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Editor: L.G.W. Verhoef

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- Problems and Possibilities -

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Renovation and Maintenance

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INTRODUCTION ON CONCRETE

Concrete as a conglomerate of sand, stone and a binder, is a very old material indeed. In the Roman period earth from Puozzoli, together with lime and water could bind the sand and the stones to form a conglomerate that has an affinity to our modern concrete. Later, in the more northern areas of Europe, the use of trass, plus water for a reaction, also produced cement. The Romans proved that they could build in a durable way with earth from Puozzoli, because even today we can still enjoy the aqueducts built all over Europe and the water reservoirs in Rome, which are still functioning. From building specifications for the Basilica of Constantine and the Pantheon, both in Rome, we know that cost could be reduced by using a mixture of stone fragments, Puzzuoli earth, lime and water for the inner parts of the walls. For the inner part of the vaults, the mass is varied in a very subtle way by using lighter stones for the top of the vault. The extremely long lifetime of this type of concrete can be explained by the close match between the form of the concrete mass and the function it has to fulfil. No tension forces are acting in this type of structure.

1. Aqueduct, Nimes
2. Water reservoir, Rome
3. Pantheon, Rome

The process of cement production was improved when, in 1824, the Englishman John Aspdin was granted a patent on the production of Portland cement. This was followed 20 years later by Isaac Johnson who improved the cement production process still further, to obtain a new and more reliable binder. Again, 20 years later the Frenchman Joseph Monnier discovered that when iron bars were added the concrete the material became stronger; even tension forces could be taken up. Concrete became a reliable engineering material. The strongest argument for its use was the relatively low cost of the

Concrete as a bearing material for more attractive finishes
material in relation to its potentiality. It had no esthetical quality. It was a material to build with, not to look at. In England, as in France, architects tried to find architecturally acceptable ways in which the cheap concrete could be used. Because of the more or less grey colour and the irregularity of the texture, most of the concrete was covered by facing materials. In England the architect John Ruskin analysed the visual aspects of the new material for an ethical point of view. In his opinion, to give this material a texture in such a way that it looked like a different material was fraud. Thinking in this way, he encouraged architect colleagues to use concrete only as a bearing material for more attractive finishes.

In France, at the end of the nineteenth century, an art movement known as ‘art nouveau’ started. This movement was closely related to the art of sculpture. In architecture, the flowing or plastic forms had to be cut out in brick. Concrete, with its potential for plastic forms was very compatible with the ideas of the art nouveau. However, the technical problems of successfully producing the necessary moulds for the constructions were such that very few architects could make use of the plastic possibilities, although the Spanish architect Gaudi did realise some of his designs.

Left: Chnimneys and entrance to the roof (Gaudi)
Right: Mercurius designed by Mart Stam (ca. 1910)

At the start of this century the number of buildings with a concrete bearing structure increased. The initiative and stimulation for the use of concrete for bearing structures came from insurance companies and from the big storeowners. They discovered that a concrete structure not only leads to a building that needs a lower budget, but also that monolithic concrete structures were more resistant to calamities. After the 1926 earthquake in 1906 in San Francisco and the following fire, it was the concrete structures that survived and could be re-used.

One of the first buildings in which concrete was used in an architectural way as well as in a structural way was a garage in 51,rue de Ponthieu in Paris. This was designed and realised by the French architect Auguste Perret in 1905.

The real breakthrough was after 1945, when concrete was extensively used as an alternative building material. Shortages in practically every field, but especially in materials and expert manpower,
led to a dramatic increase in the use of this material. New architectural movements such as ‘brutalism’ meant that exposed concrete could be used to express bold forms in facades. Architects like Le Corbusier, Breuer and, for instance, the Dutch architect Bakema showed that it was possible to express their ideas by using concrete for their facades. Until the seventies concrete was used to such an extent in facades that they gave towns and cities a new face. After the seventies, under the influence of the widespread wish to be more careful with energy, fewer new concrete facades were seen.

In the seventies it also became more clear that concrete is a material that needs maintenance and especially preventative steps to reduce the need for maintenance. The density and the thickness of the covering of the steel by concrete by no means always sufficient. Under the influence of the environment, the steel corroded. Carbonation became a well-known feature, as also were the effects of hidden movements due to shrinkage, temperature or moisture content, which were not always very well understood. Modern concrete showed that it did not have the long-term existence of the Roman concrete.

