EN 12188 "Determination of adhesion steel to steel for characterisation of structural bonding agents", and the requirement is that the mean value should not be smaller than 12 N/mm^2 for bonded plate reinforcement and 6 N/mm^2 for bonded mortar or concrete, respectively.

For chapters 6, 7 and 8, reference is made to prEN 1504 – Part 8.

Each product specification standard includes an "Annex ZA" which contains those clauses that specifically address the provisions of the CPD. It identifies those performance characteristics, which the mandate M/128 specifies shall be covered by the harmonised standard, together with the relevant requirements clauses.

For example, it is stated in prEN 1504 – Part 4 that the following essential characteristics for bonded mortar or concrete are (this may be found by combining Tables 1 and 6):

- Bond/adhesion strength.
- Shear strength
- Compressive strength
- Shrinkage/expansion
- Workability

- Sensitivity to water
- Modulus of elasticity
- Coefficient of thermal expansion
- Glass transition temperature
- Durability

This means that not all the characteristics listed under “Performance characteristics for intended uses” are necessary to be documented in order to fulfil the requirements for CE marking.

The Annex ZA also contains information on the relevant system of attestation of conformity, including the assignation of tasks for the manufacturer with respect to initial type testing and factory production control, and for the notified body with respect to certification of factory production control. This is described earlier in this paper.

The last part of Annex ZA provides guidelines on CE marking. An example is shown in Figure 1 for structural bonding product for bonded plate reinforcement for uses other than low performance requirements.

TEST METHOD STANDARDS

In the product specification standards, reference is made to more than 70 test method standards. Most of these standards are new, being drafted by SC 8. In most cases these are based upon proven existing techniques. In other cases further research has been, or will be, necessary before test methods for standardisation purposes can be recommended.

STATUS

The work on this system of standards started in 1989. The status today (spring 2003) of the standards within the 1504 series is outlined in Table 3. Some product specification standards will be completed before others, and may be put into use before all standards are completed. However, it may be more convenient to introduce all standards at the same time. This is planned to be December 2004.

Around 70 new test method standards have been drafted by SC 8. Most of these have been published or are ready for or in the process of Formal Vote. Some are still under preparation, but the drafts are relative mature for all of them.

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Table 1. Performance characteristics of the products for the protection and repair of concrete structures to be covered by the harmonised standard

ER	Performance characteristics	Durability
1	<p>Mechanical resistance and stability <i>As relevant to the type of product:</i></p> <ul style="list-style-type: none"> ○ Bond/adhesion strength ○ Shear strength ○ Compressive strength ○ Tensile strength ○ Bending strength ○ Shrinkage/expansion ○ Workability ○ Sensitivity to water (incl. seawater) ○ Pull-out behaviour ○ Crack bridging (static and dynamic) ○ Diffusion resistance ○ Filling share ○ Penetration behaviour ○ Composition (e.g. chloride content, ...as relevant) ○ Corrosion protection/inhibition ○ Water repellence ○ Modulus of elasticity ○ Coefficient of thermal expansion ○ Glass transition temperature 	<p>Yes <i>(Against alkali, corrosion, abrasion, frost, de-icing salt, temperature change, ...as relevant)</i></p>
2	<p>Safety in case of fire</p> <ul style="list-style-type: none"> ○ Reaction to fire 	
3	<p>Hygiene, health and the environment</p> <ul style="list-style-type: none"> ○ Water vapour permeability ○ Water permeability ○ Release of dangerous substances¹ 	
4	<p>Safety in use</p> <ul style="list-style-type: none"> ○ Skid resistance 	
5	<p>Protection against noise</p>	
6	<p>Energy, economy and heat retention</p> <ul style="list-style-type: none"> ○ Thermal conductivity 	

1) In particular, those dangerous substances defined in Council Directive 76/769/EEC, as amended.

Table 2. System of attestation of conformity

Product(s)	Intended use(s)	Level(s) or class(es)	Attestation of conformity system(s)
Concrete protection and repair products	For uses with low performance requirements in buildings and civil engineering works	-	4
	For uses in buildings and civil engineering works	-	2+
Concrete protection and repair products	For uses subject to reaction to fire regulations	A1*, A2*, B*, C*	1
		A1**, A2*, B**, C**, D, E	3
		(A1 to E)**, F	4
<p>System 1: See CPD, Annex III.2(i), without audit-testing of samples System 2+: See CPD Annex III.2 (ii) (First possibility, including certification of the factory production control by an approved body on the basis of initial inspection of factory and of factory production control as well as of continuous surveillance, assessment and approval of factory production control. System 3: See CPD, Annex III.2(ii), Second possibility System 4: See CPD Annex III.2(ii), Third possibility</p>			
<p>*Products/materials for which a clearly identifiable stage in the production process results in an improvement of the reaction to fire classification (e.g. an addition of fire retardants or a limiting of organic material) **Products/materials not covered by footnote (*) *** Products/materials that do not require to be tested for reaction to fire (e.g. Products/materials of class A1 according to the Decision 96/603/EC, as amended by Decision 2000/605/EC).</p>			

Table 3. System of Standards for "Products and Systems for the Protection and Repair of Concrete Structures"

Standard No	Title	Status (spring 2003)³
EN 1504-1	General scope and definitions	Published as EN in December 1997.
prEN 1504-2 ¹	Surface protection systems for concrete	CEN enquiry completed September 2000. Under final preparation for Formal Vote, decided April 2002.
prEN 1504-3 ¹	Structural and non-structural repair	CEN enquiry completed September 2001. Under preparation for Formal Vote decision.
prEN 1504-4 ¹	Structural bonding	CEN enquiry completed September 2000. Under final preparation for Formal Vote, decided April 2002.
prEN 1504-5 ¹	Concrete injection	CEN enquiry completed April 2002. Under preparation for Formal Vote decision.
prEN 1504-6 ¹	Anchoring products	Under preparation. CEN enquiry decided October 2002.
prEN 1504-7 ¹	Reinforcement corrosion prevention	Under preparation.
prEN 1504-8 ¹	Quality control and evaluation of conformity	CEN enquiry completed November 2001. Under final preparation for Formal Vote, decided April 2002.
ENV 1504-9 ²	General principles for use of products and systems	Published as ENV in July 1997. Will be revised and changed to an EN when all parts of 1504 series are completed.
prEN 1504-10 ¹	Site application of products and systems and quality control of the works	Formal vote completed December 2002.

1) Under preparation.

2) Will be revised and changed to an EN when all parts of the 1504 series are completed.

3) There may be a delay after formal vote due to formal comments from the Group of Notified Bodies.

Table 4. Principles and Methods for Protection and Repair of Concrete Structures related to defects in concrete and reinforcement corrosion.

Principle	Definition	Methods of Protection and Repair
1. Protection against ingress	Reducing or preventing the ingress of adverse agents	<ul style="list-style-type: none"> ○ Impregnation ○ Surface coating ○ Filling cracks
2. Moisture control	Adjusting and maintaining the moisture content in the concrete within a specified range of values	<ul style="list-style-type: none"> ○ Hydrophobic impregnation ○ Surface coating
3. Concrete restoration	Restoring the original concrete of an element of the structure to the originally specified shape and function	<ul style="list-style-type: none"> ○ Applying mortar by hand ○ Recasting with concrete ○ Spraying concrete or mortar
4. Structural strengthening	Increasing or restoring the structural load bearing capacity of an element of the concrete structure	<ul style="list-style-type: none"> ○ Installing bonded rebars ○ Plate bonding ○ Adding mortar or concrete ○ Injecting cracks, voids or interstices ○ Filling cracks, voids or interstices
5. Physical resistance	Increasing resistance to physical or mechanical attack	<ul style="list-style-type: none"> ○ Overlays or coatings ○ Impregnation
6. Resistance to chemicals	Increasing resistance of the concrete surface to deterioration by chemical attack	<ul style="list-style-type: none"> ○ Overlays or coatings ○ Impregnation
7. Preserving or restoring passivity	Creating chemical conditions in which the surface of the reinforcement is maintained in or is returned to a passive condition	<ul style="list-style-type: none"> ○ Increasing cover to reinforcement ○ Replacing contaminated or carbonated concrete ○ Realkalisation of carbonated concrete by diffusion
8. Increasing resistivity	Increasing the electrical resistivity of the concrete	<ul style="list-style-type: none"> ○ Limiting moisture content by surface treatments, coating or sheltering
9. Cathodic control	Creating conditions in which potentially cathodic areas of reinforcement are unable to drive an anodic reaction	<ul style="list-style-type: none"> ○ Limiting oxygen content by saturation or surface coating
10. Cathodic protection		<ul style="list-style-type: none"> ○ Applying electrical potential
11. Control of anodic areas	Creating conditions in which potentially anodic areas of reinforcement are unable to take part in the corrosion reaction	<ul style="list-style-type: none"> ○ Painting reinforcement with coatings containing active pigments ○ Painting reinforcement with barrier coatings ○ Applying inhibitors to the concrete

Table 5 List of content of Parts 2 – 7 of the 1504 series of product specification standards


Chapter no.	Content
	Foreword
	Introduction
1	Scope
2	Normative references
3	Definitions
4	Performance characteristics for intended uses
5	Requirements
6	Sampling
7	Evaluation of conformity
8	Marking and labelling
Annex A	Examples of how to use the classification system
Annex B	Release of dangerous substances
Annex XA	<p>Clauses addressing the provisions of EU Construction Products Directive.</p> <p>ZA1: Clauses of this European Standard addressing the essential requirements of EU Construction Products Directive.</p> <p>ZA2: Attestation of conformity</p> <p>ZA3: CE marking and labelling.</p>

Table 6 Performance characteristics for all and certain intended uses (Table 1 in prEN 1504 – Part 4)

Performance Characteristic	Principle of Repair 4 Structural Strengthening	
	Repair Method 4.3 Bonded Plate Reinforcement	Repair Method 4.4 Bonded Mortar or Concrete
1. Suitability for application : a) to vertical surfaces & soffits b) to top horizontal surfaces c) by injection	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>
2. Suitability for application and curing under the following special environmental conditions: a) low or high temperature b) wet substrate	<input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input checked="" type="checkbox"/>
3. Adhesion: a) plate to plate b) plate to concrete c) corrosion protected steel to corrosion protected steel d) corrosion protected steel to concrete e) hardened concrete to hardened concrete f) fresh concrete to hardened concrete	<input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/>
4. Durability of composite system: a) thermal cycling b) moisture cycling	<input checked="" type="checkbox"/> <input checked="" type="checkbox"/>	<input checked="" type="checkbox"/> <input checked="" type="checkbox"/>
5. Material characteristics for the designer: a) open time b) workable life c) modulus of elasticity in compression d) modulus of elasticity in flexure e) compressive strength f) shear strength g) glass transition temperature h) coefficient of thermal expansion i) shrinkage	<input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/>	<input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/> <input checked="" type="checkbox"/>

■ = a performance characteristic which shall be considered for all intended uses
□ = a performance characteristic which shall be considered for certain intended uses

Figure ZA.1 CE marking information

 01234	
AnyCo Ltd, PO Bx 21, B-1050 00	
0123-CPD-0456 EN 1504-4 Structural bonding product for bonded plate reinforcement for uses other than low performance requirements Bond/adhesion strength: Pull off strength $\geq 14 \text{ N/mm}^2$ Slant shear strength at: 50° $\geq 50 \text{ N/mm}^2$ 60° $\geq 60 \text{ N/mm}^2$ 70° $\geq 70 \text{ N/mm}^2$ Shear strength: $\geq 12 \text{ N/mm}^2$ Shrinkage/expansion: $\leq 0.1\%$ Workability: 40 minutes at 20°C Modulus of elasticity: $\geq 2000 \text{ N/mm}^2$ Coefficient of thermal expansion: $\leq 100 \times 10^{-6}$ per K Glass transition temperature: $\geq 45^\circ\text{C}$ Reaction to fire..... Euroclass B Durability..... Pass Dangerous Substances Comply with clause 5.4	

CE conformity marking consisting of the CE symbol given in directive 93/68/EEC
Identification number of the notified body (for products under systems 1 or 2+)
Name or identifying mark and registered address of the producer
Last two digits of the year in which the marking was affixed
Number of the EC Certificate (for products under system 1) or the FPC certificate (for products under system 2+)
No of European standard
Description
and
information on product and on regulated characteristics

This product should be accompanied, when and where required and in the appropriate form, by documentation listing any legislation on dangerous substances for which compliance is claimed, together with any information required by that legislation.