The next step in the development was an industrial one. The industry could prefabricate concrete elements of a high quality and transport them to the building site, where they could be assembled easily and quickly. Time is money. The quicker the concrete elements could be taken out of the moulds the higher the productivity. The discovery of the use of CaCl to accelerate that hardening of the concrete was a further step forward.

Using CaCl had side effects, which at that time were not understood. Years after prefabricated concrete elements with too high a content of CaCl had been used, a high percentage of damage became visible. The CaCl itself attacked the steel and led to pit corrosion. Another side effect of using big facade elements of concrete, which can be more often seen in the countries of Eastern Europe, is the mechanical damage caused by using the wrong means of transportation.
Cantilever beam with Cl· Removing the concrete (old way of repairing)
Replacing cantilever beam (modern solution)

Since people learn quickly from mistakes, many sources of damage to concrete have been eliminated as a result of:
- the production of a good concrete mass, including many kinds of additives; quality control at the factory is a normal procedure nowadays;
- the production of correct moulds, which belongs to the expertise of the sub-contractor;
- the improved methods of compacting concrete;
- the choice of the correct covering for the steel reinforcement in relation to the environmental conditions;
- the use of a correct after treatment.

Of course, making mistakes is human, but the big problems are now under control and this is leading again to an increasing use of concrete in a visual and architectural way for new buildings.
In contrast to this, we must consider the value of existing buildings - our building heritage. This value can be seen as an urban quality, but also as a money or an energy saver if we can make changes in such away that existing buildings can fulfil new purposes through the renovation of buildings. The changing, adapting and possibly restrengthening of concrete is a contemporary task for architects and structural engineers. They have to work together to see how the modern knowledge of concrete in all its aspects, including high density, self compacting or shrinkage reduced concrete, can be used for the adaptation of existing buildings. Some of these aspects form the theme of this congress. We must think about how existing concrete structures can be restrengthened, which may be necessary if the load or, even more importantly, the original concept of the building changes. This can also be necessary after earthquakes although it would be even better if countries situated in earthquake areas could improve the strength of structures to prevent damage. A task for authorities with a well-developed sense of responsibility.

Before renovation
Renovation architect:
Prof. J. van Stigt

Making openings for daylight

After renovation in use for dwellings and offices
Another big issue is our maintenance task. A lot of money is needed for maintenance. On average, about 1 to 2% of the investment is needed in each year. For a country like the Netherlands, this is in the order of 500 Euro per inhabitant. If this sum can be reduced, the amount saved can be used for more productive purposes. A change in the direction of thinking about concrete is due, from maintenance after damage to maintenance to prevent damage. Development of non-destructive research techniques will be a hot item for the next ten years.

Delft, 1999-10-08

Leo G.W. Verhoef
TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Introduction on COST</th>
<th>F. Charmaison</th>
<th>xv</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete damage - formation,</td>
<td>Prof. W. Brameshuber</td>
<td>21</td>
</tr>
<tr>
<td>detection, repair</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corrosion in concrete</td>
<td>Ir. P.A.M. Zuidgeest</td>
<td>31</td>
</tr>
<tr>
<td>and fire damage</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Changes in concrete structures for</td>
<td>Assoc. Prof. L.G.W. Verhoef</td>
<td>37</td>
</tr>
<tr>
<td>future use</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inspection and monitoring of</td>
<td>Ir. J. Bakker</td>
<td>51</td>
</tr>
<tr>
<td>reinforced concrete structures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Potential mapping - part of a</td>
<td>Ir. R.G.J. Ackerstaff</td>
<td>77</td>
</tr>
<tr>
<td>new concrete repair strategy</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rehabilitation of prefabricated</td>
<td>Ir. P.C. Nuiten</td>
<td>85</td>
</tr>
<tr>
<td>concrete floor-slabs contaminated</td>
<td></td>
<td></td>
</tr>
<tr>
<td>with chlorides, by means of external</td>
<td></td>
<td></td>
</tr>
<tr>
<td>post-tensioning cables</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strengthening of reinforced concrete</td>
<td>Prof. R. Zarnic</td>
<td>91</td>
</tr>
<tr>
<td>by fiber reinforced plastics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Possibilities and problems for post-</td>
<td>Dr. D.A. Hordijk</td>
<td>103</td>
</tr>
<tr>
<td>installed anchors used for changing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>concrete structures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Protection of concrete by water</td>
<td>Prof. F.H. Wittmann</td>
<td>113</td>
</tr>
<tr>
<td>repellent mortars or impregnation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Structural strengthening by grouting</td>
<td>Prof. S. Chandra</td>
<td>123</td>
</tr>
<tr>
<td>Collaboration between existing and</td>
<td>Prof. C. van Weeren</td>
<td>131</td>
</tr>
<tr>
<td>newly added concrete structures</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strengthening concrete structures</td>
<td>Ir. W.B. Grundlehner</td>
<td>139</td>
</tr>
<tr>
<td>with the aid of bonded reinforcement</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Introduction on COST

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Mission

Founded in 1971 COST, is an intergovernmental framework for European Co-operation in the field of Scientific and Technical Research, allowing the co-ordination of nationally funded research on a European level. COST Actions cover basic and pre-competitive research as well as activities of public utility.

The goal of COST is to ensure that Europe holds a strong position in the field of scientific and technical research for peaceful purposes, by increasing European co-operation and interaction in this field.

COST has clearly shown its strength in non-competitive research, in pre-normative co-operation and in solving environmental and cross-border problems and problems of public utility. It has been successfully used to maximise European synergy and added value in research co-operation and it is a useful tool to further European integration, especially concerning Eastern and Central European countries.

Ease of access for institutions from non-member countries also makes COST a very interesting and successful tool for tackling topics of a truly global nature.

To emphasize that the initiative came from the scientists and technical experts themselves and from those with a direct interest in furthering international collaboration, the founding fathers of COST opted for

a flexible and pragmatic approach. COST activities have in the past paved the way for Community activities and its flexibility allows COST Actions to be used as a testing and exploratory field for emerging topics.

The member countries participate on an “à la carte” principle and activities are launched on a “bottom-up” approach. One of its main features is its built-in flexibility. This concept clearly meets a growing demand and in addition, it complements the Community programmes.
COST has a geographical scope beyond the EU and most of the Central and Eastern European countries are members. COST also welcomes the participation of interested institutions from non-COST-member states without any geographical restriction.

COST has developed into one of the largest frameworks for research co-operation in Europe and is a valuable mechanism co-ordinating national research activities in Europe. Today it has almost 200 Actions and involves nearly 30,000 scientists from 32 European member countries and more than 50 participating institutions from 11 non-member countries.

**COST Status 1999**

In total, institutions from 43 countries participate in COST under different forms:

**32 member states:**
Austria, Belgium, Bulgaria, Croatia, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, The Netherlands, Norway, Poland, Portugal, Rumania, Slovakia, Slovenia, Spain, Sweden, Switzerland, Turkey, United Kingdom.

**11 states with participating Institutions:**
COST has a geographical scope beyond the EU. Institutions from non-COST countries may join COST Actions and at present, there are institutions from the observer states and the following states: Australia (4), Canada (5), Egypt (1), India (1), Israel (11), Japan (3), Kazakhstan (2), New Zealand (1), Russia (22), Ukraine (3), USA (7).
COST actions

COST is based on Actions. These are networks of co-ordinated national research projects in fields which are of interest to a minimum number of participants (at least 5) from different member states. The Actions are defined by a Memorandum of Understanding (MoU) signed by the Governments of the COST states wishing to participate in the Action.

The duration of an Action is generally 5 years. Running Actions, their participating countries (signatories) and their duration are shown in the table.

Number of actions:
The success of COST can be best seen by the ever increasing number of COST actions (see below). Together with the new actions under preparation the number of COST actions will reach a total close to 200 during 1999.

Level of participation:
The participation of the various countries in COST actions is shown below. In general the participation over the member countries appears to be evenly distribute with no dominating country. No significant change in this distribution has taken place over the last 5 years.

Scientific domains
COST covers a wide range of scientific and technological domains. The present 17 domains and their part of running actions in 1999 are shown below.

Yearly evolution of the Running and Starting Actions

Funding
COST represents an estimated volume of national funding of more than 1.5 billion Euro per year. This funding is basically used to cover co-ordination expenditures such as contributions to workshops/conferences, travel costs for meetings, contributions to publications and short term scientific missions of researches to visit other laboratories.