NOTE European legislation without national derogations need not be mentioned.

Figure 1 Gives an example of the information accompanying the CE marking.

NORECON Workshop
Decision and Requirements for Repair
Lund, April 3-4, 2003

CONDITION ASSESSMENT
for
PREDICTION OF THE RESIDUAL SERVICE LIFE OF
CONCRETE STRUCTURES

Göran Fagerlund
Div. of Building Materials, Lund Institute of Technology

CONDITION ASSESSMENT for PREDICTION OF THE RESIDUAL SERVICE LIFE OF CONCRETE STRUCTURES

1. European projects

Two European projects have been performed on the actual subject:

1. BRITE-EURAM Project “*The Residual Service Life of Concrete Structures*” 1992-1996.

Partners were:

From UK:

The British Cement Association (Coordinator)

From Spain:

Instituto Eduardo Torroja

Geocisa

From Sweden:

Swedish Cement and Concrete Research Institute

Cementa

Div Building Materials, Lund Institute of Technology

The aim of the project was to develop means of analysing the present status, especially the structural stability of a deteriorated structure, and also to predict the future deterioration rate. Experimental work was performed together with theoretical analyses of deterioration rate and development of practical service life models. The project was a combination of materials research and research on structural stability

Three destruction mechanisms were treated, (i) alkali-silica reaction, (ii) reinforcement corrosion, (iii) frost attack. Some attempts were also made to investigate synergism between different mechanisms.

31 official deliverables were produced. Besides 36 other reports and papers in journals and at conferences were produced by individual participants.

The final result of the project was a *Manual* on assessment technique for damaged structures.

The Manual together with a list of all deliverables and other publications have been published¹.

¹ Manual for Assessment of Residual Service Life of Reinforced Concrete Structures. Div. Building Materials, Lund Institute of Technology, Report TVBM-7117, Lund, 1997.

2. INNOVATION Project “ *CONTECVET; A Validated Users Manual for Assessing the Residual Service Life of Concrete Structures*”, 1998-2001

Partners were:

From UK:

British Cement Association (Coordinator)
Transport Research Laboratory
National Car Parks Ltd

From Spain:

Instituto Eduardo Torroja
Geocisa
Dirección Generalde Arquitecturay Vivienda. Generalitat Valenciana
IBEDROLA
ENRESA

From Sweden:

Swedish Cement and Concrete Research Institute
Vattenfall Utveckling AB
Vägverket
Banverket
Skanska
Div. Building Materials, Lund Institute of Technology

The aim of the project was to test and further develop the previous Manual – to “validate” it- by applying it to real damage cases. Therefore a number of damaged structures were analysed by directly testing the structure or by using available data obtained from earlier studies. Besides laboratory studies from the previous EU-project were further analysed. Results from ongoing research were collected. All this information was used for developing *three manuals*, one for each destruction mechanisms^{2 3 4}. All Manuals are available from Department of Building Materials, Lund Institute of Technology. They can also be obtained from the partner that was responsible for the actual Manual, see the footnotes.

The three manuals cover the same mechanisms as the previous project; ASR, reinforcement corrosion, frost damage. The case studies also covered these mechanisms. The partner Vattenfall Utveckling, however, was also interested in leaching. Therefore a report was also produced on that destruction mechanism⁵. It also contains some information on the effect of synergy between different destruction mechanisms.

The Manuals give information on how an assessment can be performed for the different mechanisms. They inform on what measurements on the structure shall be made and on how the results can be used for an assessment of the present and future status. The main focus is on the structural stability and safety of the structure. Thus, it is shown how strength data for the concrete, or data for corrosion rate, or data for expansion of an ASR-affected structure, together with information of the destruction kinetics can be translated to structural stability.

²) Manual for Assessing Structures Affected by ASR. British Cement Association

³) Manual for Assessing Corrosion-Affected Concrete Structures. Instituto Eduardo Torroja

⁴) Manual for Assessing Concrete Structures Affected by Frost. Div. Building Materials. Lund Inst. of Technol

⁵) G. Fagerlund: Leaching of Concrete. The Leaching process. Extrapolation of Deterioration. Effect on Structural Stability. Div. Building Materials. Lund Inst. of Technology. Report TVBM-3091, 2000

2. Assessment procedure

The principles for an assessment of the present and future structural stability (future service life) are described in a *General Introduction to the Manuals*. These principles were prepared by the Coordinator of the CONTECVET-project, Professor George Somerville who was working for British Cement Association. This introduction is added as an Appendix below.

Basic activities in an assessment are clearly shown in this Appendix. In short, they are:

- 1: Quantify the effect of deterioration on the present structural stability and serviceability
- 2: Identify the causes of damage
- 3: Predict the future deterioration of structural stability and serviceability
- 4: Establish values for minimum acceptable performance with regard to structural stability and serviceability
- 5: Take decisions on present and future action (demolish, repair, wait)

An assessment therefore requires information from the status of the structure. This requires testing of the structure like determination of strength, monitoring corrosion rate, determining carbonation front and chloride profile, measurement of moisture content etc.

It also requires information of the time process of individual destruction processes and on the changes in time process depending on synergetic effect.

The destruction process of the material (concrete or reinforcement) must be translated to destruction of structural stability and serviceability of the structure as a whole (e.g. a certain level of material destruction in one part of the structure might have minor influence on the structural stability while the same level of material destruction in another part might be disastrous).

The owner of the structure (and the society) must be able to quantify the acceptable risks and the acceptable general performance of the structure (also considering what is sometimes called non-technical issues).

APPENDIX

General Introduction to Manuals on “Assessment of Residual Service Life of Concrete Structures”

George Somerville
British Cement Association

Extract from
“CONTECVET. Manual for Assessing Concrete Structures Affected by Frost”

1. Scope

This Manual is written for use by experienced and expert personnel, and gives recommendations on the practice, principles and performance requirements for the inspection and assessment of deteriorating reinforced concrete structures and elements. The scope is limited to buildings, bridges and dams in reinforced concrete, although subject to amendment and further elaboration in complementary documents, the basic approach may also apply to other types of structure, or forms of concrete construction.

The primary cause of deterioration considered in this Manual is frost; companion Manuals cover alkali-silica reaction (ASR) and reinforcement corrosion. It is recognised that other causes of damage or deterioration can occur, singly or in combination with these primary causes; in such cases, further elaboration of the approach in this Manual may be necessary.

2. Purpose of Assessment

Before beginning an assessment, it is important to be clear on the objectives. In general, assessment may be concerned with a proposed change in use, or in loading conditions. This Manual relates only to assessment where deterioration is involved, as defined in Section 1. In that context, assessment is an aid to decision making, as part of the asset management process.

A prime concern to the owner will be safety. However, he will also be concerned with maintaining the function of the structure, during its expected life time, at minimum total cost, ie with the development of an optimum management strategy. This means that deterioration, as such, is secondary to the effect that it can have on the strength, stiffness and serviceability of the structure.

In setting objectives for an assessment, it is important to remember that the owner will want advice on possible future actions. Some of these are shown typically in Table 1, where it may be seen that the results from the assessment are not the only factor involved.

Table 1: Possible future actions after assessment and factors involved.

ACTIONS

1.	Do nothing; inspect again in x years	
2.	No action now, but monitor	
3.	Routine maintenance; cosmetics; some patch repairs] evaluate cost/
4.	Remedial action: specialist repairs and/or protection] benefit in whole
5.	Partially replace, or upgrade, or strength] life costing
6.	Demolish and rebuild] terms

TIMESCALE

Now
 1-5 years
 5-10 years
 10-25 years
 Longer term

FACTORS

Results from assessment
 Future change in function
 Future change in standards
 Type and nature of structure
 Risk and consequences of failure

Recognition of this is important in ensuring that the right information is obtained, to permit confident management decisions to be made.

3. Overall approach

The recommendations in this Manual are based on the principle of progressive screening. This means that the investigation should be taken no further than is necessary to reach a decision, i.e. to decide on which action, given in Table 1, is appropriate, with an acceptable level of confidence.

In the sections which follow, two primary stages are foreseen:

- 1) Preliminary Assessment
- 2) Detailed Assessment

In general, preliminary assessment is a qualitative approach to determine whether or not a further, more rigorous, evaluation is necessary (the Detailed Assessment). However, in some cases, it can be self-contained when associated with simple analysis and calculations; in this Manual, this is then called the Simplified Method.

A schematic outline of progressive assessment procedures is given in Table 2. Table 2 indicated the type of input necessary both for Preliminary and Detailed Assessment.

Table 2: Schematic outline of progressive assessment procedures

Assessment Phase	Conclusion			Recommendations
	Based on	Result	Reason	
Preliminary	Records Survey data	Adequate	Sufficient residual service life and load-carrying capacity.	Monitor
	Site Measurement Cores Crack pattern & widths Simple analyses	Borderline	Insufficient data; or residual service life and load-carrying capacity marginally less than that required.	Detailed assessment
		Inadequate	Insufficient residual service life and load-carrying capacity.	Modify adequacy criteria, and reassess. Consider alternative remedial actions. Detailed assessment.
Detailed	As preliminary plus: Monitoring Laboratory tests More sophisticated analyses (i.e. more INSIGHT, Figure 1)	Adequate	Sufficient capacity for required loading (by calculation or load test).	Monitor
		Borderline	Insufficient data; or residual service life and load-carrying capacity marginally less than that required.	Load test to classify as adequate or inadequate. Consider future management and maintenance.
		Inadequate	Insufficient residual service life and load-carrying capacity (by calculation or load test).	Options are: Modify adequacy criteria and/or evaluate actual loading, and reassess. Consider possible actions in Table 2.

In this Manual, the decision-making process, at the end of the Preliminary phase, is based on a Simplified Index of Structural Damage (SISD rating).

The necessary input in Table 2 is targeted mainly at establishing the extent of the damage due to deterioration - and, of course, with identifying the primary cause of that damage. This involved a mix of ‘Overview’ and ‘Insight’ as illustrated in Figure 1. An overview will always be necessary; how much insight is required will depend on the nature and scale of the symptoms of deterioration. The further an Assessment has to proceed, the more insight is necessary.

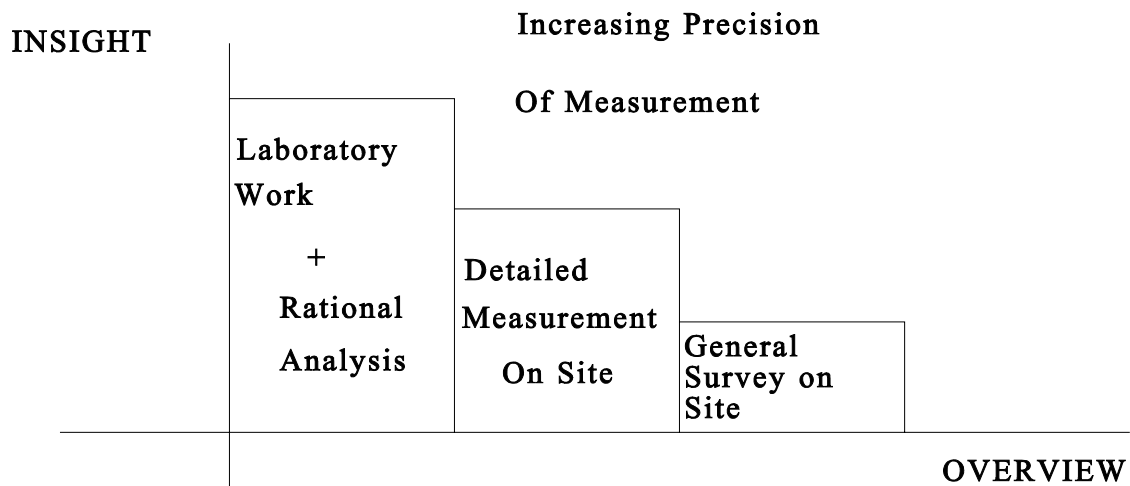


Figure 1: The essential balance between Insight and Overview in assessment

Table 2 also indicated that some analysis and calculations are always necessary, since the prime objective is structural assessment. Again this need can be treated as progressive, with the following options:

- 1) Simple (elastic) analysis, with full (design) partial factors.
- 2) More refined analysis, with better structural idealisations.
- 3) As for 1 and 2, but with assessment-specific imposed loads (usually reduced).
- 4) Taking account of additional safety characteristics (eg, partial redundancy; membrane action; the influence of non-structural elements).
- 5) Full reliability analysis (for exceptional cases).