An average of 50 000 to 60 000 Euro is available per action depending on size and activity of the action. The EU co-ordination expenditure represents in the average 0.5% of the overall national funding which shows that COST gives excellent value for money.
COST Organisation Structure
Committee of Scientific Officials (CSO)
The CSO is the decision making and highest body in COST. It is composed of COST member states representatives, one of whom in each case acts as COST National Co-ordinator (see below). Technical Committees (TC) are responsible for a particular sector under the authority of the CSO and has mainly to prepare proposals for research projects. A TC is also responsible for technical preparation work and overseeing the proper implementation of the projects as well in an advisory capacity in the co-ordination and evaluation of the ongoing projects. Management Committees (MC) are in charge of the implementation, supervision and co-ordination of a COST Action. A MC is formed by not more than two representatives of each Signatory Country.
COST National Coordinator (CNC)
One member of the CSO from each member state acts as national coordinator (CNC) for COST projects. The CNC provides the liaison between the scientists and institutions in his country and the Council COST secretariat.
The CNC has specifically:
- Ensure that the national funding is available;
- Appoint or officially forward the names of the national delegates to the technical committees (TC) and management committees (MC);
- Forward proposals from other countries to experts in his own country;
- Assess at national level all projects undertaken within the COST framework.

Secretariat
The secretariat is provided by the Secretariat General to the Council of the European Union (secretariat of the CSO) and of the European Commission (scientific and administration matters).

COST Urban Civil Engineering
- Technical Committee on Urban Civil Engineering
- Finished Actions as of August 2000
  C1: “Control of the semi-rigid behaviour of civil engineering structural connections”
  C2: “Large scale infrastructures and quality of urban shape”
  C3: “Diagnosis of Urban Infrastructure”
  C4: “Management and information application development in Urban Civil Engineering”
- Ongoing COST UCE Actions
  C5: “Urban heritage - Building maintenance”
  C6: “Town and infrastructure planning for safety and urban quality for pedestrians”
  C7: “Soil-Structure interaction in Urban Civil Engineering”
  C8: “Best practice in sustainable urban infrastructure”
  C9: “Processes to reach urban quality”
  C10: “Outskirts of European Cities”
  C11: “Greenstructures and urban planning”
  C12: “Improving buildings structural quality by new technologies”
  C13: “Glass and interactive building envelopes”
  C14: “Impact of wind and storm on city life and built environment”
  G3: “Industrial Ventilation”
CONCRETE DAMAGE - FORMATION, DETECTION, REPAIR

Prof. Dr.-Ing. W. Brameshuber
Institut für Bauforschung Aachen (ibac), Germany

Introduction

Three examples of formation and repair of concrete damage will be shown in this contribution. Especially the detection of damage zones and progress of damage show the advantages of using indirect test methods. The examples chosen are:

- Corrosion risk detection using potential measurements
- Defects in large concrete piles, application of ultrasonic measurements
- Impact echo measurements for controlling concrete tunnel shells

The first example comes from research works of the Institute for Building Research (ibac), Aachen, as well as some consultants. The two other examples have been developed to application during the author’s work in the construction firm Bilfinger+Berger, Mannheim. The work in research and application of impact-echo-technique will be continued at ibac.

Potential measurements for detection of corrosion risk

Reinforcement usually is protected against corrosion due to high alcalinity of concrete. Carbonation of concrete as well as chlorides may induce corrosion. The method of measuring the potential of the reinforcement seems to be a suitable way for checking the corrosion risk of the reinforcement in the concrete. The principle of this method is shown in figure 1. An electrode is applied to the concrete surface and connected with the reinforcement. With a special voltmeter the voltage as potential differ-

![Diagram of potential measurement](image)

Figure 1: Principle of measurement of reinforcement potential
ence between electrode and steel is measured.

According to RILEM TC 154 values of potential can be given depending on the state of the reinforcement as well as the concrete. E.g. values lower than -400 mV indicate high moisture and chloride content, but may not be critical. Therefore one needs the absolute values as well as the gradient of the potential to assure serious results. In the following an example for the potential measurement is given.

Figure 2 shows the bottom of a pillar of a parking house. Especially at the bottom chlorides will penetrate into the concrete. The zone of high corrosion risk is given in the cross-section of figure 2.