Option 1 is usually sufficient for Preliminary Assessment. As more information becomes available from survey data, modified values for reduced sections, or changed mechanical properties, can be fed in, to better represent the structural capacity of the deteriorated structure.

4. Strategy and principles

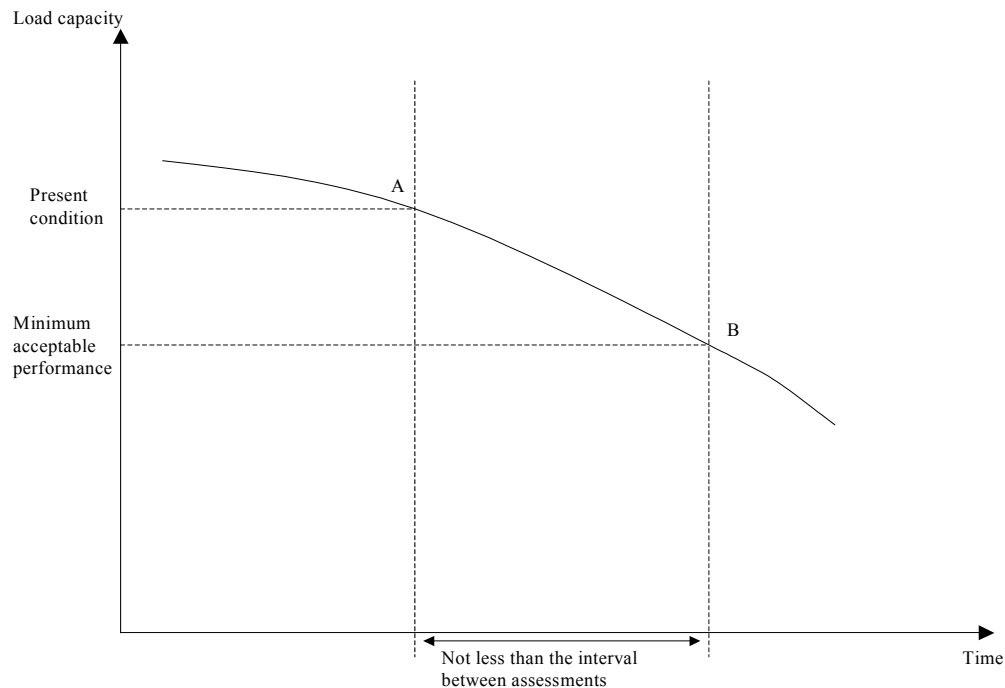


Figure 2: Assessment strategy

The assessment strategy is outlined in Figure 2. Present condition is assessed at point A. The prediction of future state should not then reach the defined minimum acceptable performance, before either the next assessment point or some remedial action is taken. The vertical axis in Figure 2 is expressed in terms of load (structural) capacity, and the Figure shows a reduction between points A and B. Prior to reaching that stage, it will be necessary to assess the extent of the deterioration at point A, and how that extends towards point B. This is stressed, since the shape of the curve A-B may be different for deterioration, compared with structural capacity.

From Figure 2, some general principles can be established for assessment, as follows:

- 1) The sequence of events is:
 - a) quantify the effects of deterioration
 - b) identify the prime causes
 - c) predict future deterioration
 - d) assess structural implications
 - e) establish values for minimum acceptable performance
 - f) take decisions on future action

- 2) Events a) to c), in principle 1 above, are central to the Preliminary Assessment stage and effectively involve damage classification. In addition, it will be necessary to 'understand' the structure – physically, plus its design basis and structure sensitivity - mainly via existing records.

- 3) In assessing structural implications, the following stages may be necessary:
 - a) The effect of the deterioration on how the structure as a whole actually carries the imposed loads. This is the ‘analysis of structure’ phase, in determining maximum values for key load effects. Any loss of stiffness will be especially important in this evaluation.
 - b) The effect of the deterioration on the resistance of sections and elements, for all critical action effects – since any particular level of deterioration may affect each of these differently.
 - c) A review of structural sensitivity, including the possibility of failure mechanisms, caused uniquely by the deterioration.
- 4) Figure 2 will require discussion with the owner at an early stage of Detailed Assessment
 - a) to establish an agreed level for minimum acceptable performance, taking account of any statutory requirements, and in the light of future operational requirements for the structure.
 - b) to agree future inspection, monitoring, or assessment procedures and intervals.

In short, to establish criteria for point B.

5. Procedures

A flow diagram is shown schematically in Figure 3, based on Sections 3 and 4, showing how to start and to proceed as far as the Preliminary Assessment stage (SISD rating). Figure 3 is general and may have to be adapted for individual cases; guidance on how to develop Figure 3 is given later in this Manual.

The key to these procedures is to focus on both the deterioration and the structure, even from an early stage and to minimise the amount of investigative work required early on. Consistent with that, there are three important stages, as shown in Figure 3.

- 1) Desk top study
- 2) Preliminary assessment
- 3) Detailed investigation (if considered necessary)

Calculations are recommended at all stages, as an aid to decision-making. In theory, an assessment might be stopped after the desk study, if conservative analyses indicate a considerable margin of safety and the rate of deterioration is low, in relation to the inspection intervals. A decision at this point will also depend on how much detailed information is available from records about the structure and on the availability of data from previous inspections and/or testing.

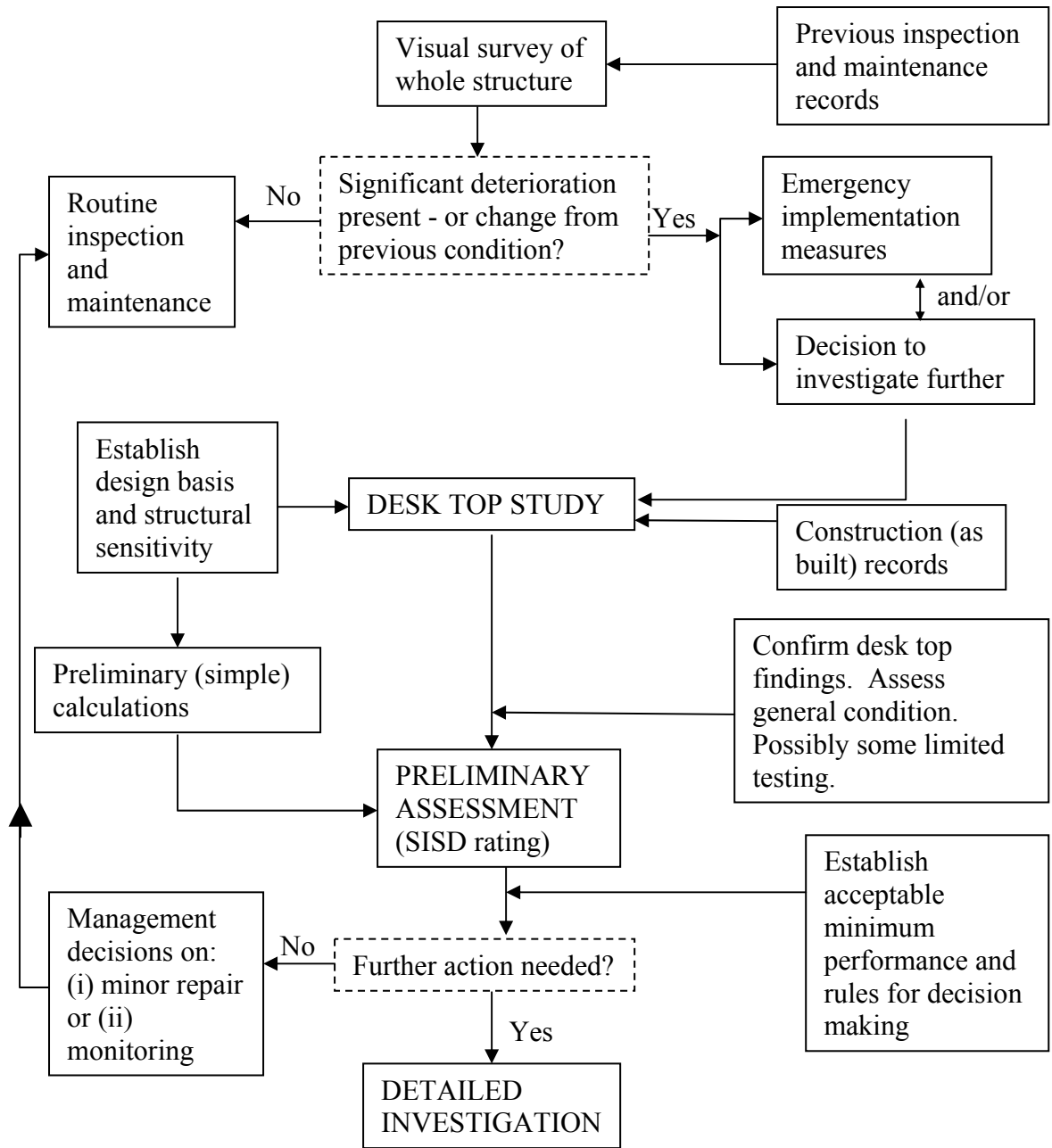


Figure 3: Flow diagram for progressive screening

More commonly, the first decision stage is at Preliminary Assessment. This is a qualitative approach to risk assessment, based on damage classification methods, with some effort made to predict future rates of deterioration. The prime purpose of the SISD rating is to prioritise actions when families of similar structures are involved and, in particular, to decide whether or not a full Detailed Assessment is required (see Section 7). However, if accompanied by some simple calculations, particularly on the residual resistance to key action effects, then it may be self-contained; in this Manual, this is designated the Simplified Method.

6. Synergetic effects

6.1 General

The format of the Manuals, listed in the Preface, is based on the assumption that preliminary investigations will identify the dominant deterioration mechanism and that all subsequent procedures follow from that (see Sections 3 and 5). However, the effects which this primary mechanism can produce may be exacerbated by defects due to other causes. Some examples are given in Section 6.2 below.

Moreover, two or more deterioration mechanisms may act simultaneously, and the combined effects may be more severe and require consideration. Some examples are given in Section 6.3 below.

Mainly, this is a question of diagnosis, in identifying primary cause from the observed effects and hence is assessing the current structural significance of the deterioration and, especially, the rate of its reduction in the future. For the examples in Section 6.3, more direct assessment of the combined effects may be required.

6.2 Defects which may influence the effects of a primary deterioration mechanism

Some examples are given in Table 3, in two separate categories. Category 1 defects tend to reduce the outer surface of the concrete, either physically or in terms of quality. Leaching can also increase the rate of carbonation or chloride penetration. Most concrete structures are subject to cracking at some stage in their lives. The examples listed in category 2 may:

- occur in different timescales
- be permanent or transitory
- be dormant (even healed) or live

Table 3: Some examples of defects and actions which may affect a primary deterioration mechanism

<i>Category 1</i> <i>Actions or defects affecting the concrete</i>	<i>Category 2</i> <i>Non-structural and structural cracking</i>
Weathering Abrasion Leaching Honeycombing Pop-outs	Plastic settlement Plastic shrinkage Early age thermal effects Long-term shrinkage Creep Ambient temperature -movement and restraint - internal temperature gradients Design loads Settlement Restraints -determinacy - non structural elements

6.3 Deterioration mechanisms acting in combination

Mostly one mechanism will dominate, but, in some cases, the effects of others may require consideration in combination. Some examples are given in Table 4. There is little experimental verification of these possible synergetic effects, but logic would suggest that they be considered, should early diagnosis indicate the significant presence of more than one mechanism.

Table 4: Some examples of synergy, due to deterioration mechanisms acting simultaneously

<i>Combination of mechanisms</i>	<i>Possible effects</i>
Surface sealing due to frost and corrosion	This may lead to a gradual reduction of the cover to the reinforcement and, hence, increases the likelihood of corrosion.
Alkali-silica reaction, and either frost action or corrosion	The expansive action of ASR may lead to wide cracks which can fill with water, and which, if frozen, may cause internal mechanical damage. This same action may also permit easier access to the reinforcement of water containing chlorides, causing more severe corrosion. On the other hand, gel caused by ASR may fill pores, thus densifying the cement matrix.
Leaching and frost action	The influx of water may increase the moisture uptake and, hence, reduce the internal frost resistance.
Leaching and corrosion	The leaching of lime from the concrete cover increases the rate of carbonation and the diffusivity of chlorides and reduces the critical threshold level.

7. Detailed investigation

7.1 General

If a Detailed Investigation is considered to be necessary, then the prime concern is with quantifying structural capacity i.e. in assessing the effect of the deterioration on strength, stiffness, stability and serviceability. This means having enough information available to:

1. fully understand the form and action of the structure;
2. interpret the effects of deterioration in structurally significant terms.

Deterioration can affect structural behaviour in a number of ways:

1. loss of section e.g. concrete spalling, corrosion of reinforcement (general or pitting)
2. reduction in mechanical properties e.g. in the strength of materials, or the stiffness and ductility of elements
3. excessive deformation (local or overall), thus inducing alternative distributions of load, or modes of failure, or rupture of critical sections.

In assessing the influence of these factors on structural capacity, it is important to note:

any particular level of deterioration (e.g. loss of rebar section due to corrosion) may influence bending, shear, bond, or other action effect, differently. It follows that each action effect (global and local) should be considered individually.

the influence of structural sensitivity on actual load capacity (e.g. the degree of redundancy; the influence of reinforcement detailing, etc.).

With regard to individual deterioration mechanisms, it should be noted that both ASR and frost action only affect the concrete directly - in terms of reduced cross-section, stiffness and reduced mechanical properties. In these cases, Detailed Investigation involved the derivation of modified (reduced) values for these properties, to be used in conventional design models for structural analysis, section strength and serviceability – where the concrete is deemed to make a contribution (see Section 3).

For corrosion, the situation is more complex. While the principles in the previous paragraph equally apply, it may also be necessary to check the validity of the design models in an assessment situation.

7.2 Minimum acceptable technical performance

The prime concern is with safety, either for the structure overall, or locally for individual elements, connections and sections. This is a matter for decision by individual authorities and owners, in deciding what is acceptable, relative to what was originally provided and to current acceptable standards – bearing in mind, the consequences of failure (see Section 8).

However, other performance criteria have to be considered, mainly under serviceability conditions. These would include:

1. a limit on cracking, due to the risk of serious local spalling, likely to be hazardous to life or property (a safety issue in some cases);
2. a limit on deflection, or other deformation, which might impair the function of the structure;
3. a limit on crack width, because of aesthetic or serviceability reasons;
4. a limit on expansion due to ASR, in the presence of restraints, in already highly-stressed sections;
5. consideration of synergetic effects, e.g. the influence of scaling due to frost action on an increase in corrosion rate.

The key point being made is that engineering judgement is essential, in interpreting the scientific data from investigations and testing regimes, in order to take sensible management decisions on what is critical and on when action is necessary.

8. Safety levels, risk, confidence levels, etc.

An owner may want a full reliability assessment taking account of variability and uncertainty in a general way, while recognising the stochastic nature of the many factors involved – in the deterioration processes at least, if not always in their effects on structural capacity. He may also wish to directly compare the assessed capacity with that provided in the original design by the use of traditional limit state design (semi-probabilistic, using partial safety factors).

Either way, there will be decisions to take on what is acceptable. Assuming that the same overall reliability is the norm, and taking the partial safety factor approach for purposes of illustration, then there is a case for lower values for the safety factors compared with design. The reasons for this are given in Table 5, which shows that, in general, more reliable information is available in assessment, compared with that in design.

Table 5: Design v Assessment: Significant Differences

<i>Item</i>	<i>Design</i>	<i>Assessment</i>
Material properties	Assumed	Measured
Dead loads	Calculated	Accurately determined
Live loads	Assumed	Assessed
Analysis	Code based	More rigorous alternatives
Load effects	Bending, shear compression, cracking dominate	Anchorage, bond & detailing may be more important
Environment	Assumed classification	Definition of macro-and micro-climates
Reliability	Code values for safety factors	Small factors for the same reliability

This might justify lower values for partial safety factors. On the loading side of the design (or assessment) condition:

$$S_d \leq R_d$$

this could be associated with progressively more rigorous analytical methods (see Section 1.3) when establishing safety criteria for point B in Figure 2. On the resistance side, reductions are again possible; however, this will depend on the action effect under consideration, since the basis for some design models is empirical, eg shear, and may not translate directly to the assessment situation.

It is not possible to make recommendations for reduced partial factors, which are generally acceptable. Each case has to be considered individually and there may be minimum statutory requirements for particular types of structure in individual countries, However, the principles behind Table 5 are valid and such an approach has been developed to some extent in some countries (eg, the Highways Agency assessment standards from bridges in the UK).

9. Asset Management

As stated in Section 2, structural assessment is an aid to decision-making as part of the asset management process and, as such, is an addendum to existing inspection, maintenance and management procedures. Table 1 indicates the type of management decision which may have to be made and Table 2 shows how assessment could progress as an aid in that direction.

So far nothing has been said on how to choose the most effective remedial action and, indeed, that is beyond the scope of the Manual. However, assessment and choice of remedial action are inter-related. Not only must the repair option be effective and compatible with the structural system, but also its expected life may influence the future inspection and assessment strategy.

This point is illustrated in Figure 4, which has been developed from Figure 2, with points A and B having the same meaning. If a decision is taken at point C to take remedial action and the choice is between repair options 1 and 2, restoring load capacity to level i) and ii) respectively, then the shorter life of option 2 would influence the interval between inspections.

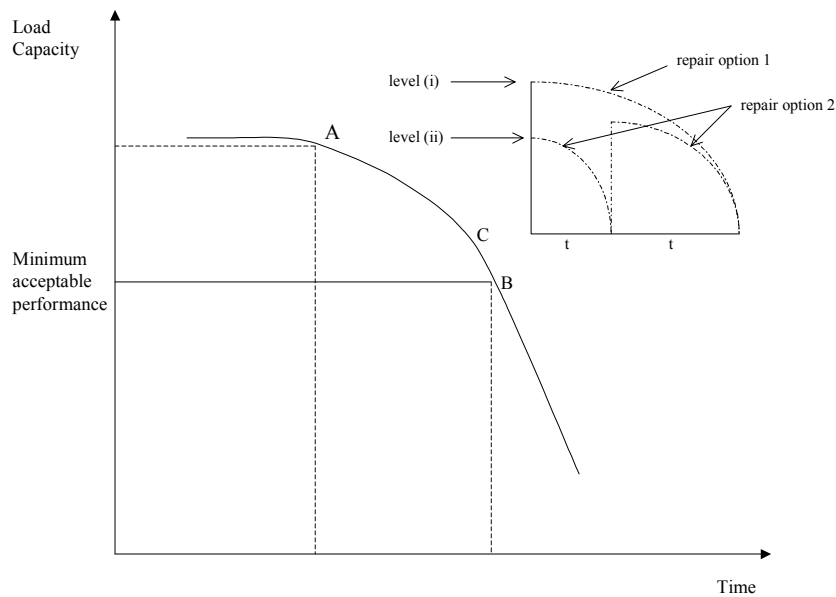


Figure 4: Schematic illustration of two different repair options

Although option 2 may be the cheapest in first cost terms, option 1 may be preferable in whole life costing terms, if the cost of disruptions, and having to repair twice in time $2t$, is taken into account.

The scenario in Figure 4 has deliberately been made simplistic and the situation will rarely be this straightforward in practice. It is included here to illustrate the interaction between assessment and remedial action - and to make the important point that any assessment method - whether simple or complex - must fit within asset management systems such as this.

PATCH REPAIR – SUMMARY by Jussi Mattila

Patch repair is a traditional way of repairing local damage in all kinds of concrete structures. The basic idea of the repair is to remove deteriorated concrete and expose corroding steel and then replace the removed concrete with cast concrete, shotcrete or repair mortar especially made for repairs.

Patch repairs are usually exposed to nearly the same loads and environmental and other stresses as the structure before the repair, particularly if no protection, like a coating, is applied as a part of the repair. Since the materials used in the repair work are usually cementitious like concrete, the repaired area will be subjected to nearly the same harmful processes as the surrounding structure.

It is important to note that the service life of a repaired structure does not depend merely on the durability of repaired areas but essentially also on the durability of other parts of the structure that have not yet been repaired. In many cases, the degradation of parts other than the repaired ones will determine the actual service life of repaired structures.

Practical experience as well as studies on some completed repair projects prove that the quality of the repair may often be poor. In many cases, insufficient removal of concrete seems to be the weakest part of the repair process. One of the reasons for the quality problems seems to be that patch repair is used where it is not appropriate. This is why some additional criteria should be established and adhered to in the future in the evaluation of the feasibility of patch repair. These criteria are related to quality assurance procedures, to the extent of present and future damage as well as to a reliable condition investigation. An essential precondition for the utilisation of patch repair is that the owner of the structure undertakes to arrange proper quality assurance. For the quality control to be possible, patch repairs should be utilised only when there is a very limited amount of damage to be repaired and the repair work is easy and simple. A good rule of thumb is that local repairing should be applied only to local damage. Prior to design, a careful condition investigation has to be always carried out to evaluate the feasibility of patch repair.

Another important part of the repair process is the technical design of the repair. This should include two important parts: 1) Determination of exact quantities to be repaired and 2) Specification of materials and methods as well as all other criteria accurately enough to ensure the intended quality. One of the most important things is that the location of repaired areas is instructed so that all areas in need of repair are located and opened. In the case of incipient corrosion damage the only rational procedure is to establish a clear cover depth limit on the basis of condition investigation and follow this criteria strictly by using a covermeter.

In the execution of patch repair there are several working phases that are all critical from the viewpoint of the quality of the result. Actually, the quality of patch repair is most strongly influenced by the workmanship on site. This is why the quality of the repair result cannot be assured just by including accurate specifications in documents. The fulfilment of the specifications has to be assured by continuous quality control on site.

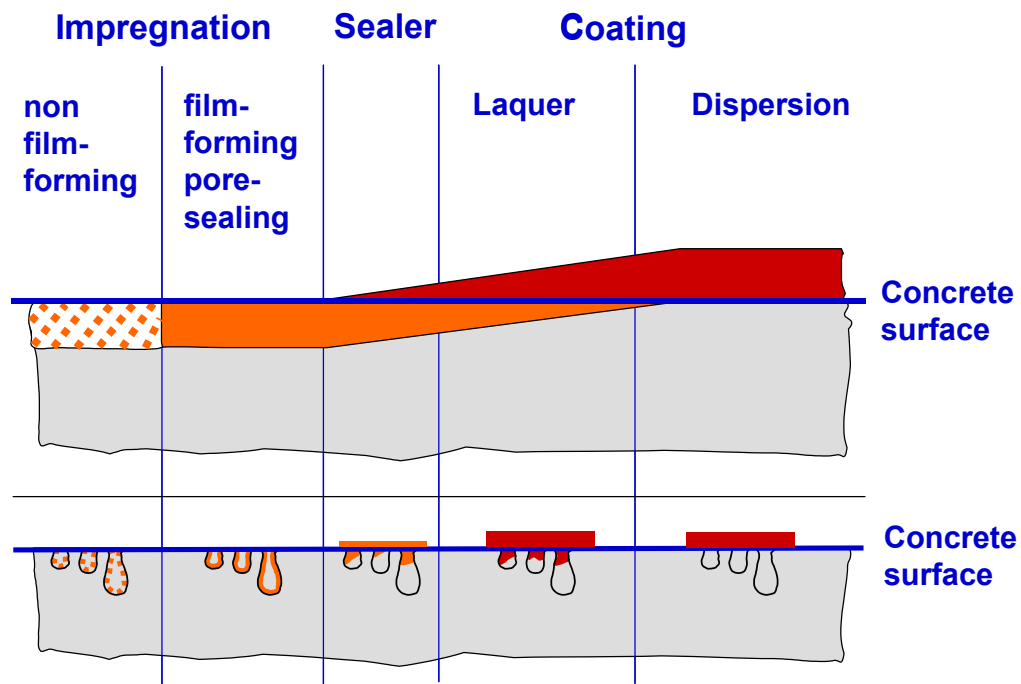
Norecon workshop:
Decisions and Requirements for Repair
3-4 April 2003

Surface treatment

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Provided suitable concrete quality, good design and workmanship concrete structures have excellent durability in most environments. But in some cases it is necessary to employ some form of surface protection to achieve the needed durability and, thus, service life. This may apply to extremely aggressive environments, or when the durability of the structure was insufficient from the beginning, for example as the result of a too thin concrete cover.

Different types of surface protection are used for different purposes and on different types of structures. They are often classified according to their ability to penetrate the concrete and the thickness of the layer they form on the concrete surface, FIG 1.



Rösli et al,1986

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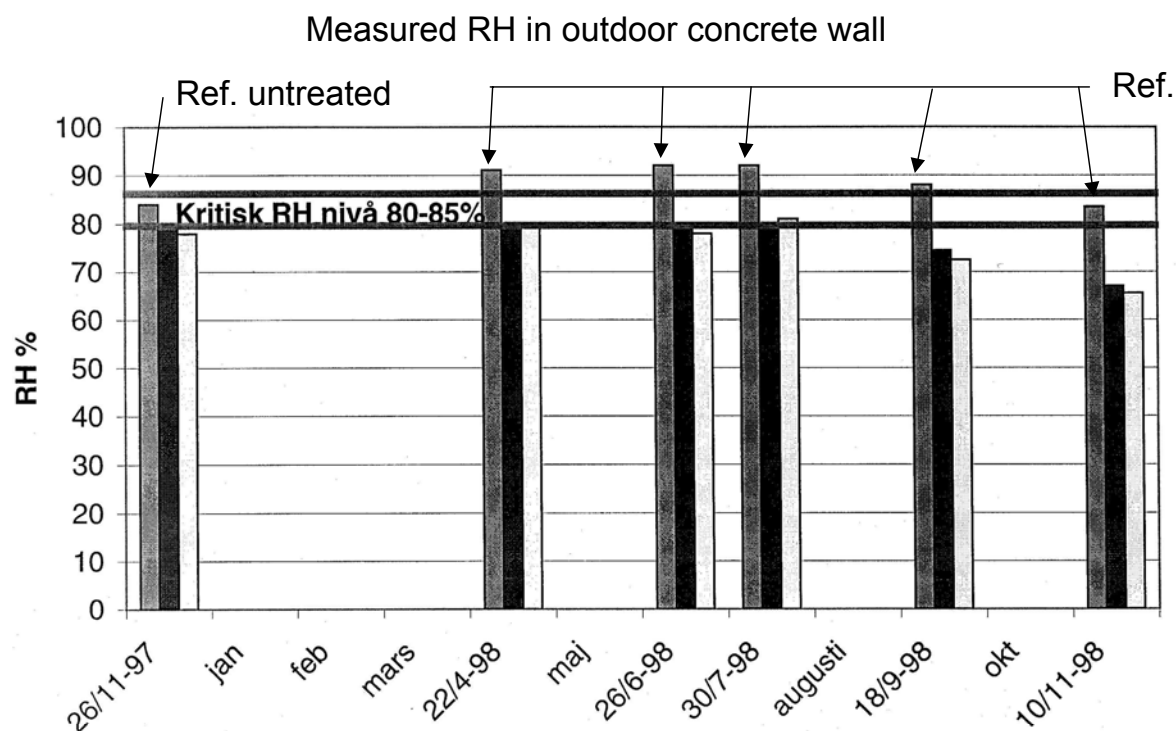
The effect of a surface protection is however also dependent on the type or types of material of which it is composed. Certain types have been used since long time ago, such as bitumen based products for bridge decks, foundation walls etc. The development of new types of products has led to an enormous increase in the situations, type and number of structures where surface protection treatments are used. The main type of binder in a product (often a polymer) normally rules the properties of the product. A bit more than twenty such different groups of products (or binder agents) are described in the literature. However in practice only a handful are more frequently used, in the Swedish market for example products based on

acrylic polymers, bitumen, cement, epoxy, silicates, silanes and siloxanes for outdoor structures.

Each surface protection type has its own characteristic properties. The variations between products within one and the same group, however, can be greater than those between different groups. When selecting a surface protection product it is therefore necessary to check that it really possess the intended properties. To achieve the intended performance it is also necessary that it be applied in the correct manner and that the concrete surface to which it is applied possess the correct prerequisites.

To get protection against a certain attack the product must possess certain properties. For example a certain degree of impermeability to carbon dioxide to prevent carbonation of the concrete. In addition to properties related to a particular protective function, the surface treatment must also possess a number of other properties to perform satisfactorily, for example adhesion to the substrate, a certain water vapour permeability, a certain ability to bridge cracks and durability in the environment to which it will be subjected. Certain functions and properties have been relatively well studied, but information about many important factors is lacking. Generally speaking the relationships between different properties of surface treatments measured in laboratory and their behaviour in reality are relatively little known. It is therefore generally difficult to evaluate the effect of a protective treatment in practice and its service life is even more difficult to assess. But it has been shown that with a suitable product correctly applied the service life of concrete structures can be prolonged.

Products for carbonation protection are typical examples on the assessment difficulties. The effect on the carbonation rate can be evaluated by calculations from the well-known square root equation based on Ficks 1:st law. Just put in figures on carbon dioxide permeability from the product information into the calculations often indicates that an enormous retarding effect on the carbonation rate will be achieved. However a carbonation retarding coating also will hinder access of liquid water to the concrete meaning that the concrete normally will be dryer, FIG 2.



This in turn means an increase of the diffusion coefficient for carbon dioxide in the concrete. Data on this effect is insufficient but it can be estimated in rough figures.

Added to this effect comes the fact that tests of carbonation protecting treatments made on concrete specimens usually shows a much higher carbon dioxide permeability than the values given in the product information. The differences which can be as much as 80-90 % are caused by differences in the test methods (product information values are normally based on tests where the product is tested on some other substrate than concrete). Finally the ageing effects are more or less unknown. Published results from ageing tests are contradictory.

Despite these shortcomings products of this type seems to work well in practice, i.e. the final effect, to delay the start of reinforcement corrosion seems to have been achieved. Some surface treated structures have been checked a number of years after surface treatment, at most up to 20 years, and there has been no sign of ongoing reinforcement corrosion. These structures suffered from reinforcement corrosion caused by carbonation. Local repairs were made and the surface treatments were meant to protect the undamaged areas. Without surface treatments new corrosion damages had been expected in a relatively short time. The absence of new damages could be an effect of carbonation prevention but it could just as well be an effect of dryer concrete.

Mechanical repair

The Sørsundet Bridge

Experiences after 40 years of exposure on the coast of Norway

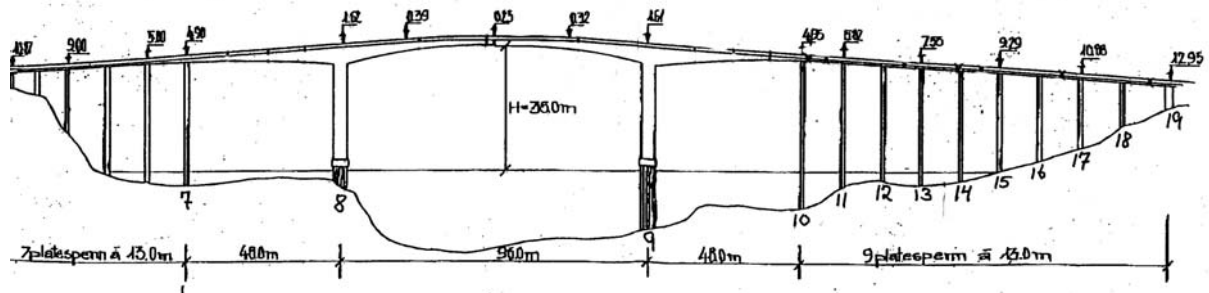


Fig. 1 : The Sørsundet bridge

1. Introduction

The Sørsundet Bridge is located in the city of Kristiansund on the west coast of Norway. The city is built on three islands and the bridge was the first road connection to the smallest of the three islands Innlandet. The bridge crosses the main route for ships coming to the city's harbour and the military and ship transportation authorities therefore demanded a free height of minimum 35m under the bridge. There were several alternative bridge designs considered to achieve this demands, but finally a cantilever design was chosen. The Sørsundet bridge was completed in 1963 and was the second cantilever bridge built in Norway. (The Tromsø bridge was completed a year earlier) This bridge design has since become a trademark for Norwegian bridge building technology. It is therefore interesting to take a closer look on one of the first cantilever bridges built in Norway and our experiences so far after 40 years of exposure in an aggressive chloride climate.

The Bridge Department in The State Roads Administration in the county of Møre og Romsdal in co-operation with the Bridge Maintenance Office, State Road Administration in Oslo have performed the inspection and maintenance routines.

2. Description of the bridge

The Sørsundet bridge has a total length of 400 meters. The superstructure is reinforced concrete plates in the side-spans and a cantilever design in the middle. On the north side of the bridge there are 7 plate spans of 13 metres . The cantilever superstructure have two main pillars with two symmetric cantilever arms of 48 metres. This makes two side spans of 48 metres and a main span of 96 metres. The free height under the main span is 35 metres. On the south side to the island Innlandet there are 9 plate spans of 13m.

The columns supporting the side-spans is circular with a diameter of 1,4 metres and the two main pillars is rectangular 4 x 4 metres .

The concrete is in the original documentation C35. The concrete cover is 40 mm underneath in the superstructure and 70mm in the columns.

3. Inspections

The bridge has now been exposed to aggressive marine environments in 40 years. We have planned and performed several major and special inspection on this bridge and have recorded an increasing degree of damage in our bridge management system BRUTUS in the last 5 years.

Inspections :

1. Major inspections - visual inspections from bridge lift
2. Major inspections underwater
3. Special inspections
 - levelling on the superstructure
 - corrosion investigations (ECP)
 - chloride-profiles
 - concrete investigations (electrical resistance, E-module, capillary absorption, porosity)

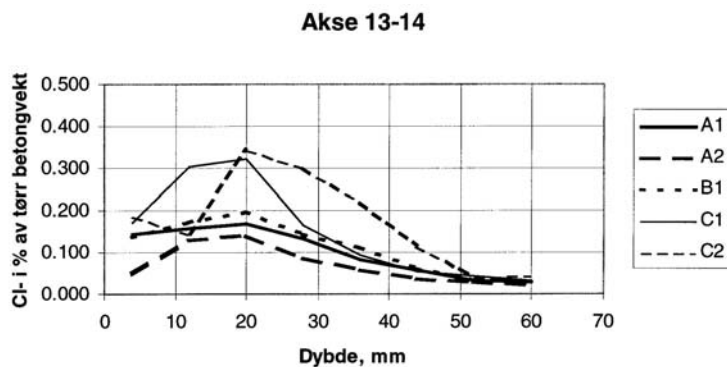


Fig. 2 : Chloride profile from the south side-spans (sept. 2001)

The bridge was designed to the loads and bridge design rules in 1960. To ensure that the bridge has sufficient load capacity to service traffic loads of the day, the capacity of the bridge construction is controlled.

4. Experiences from maintenance

We have performed several maintenance duties in the last ten years to repair damages and to increase the bridges lifetime.

- Underwater damages
Both the main pillars in the cantilever main spans and several of the columns in the side-spans have been repaired because of casting damages from the construction period.
- Railway guard
The original reinforced concrete poles is replaces with steel poles.
- Strengthening of the cantilever spans
Levelling of the superstructure recorded a deflection of 40cm in the middle of the main span. The design control concluded that this was caused by long time deflection and a too low moment capacity in the concrete box design. The cantilever spans was in 1999 reinforced by casting a bottom slab in parts of the cantilever spans.
- Mechanical repairs on the superstructure
In the last 5 years we have observed an increasing corrosion activity in the superstructure. On parts of the bridge, concrete parts up to 5 kg have fallen down. Since there is several building, crossing roads and a shipyard placed underneath the bridge, this is a serious security problem. So far there has been only material damages on cars and roofs. Corrosion investigations and chloride profiles shows that this is caused by chlorides from the sea. This is not a local problem on parts of bridge but is an increasing problem for the entire superstructure.

In 2002 we performed a mechanical repair on the superstructure and 6 m³ of concrete was repaired. In our procedures we chose to repair all local damages in the concrete cover layer to prepare the superstructure for a cathodic protection system. If we had chosen to remove all concrete exceeding an upper limit of chloride content, the repair volume was calculated to 20m³. This would have forced us to reduce traffic loads during work and to calculate all consequences for the load capacity on each of the larger repair areas. We considered this operation to be too insecure, too costly and with no long time guarantee for further chloride and corrosion problem. We decided to repaired all the local damages in the cover zone from rebar corrosion and to repaired all damages in the superstructure from the original casting of the bridge in 1963 in order to get a homogeneous concrete cover. We are now planning to send out a tender for protecting the superstructure with a cathodic protection system in 2004.

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Realkalisation and Chloride Removal

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Corrosion of reinforcing steel represents the main cause of maintenance of reinforced concrete structures. The majority of maintenance costs is spent on breaking out concrete and conventional repair methods. Over the last decades, electrochemical methods for protecting reinforcement in concrete structures have become available. When these electrochemical maintenance methods are applied, it is not necessary to remove large parts of the concrete cover, only those parts that have already cracked or spalled. This is different from the need to remove much of the chloride contaminated or carbonated concrete with conventional repair, even when still physically sound.

All electrochemical maintenance methods have principles and practical details in common. The main differences are the amount of current flowing through the concrete and the duration of the treatment as given in Table 1 /1/.

Table 1 Main differences between electrochemical maintenance methods

Method	Duration	Typical current density
Cathodic protection	Permanent	10 mA/m ²
Realkalisation	Days to weeks	1 A/m ²
Chloride removal	Weeks to months	1 A/m ²

By means of an external conductor, the anode, a direct current is flowing through the concrete to the reinforcement which thereby is made to act as the cathode in an electrochemical cell. The final result of the current flow is to stop the corrosion by removing the aggressive ions (chloride) from the pores of the concrete (chloride removal) or by reinstating the alkalinity of the pore solution (realkalisation). A schematic illustration is shown in figure 1.

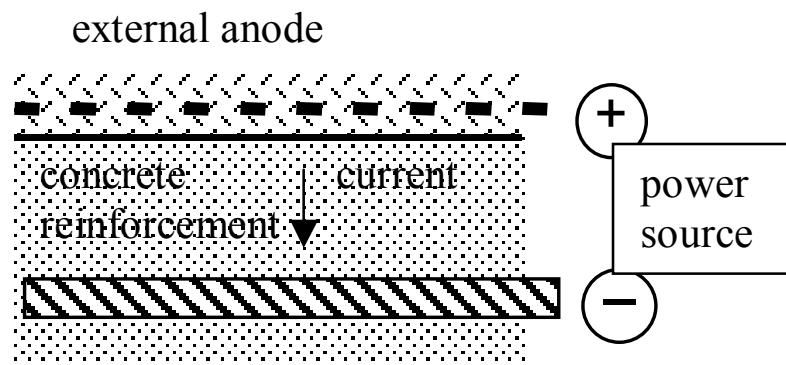


Figure 1 Schematic illustration of electrochemical maintenance methods /1/

Electrochemical realkalisation and chloride extraction are more recent methods that already have made a great impact. The current is applied for a limited time and after the treatment period, the anode system is removed from the structure. The concrete is left behind with reinstated protective properties, which may last to be protective for many years. Of course, further chloride ingress or carbonation has to be eliminated.

Common for the electrochemical methods is that they can be applied only on structurally safe structures. One of the main advantages is that they require only the removal of spalled and delaminated concrete, while mechanically sound but chloride contaminated or carbonated concrete may be left in place. Consequently less material has to be removed as compared to conventional repair, causing less noise to be produced and possibly resulting in shorter execution times. In specific cases temporary structural support during concrete removal and repair, needed with conventional repair, may not be necessary with electrochemical methods.

The following processes take place:

Electrolysis

Current flow in concrete causes electrochemical reactions at the electrodes (anode, cathode), including electrolysis. These reactions produce hydroxide ions at the cathode. This causes an increase of pH in the pore water close to the reinforcement, thereby contributing to both repassivation of the steel and to a higher critical chloride content (the amount of chloride that initiates corrosion). The magnitude of electrolysis and the relative contribution of specific reactions depend on the current density or the applied voltage. If the potential becomes very negative, hydrogen atoms may be produced, invoking the risk of hydrogen embrittlement of high strength steel. The production of hydroxide at the steel is one of the effects and objectives of realkalisation.

Electromigration

Negatively charged ions will move from the steel towards the positive electrode (the anode) and positively charged ions move towards the negative electrode, the reinforcement (Figure 2). Together these flows of ions carry the current through the concrete. The ions participate in this process in a relative proportion to their transport numbers. The transport number of a given ion is a function of its mobility and concentration. In concrete pore water most of the current is carried by hydroxyl ions and alkali metal ions and by chloride ions when present in significant amounts. Electromigration will always occur when current is passed through concrete. Its effects are beneficial to the steel. Transport of chloride from the steel to the anode is the main effect and objective of chloride extraction.

Electro-osmosis

Electro-osmosis is transport of liquid in a porous material due to an applied electrical field. The rate of transport depends on the properties of the liquid, those of the solid and the potential applied. Electro-osmotic flow of alkaline liquid into concrete is a beneficial process, because it assists repassivation of the steel, which is one of the main effects of realkalisation. Electro-osmosis in concrete has been documented during realkalisation of carbonated concrete; it does not seem to occur in alkaline concrete, for which the reasons are not completely understood.

Preliminary investigation

A preliminary investigation of the structure should include a general survey, identifying the presence of structural cracks, deformations and other obvious defects. If such defects are present to a significant level, the treatment should be reconsidered and structural repairs carried out. Where structural repairs are not necessary, the inspection should focus on the preparation for electrochemical treatment:

- Concrete cover to the steel
- Chloride content and distribution
- Depth of carbonation
- Electrical continuity of the reinforcement

- Electrical continuity of the concrete
- The presence of potentially alkali-reactive aggregates
- The presence of prestressing steel

Possible side effects induced by electrochemical methods

- Alkali-silica reaction
- Concrete degradation by the produced acid
- Adhesion loss
- Hydrogen embrittlement of prestressing steels

Investigations show that only hydrogen embrittlement of prestressed steel without ducts should be seriously considered. On such structures chloride removal or reralkalisation should not be applied.

Reference

COST 521: “Corrosion of Steel Embedded in Concrete”, Final report, Ed. Romain Weydert, Luxembourg, 2002

Stainless Steel Reinforcement:

Why and when should we consider its application for repair of concrete structures?

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1. General about stainless steel

Definition of stainless steel: All steel alloys with content of chromium more than 12 % are called stainless steel. Due to specified requirements of corrosion protection it is necessary to specify stainless steel in more details.

Normally stainless steel is divided in to 4 groups, all including a large number of alloys with the characteristics shown in table 1 (1).

Table 1: Groups of stainless steel

Type	% Cr	% Ni	% Mo
Martensitic	-	-	-
Ferritic	12-19,5	0	0
Austenitic	18-26,0	8-21,0	2-4,0
Austenitic/Ferritic steel named "Duplex"	21-28,0	4-6,0%	1,5-6,0

The lowest alloyed martensitic steel has no relevance as reinforcement. Ferritic steel has found limited application in construction industry. Ferritic steel should not be considered in extreme aggressive corrosion environments like in the Gulf region. The most common used stainless steel reinforcement is manufactured from the austenitic steel alloys. Due to the price decrease of stainless steel also duplex-based alloys are now used as the concrete reinforcement. The reinforcement based on duplex steel is primarily used in the very aggressive corrosion environments, like previously mentioned Gulf region.

2. Why to use stainless steel as reinforcement in concrete ?

Corrosion properties of stainless steel: Steel reinforcement embedded in concrete will not normally corrode due to the formation of a protective ion oxide film, which passivates the steel in the strongly alkaline conditions of the concrete pore water. This passivity can be destroyed by chlorides penetrating through the concrete and due to

carbonation. Corrosion, which is an electrochemical process involving establishment of corroding and passive sites on the metal surface, is then initiated.

The corrosion resistance required for use in concrete is primarily resistance against localized corrosion (pitting, crevice corrosion) in chloride containing environments. This resistance depends on the alloying elements of chromium, molybdenum and nitrogen. Whereas chromium is the main alloying element, molybdenum and nitrogen have more effect on the localized corrosion resistance. In order to compare stainless steel grades with different alloying, correlation of the influence of the different elements has been made resulting in the expression of pitting resistance equivalent (PREN). This expression can be considered as a relative measure of the total resistance resources for the steel grade and thus a comparable value. The expression is calculated from the content of the alloying elements in the steel grade (2).

For austenitic steels the expression is

$$\text{PREN} = \% \text{Chromium} + 3,3 * \% \text{Molybdenum} + 16 * \% \text{Nitrogen}$$

For duplex steels the effect of nitrogen is considered higher resulting in the expression

$$\text{PREN} = \% \text{Chromium} + 3,3 * \% \text{Molybdenum} + 30 * \% \text{Nitrogen}$$

This expression can be considered as a relative measure of the total corrosion resistance resources of the steel grade and thus as comparable value between different steel types. PREN values are presented together with price comparison and weldability in table 2.

Table 2: PREN Numbers and corresponding relative price and weldability

Grade	Price compared to carbon steel	PREN	Weldability
Carbon Steel (mild)	1,0	<1	Excellent
1.4301, AISI 304	4,5	19	Excellent
AISI 304 LN	4,5	19	Excellent
1.4401, AISI 316	5,5	25	Excellent
1.4429, AISI 316 LN	5,5	25	Excellent
1.4462, AISI 318	5,5-6	34	Good

In general stainless steel rebar grades can be considered for the following working environments:

1.4301, AISI 304 / 1 / LN: inland, low chloride containing environments

1.4401, AISI 316 / 1 / LN: coastal and high chloride containing environments

1.4462, AISI 318, DUPLEX: high chloride containing and extreme environments

Resistance to galvanic corrosion: Stainless steel freely exposed to seawater may, if in galvanic contact with less noble metal such as carbon steel, initiate a rapid galvanic type of corrosion of the less noble metal. The otherwise slow cathodic oxygen reduction at the stainless steel surface is catalysed by a bacterial slime, which forms after a few weeks in seawater.

When stainless steel is cast into concrete, however, the cathodic reaction is a very slow process, since no such catalytic activity takes place on a stainless steel surface. A research project conducted at the FORCE Institute has indicated that the cathodic reaction is inhibited on stainless steel embedded in concrete, as compared to the cathodic reaction on carbon steel reinforcement in galvanic contact with corroding carbon steel (3). Publications of Pedefferri et.al (4,5) and Jägi et. al. (6) provides also results, which confirmed the above-mentioned findings.

This behaviour, and the fact that stainless steel is a far less effective cathode in concrete than carbon steel, makes stainless steel a useful reinforcement material for application in repair projects. When part of the corroded reinforcement, e.g. close to the concrete cover, is to be replaced, it could be advantageous to use stainless steel instead of carbon steel. In being a poor cathode, the stainless steel should minimize any possible problems that may occur in neighbouring corroding and passive areas after repair. The experimental results (fig.1) where the carbon steel (initially passive and later corroding) was connected to stainless steel, has confirmed this behaviour (7).

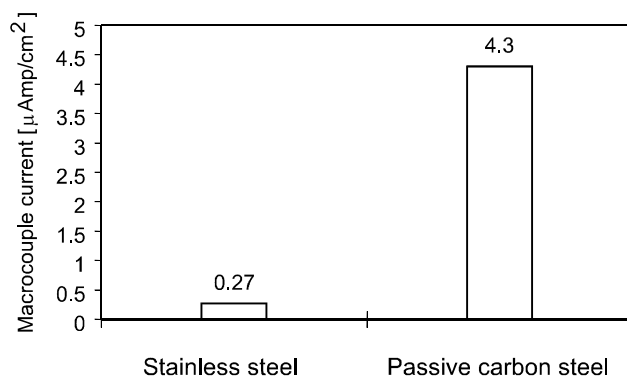


Fig. 1: Macrocouple current for stainless steel and passive carbon steel

When the current is measured between the carbon steel rebar starting to corrode and a rebar of carbon steel that is still passive, a current density value of approx. $4.3 \mu\text{A}/\text{cm}^2$ is registered. If the same corroding carbon steel rebar is connected to the stainless steel with the surface area equal to the passive carbon steel, the measured current density value is reduced to only $0.27 \mu\text{A}/\text{cm}^2$. This means a reduction in current density by a factor of approx. 15, which will result in the same decrease in corrosion rate.

At the same time, it is very important for the intelligent use of stainless steel that it be combined with carbon steel in proportions that guarantee both an optimal performance and cost-effective solution.

Stainless steel is therefore an excellent material to use for all components, which are only partially embedded in concrete, especially connected to the reinforcement. Examples are bolts, binders, ladder rungs, inserts, electrical connectors, sanitary piping and bushings.

3. When to use stainless steel reinforcement ?

General remark: It seems to be a fact, that most of civil engineers have an unfounded fear of using stainless steel and carbon steel together in the same concrete structure. In Denmark, FORCE Institute (The former Danish Corrosion Centre) has given advice to more than 100 clients on the use of stainless steel in concrete. Nearly always the clients had to be convinced, that it is in the fact good and safe practice to use stainless steel in the most chloride exposed concrete, with the stainless steel in good -often-welded - connection with the carbon steel in the main reinforcement.

This happens because engineers do not distinguish between the working environments, namely atmospheric air or steel embedded in concrete. In many cases this will lead to very high alloy types are specified, and consequently the material is either not available or far too expensive.

Some design aspects: There are several conventional options open to the designer when long life is required or corrosion is anticipated. At the head of the corrosion prevention table are good design, good site practice and quality control. Contributory to these requirements are details such as adequate concrete cover, maximum water/cement ratio, high cement content, using great care with any additives and adequate compaction.

Consideration of environmental and design factors will produce different solution for individual projects in order to avoid this dangerous situation. Cases that difference between normal reinforcement with high quality concrete and good cover, or, a corrosion free reinforcement system with less cover and acceptance of lower quality concrete on site, are a matter of engineering judgement (8).

4. Why to use stainless steel reinforcement in repair of concrete structures ?

Concrete structures deteriorated by corrosion can benefit from replacing the damaged reinforcement with stainless steel reinforcement. It is because the damaged - and thus highly exposed - part of the structure is repaired with a non-corroding reinforcement, hence solving the corrosion problem locally.

Additionally, as mentioned before under item regarding the galvanic corrosion, the risk of developing macro-cell corrosion in the neighbouring carbon steel reinforcement not being replaced is reduced. It is because the cathodic reaction on stainless steel embedded in concrete is inhibited, as compared to the cathodic reaction on ordinary carbon steel reinforcement in galvanic contact with corroding normal carbon steel. This is the reason why it has become attractive to replace corrosion damaged carbon steel reinforcement in highly exposed concrete members with stainless steel reinforcement, see figure 2.

"Repair and Maintenance of Concrete Structures, NORECON Workshop, Decision and Requirements for Repair, 3-4 April 2003, Lunds Technical University, Lund (S)



Figure 2: Repair of damaged bridge due to chloride initiated corrosion. Stainless steel reinforcement shall replace corroded carbon steel reinforcement.

5. Examples with application of stainless steel reinforcement

A number of specific examples of applications where stainless steel reinforcement has been used for both conventional concrete structures and for general supports are listed below. This list is not presented in the chronological-, but in the random order. It shall be emphasised that stainless steel has been used in many further projects, which names are not available for the author of this report.

Examples of constructions with stainless steel reinforcement:

- Bridge Deck Reinforcement, Trenton, New Jersey,
- Progresso Pier; Mexico (9)
- Rock Anchors A55, North Wales,
- Foundation Supports, Mansion House, London,
- Scarborough Spa, marine application
- Val de Grace, rebar MRI application
- Sydney Opera House, promenade, marine application,
- Manchester Airport, dowels/slab,
- Tie bars with couplers, bridge strengthening
- Emmanuel College, Cambridge - posttensioned bars
- Thames Bank at Wapping, brick faced precast concrete panels
- M4 Motorway Reconstruction- Slough/Maidenhead/Berkshire, bridge repair
- Mersey Tunnel, replacement of corroded reinforcement
- Cambridge University/Bio-Technology Laboratory, precast facade panel and basement

- Guidhall Yard East, conservation of the historic building
- St. Paul's Cathedral, conservation of historic building
- Bridge on Highway 407, Toronto, Ontario, reinforcement in bridge deck and reinforcing bar in the parapet wall.
- Oland Bridge, Sweden, replacement of the corroded reinforcement

Great Belt Connection, Denmark, some parts which need to protrude from the surface of the concrete, e.g. earthing rods and wires for making other electrical connections to reinforcement.

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**Summary of a paper presented by Ervin Poulsen at the
Nordic Network on Repair and Maintenance of Concrete Structures
NORECON in Lund, Sweden 2003**

Strengthening of RC components with fibre composite materials. State-of-the Art Report

There has always existed a need for the strengthening of structures. Since 1967 steel plate bonding has been dominating, but since approx. 1990 the epoxy-bonded CFRP laminates have increased their share of the market. Today the strengthening of RC structures with CFRP laminates is almost the predominant system of strengthening.

Carbon fibre composite materials have been on the market for building materials in the Nordic countries during the last approx. 10 years. During that period of time the fibre composites and the types of adhesives have developed their properties, e.g.:

- New types of composite materials have been developed.
- Anchorage devices for the anchorage of CFRP laminates (Carbon Fibre Reinforced Polymer laminates) have been developed.
- Improved types of adhesives have been developed, i.e. adhesive less sensitive against humidity and temperature.
- Post-tensioning of concrete structures has been developed.
- Documentation of types of insulation against fire.

The application of fibre composite materials for strengthening has developed since the first reinforced concrete structure was strengthened by means of epoxy bonded CFRP laminates (*Ibach bridge* near Luzern in Switzerland in 1991). Both properties, design techniques and execution of work have developed.

Almost all types of structural components can be strengthened by epoxy-bonding of fibre composite materials. A great variety of fibre composite materials are available today. Thus, plane structural components like beams, slabs and walls as well as curved components like columns with circular cross-section, arches, silos and double-curved shells may be strengthened.

Internationally, a large number of structures have been strengthened. The increased application has caused a reduction in the cost of CFRP laminates and during the last 12 years the cost has slumped to about 5 % of the cost at the time of introduction. The strengthening by means of fibre composite materials has been very successful and solved a very important problem: the society wants to utilize the existing concrete structures even when the structures become aged. Thus, the method of strengthening structures by means of epoxy bonded fibre composite materials is here to stay.

The paper describes the laboratory tests carried out in order to develop anchorage devices for CFRP laminates which are epoxy-bonded to concrete for the purpose of strengthening against failure caused by bending, shear, torsion and compression.

The structural behaviour and the mechanism of failure of structures strengthened by means of fibre composite materials are explained and compared with observations made during the laboratory tests in order to develop a mathematical model of the bearing capacity of structural components strengthened by fibre composite materials. ■

Mapping Corrosion of Steel in Reinforced Concrete Structures

Tang Luping, SP, Borås

The corrosion damage is one of the big problems and the cost of repairs is a considerable part of the annual budget of the Swedish National Road Administration. The engineers need a rapid method to accurately map the corrosion of steel in the concrete structures in order to make a proper plan for repair and maintenance work. The half-cell potential measurement as described in ASTM C 876 is the simplest electrochemical method. This method can, however, only give some information about the risk of corrosion, but cannot tell if the steel is really corroding or not. There exist limited types of commercial instruments for the field measurement of corrosion rate, but different instruments give different corrosion rates and the differences can sometimes be larger than one or two orders of magnitude! With such large differences it is difficult for engineers to practically use these commercial instruments in the inspection work. Therefore, SP carried out a project under the financial support from the Swedish National Road Administration to develop a reliable rapid method for mapping corrosion of steel in concrete structures.

In the project an instrument composed of computerised galvanostatic supplier and data acquisition system has been developed at SP for electrochemical measurement (Fig. 1). With the help of this instrument, different measurement conditions and parameters could be evaluated and many electrochemical measurement data could be collected for later analysis. A numerical model based on a 2-D FEM (2-Dimensional Finite Element Method) has been established for modelling the corrosion measurement (Fig.2). With the help of this model, the measurement parameters could be optimised and the effectively confined current could be evaluated (Figs. 3 and 4). Based on the results of numerical modelling and the studies on the small and large reinforced concrete slabs, a rapid method for measuring corrosion rate has been developed. The method involves a short time galvanostatic pulse measurement followed with the numerical calculation for correcting the preset polarisation current from the measured apparent polarisation and ohmic resistances, so as to produce “true” resistance values related to the confined area. Owing to its rapidity (in a few seconds per measurement), this method provides a useful tool for mapping corrosion rate of reinforcement steel in concrete structures. The results from a comparative measurement on both small and large reinforced concrete slabs show that the corrosion rate measured by the new rapid method is quite comparable with that measured by the Spanish Gecor instrument (Fig. 5), which uses the modulated confinement technique. The results from the field measurements on old concrete bridges also show that the corrosion extent measured by the new rapid method is in good agreement with the visual observations (Figs. 6 to 8).

Very recently, a prototype of handhold instrument has been developed. The instrument is very simple to use – no special maintenance or preparation work is needed. The measurement is automatic – no pre-setting of complicated measurement parameters is needed. Each measurement takes only a few seconds (about 5 seconds if the steel is in passive status or at a low corrosion rate, and about 10 seconds if the steel is significantly corroded). Therefore, it is very suitable for the field application by structural inspectors, engineers and also researchers without special knowledge of electrochemistry.

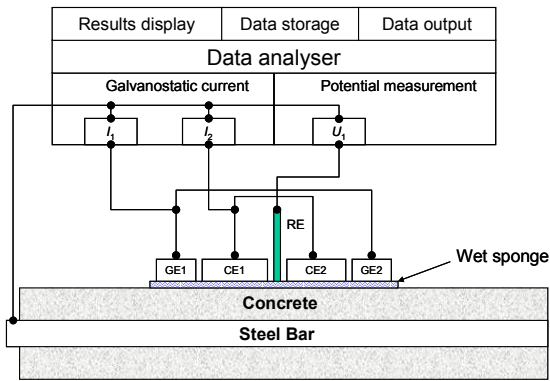


Fig. 1. Schematic of the SP Rapid Method for corrosion measurement.

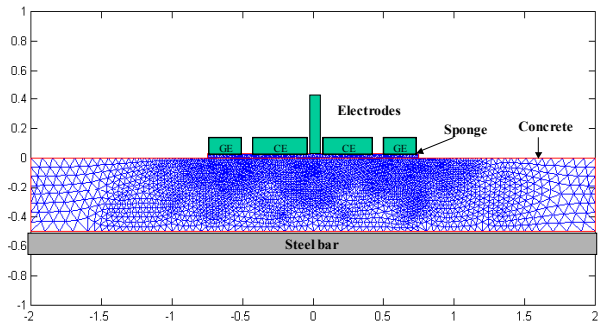


Fig. 2. Numerical modelling using 2-D FEM.

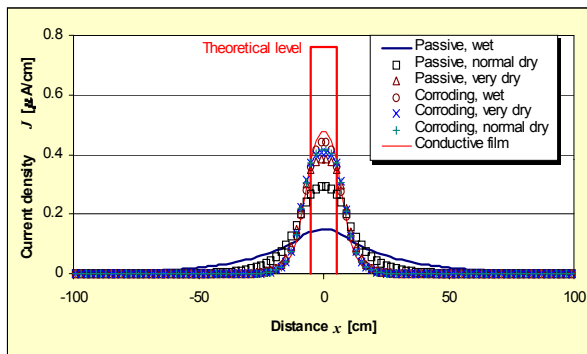


Fig. 3. Current distributions without guard electrodes.

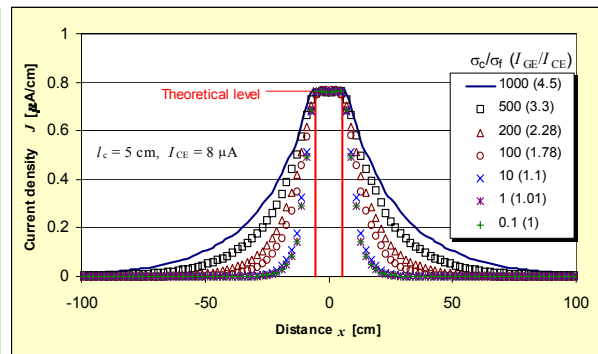


Fig. 4. Current distributions with guard electrodes.

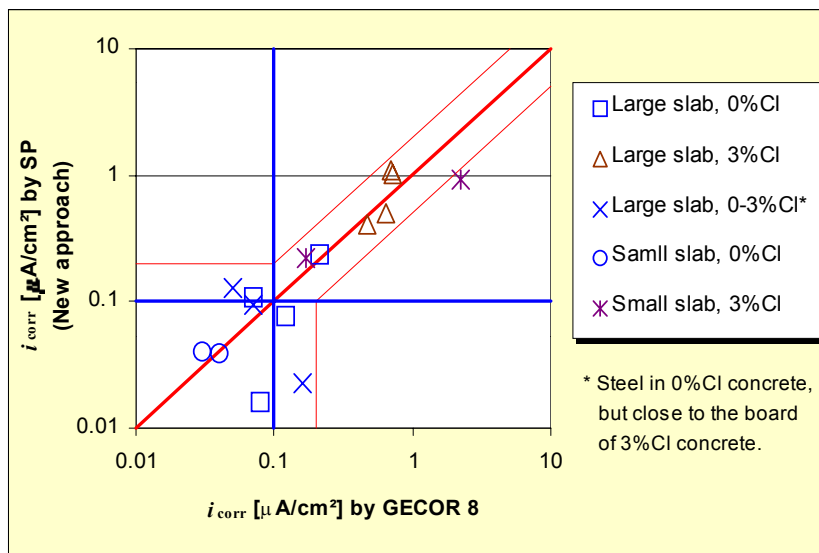


Fig. 5. Comparison in measured corrosion rate with the Spanish Gecor instrument.



Fig. 6. Bridge N29 on Road Lv 845, Åskloster.

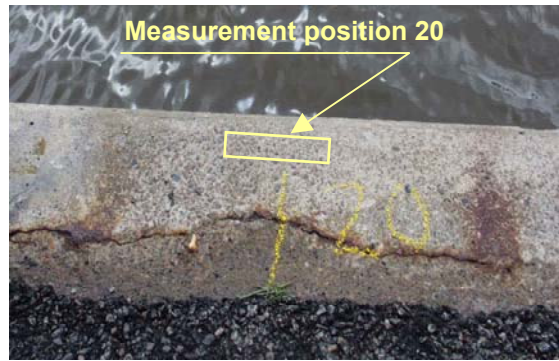


Fig. 7. Cracks and rust spots on a side beam.

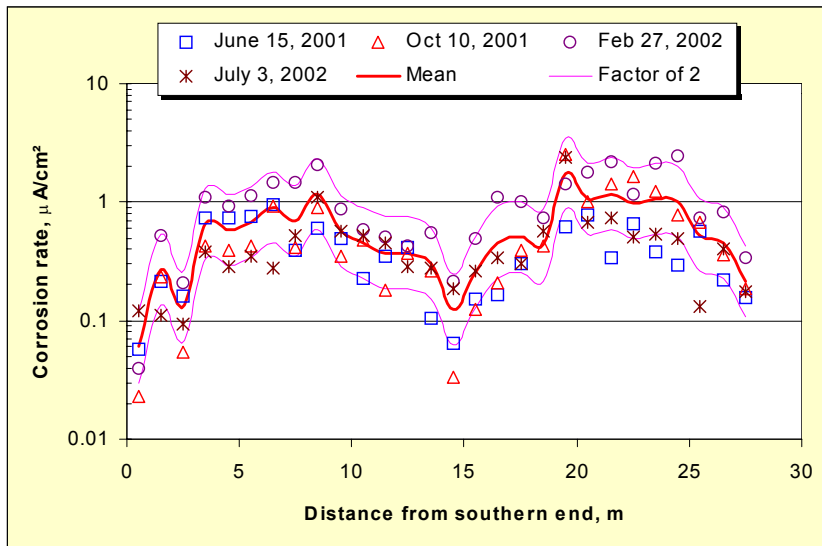


Fig. 8. Map of corrosion rate on a side beam of Bridge N29.

“Repair and Maintenance of Concrete Structures”

NORECON workshop:

Decision and Requirements
for Repair

3 – 4 April 2003

Lund Technical University
Division of Building Materials
John Ericssons väg 1, Lund,
Sweden

**a Nordic network on concrete research
and development**

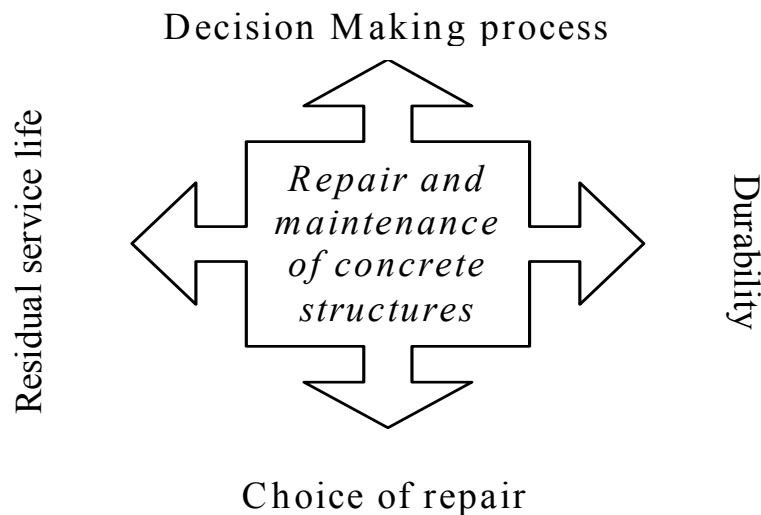


Objective

The objective of the workshop is to provide and disseminate the existing knowledge about the decision and requirements for repair of concrete structures.

Maintenance and repair costs now constitute a major part of the current costs of the European infrastructure. It has been estimated that the inspection and maintenance costs for the European infrastructure are approx. 5 billion ECU per year. The traffic delay costs due to inspection and maintenance is already estimated to be around 15% to 40% of the construction costs.

The choice of the most efficient maintenance strategy is very much dependent on the actual condition of the structure and the desired residual lifetime. The decision-making is based on results from a structural assessment, an environmental survey and existing service life models, which utilise the findings for prediction of the expected damage.



Norecon

Norecon is a Nordic network within the field of repair and maintenance of concrete structures. The main objectives of the network are to bring the results of research and best practise into the industry in the Nordic countries. The network includes researchers, developers, manufacturers, consultants, contractors and end-users.

The network seeks to meet the objectives by means of:

- Knowledge transfer
- Promotion of co-operation

The networks tasks are to collect, organise, present and disseminate knowledge on repair and maintenance of concrete structures. Also, the development of lasting personal contacts is crucial to secure future co-operation for the benefit of all parties.

PROGRAM

Thursday April 3, 2003

- 10.00 **Opening**
Oskar Klinghoffer, FORCE, DK
- Morning session, Chair: Jacob Mehus
- Owners requirements and experiences**
- 10:15 Christian Bernstone, Vattenfall Utveckling, S
On the requirements valid for existing concrete structures within the Swedish hydropower industry
- 10:45 Knut Grefstad, Statens Vegvesen, N
Alternative methods for repairing concrete bridges. Key factors for making the right decisions
- 11:15 John Bjerrum, Vejdirektoratet, DK
Visions about future repair works
- 11:45 Oddur Hjaltason, Linuhönnun Consulting, IS
Concrete repairs in Iceland
- 12:15 Gro Markeset, Forsvarsbygg, N
Owners benefits of selecting repairs following a reliability based service life evaluation
- 12:45 **Lunch**
- Afternoon session, Chair: Oskar Klinghoffer
- 13:45 Magne Maage, Selmer Skanska, N
European standards for protection and repair of concrete structures
- Condition assessment and monitoring**
- 14:30 Göran Fagerlund, Lund Inst. of Techn., S
General aspects and Contecvet project
- 15:00 Asger Knudsen, Rambøll A/S, DK
Inspection of concrete structures – the holistic approach
- 16:00 Coffee break
- Repair methods**
- 16:15 Jussi Mattila, Tampere Univ. of Techn., FIN
Durability of patch repairs
- 16:45 Lars Johansson, Cement och Betong Inst., S
Surface treatment
- 17:15 Jørn Arve Hasselø, Statens Vegvesen, N
Mechanical repair of chloride damaged structures
- 17:45 **Round off**
- 19:00 **Dinner**

Friday April 4, 2003

- Morning session, Chair: John Miller
- Electro chemical methods**
- 08.30 Allan Andersen, COWI A/S, DK
Cathodic protection – practical experience of different anode types
- 09:15 Øystein Vennesland, NTNU konst. teknik, N
Realkalization and chloride removal
- 10:00 Coffee break
- Corrosion inhibitors**
- 10:15 Gisli Gudmundsson, Icelandic Build. Research
Corrosion inhibitors for increased durability of concrete structures
- 10:45 Harald Justnes, SINTEF Bygg og miljø, N
Field experiences
- Cost estimate**
- 11:30 Trygve Isaksen, Norconsult AS, N
Cost estimate for repair and maintenance of remaining service life – Examples from real projects
- 12:15 **Lunch**
- Afternoon session, Chair: Göran Fagerlund
- Alternate reinforcing materials and strengthening of the structures**
- 13:15 Oskar Klinghoffer, FORCE, DK
Stainless steel reinforcement – why and when should we consider its application for repair of concrete structures?
- 14:00 Ervin Poulsen, E. Poulsen ApS, DK
Strengthening, overview
- 14:30 Björn Täljsten, Skanska Teknik, S
Strengthening by CFRP, overview
- 15:00 Coffee break
- 15:15 Steen Rostam, COWI A/S, DK
The necessity of integrating structural engineering, materials technology, sustainability and social aspects when selecting repairs
- 16:00 **Closing and announcement of the Norecon seminar**

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