Figure 2: Corrosion in the bottom of pillars due to chlorides

In figure 3 the result of a measurement of the potential at the bottom of a pillar is given. Five measuring lines have been carried out around the pillar. In figure 3 the surface of the pillar is represented in form of a winding-up. As can be seen, high corrosion risk is observed at the lower part of the pillar. The concrete cover is very low in the region of low potential values, the north and west side of the pillar mainly are in contact with water containing chlorides as well as the east side with high values for the concrete cover. This corresponds quite good with the corrosion risk in such a pillar.

Figure 3: Result of measuring the potential of reinforcement at the bottom of a pillar
Such cases usually are repaired by removing the contaminated concrete and restore the cross-section using special mortars. To assure, that corrosion will not go further on, chloride penetration may be observed using special sensors as described e.g. in /RAU93/.

**Defects in large concrete piles**

Large concrete piles are important elements in modern constructions. The load carrying capacity of such piles often must be up to 8 MN into very deep layers of good bearing underground. To avoid high values of settlements high requirements have to be fulfilled on the quality of piles. Buildings like the „Pont de Normandie“ nearby Le Havre, „Commerzbank II“ in Frankfurt or the „My Thuan-Brigde“ in Vietnam have foundations consisting of very large concrete piles with a length up to 100 m and a diameter up to 2.5 m. The piles of these buildings were examined by ultrasonic technology to check the quality of concrete.

The principle of the testing procedure is shown in figure 4 /GRO95/. An ultrasonic receiver and transmitter move in steelpipes, which are filled with water. The pipes were mounted to the reinforcement before concreting the pile. The position of the transducers is recorded by an electronic wheel. The ultrasonic signal is generated and recorded by a driver box and a PC. The time of the ultrasonic signal is calculated. Because the distance of the pipes is known, the sonic speed can be calculated, for hardened concrete about 3.500 to 4.000 m/sec. Four up to six steelpipes are used to ensure a complete cover of the cross-section. E.g. using four pipes, six different paths for measurement may be possible in one cross-section. The measurements begin at the bottom of the pile. Every about 6 cm an ultrasonic signal is transmitted. After finishing the procedure is started again with another path, i.e. using two other pipes.

The right part of figure 4 shows schematically the arrival time of the signal with depth. In figure 5 the calculated sonic speed with depth as measured at a real pile is shown. If the signal is transmitted by hardened concrete of good quality, the arrival time is constant with depth as well as the sonic speed, respectively. If there exist substantial anomalies in the concrete, the arrival time will increase, resulting in a decrease of sound velocity. In figure 5 such an anomaly due to problems while concreting has been detected in a depth of about 17 m. Due to missing knowledge on the influence of the type of anomaly on the shape of the signal, it was decided to drill a core down to the defect. The core from this region is shown in figure 6. It can be clearly seen, that in the depth of about 17 m the concrete was substantially separated into mortar and aggregate, which was found below the mortar.

One possibility for repairing has been carried out at the „Pont de Normandie“. An excavation around the pile with a floor lower than the defect has been build up, the material with lower strength removed, a mould mounted to the pile and the hole filled with a special concrete. After hardening, the region repaired has been checked again with ultrasonic crosshole testing. Another possibility to repair a pile with defect is the pile-in-pile-method.

A hole with a diameter of about 80 cm is drilled into the original pile. The hole must end in the concrete with good quality below the defect with a certain overlapping. The length of overlapping and the diameter depend on the design, respectively. A basket of reinforcement is placed into the hole. Finally the hole is filled with a special concrete with high strength and low shrinkage. The last described procedure is quite more cheap and has the advantage, that the environment nearby the pile is not disturbed.
Figure 4. Scheme of ultrasonic crosshole testing

Figure 5. Result of a measurement, sonic speed versus depth

Figure 6. Section of a drilled core (corresponding to figure 5)
Impact echo technique for controlling tunnel shells

Mined tunnels usually consist of a layer of shotcrete, plastic-sealings against water pressure and a concrete shell. Closed steel moulds are used for concreting the shell. During concreting the flowing of the concrete in the shell is nearly not visible. Especially in the ridge no controlling of filling the mould can be carried out. Using impact echo methods it is possible to measure the thickness of the shell in a non-destructive way and check, whether gravel nests or cavities exist. A lot of tunnels have been build worldwide. Some of them have defects in the sealing and water penetrates inside. Often it is documented, that the plastic sealing during building up the tunnel had no defects. Some days or weeks after stopping the water-retention water suddenly flows into the tunnel. Due to experience of the author /BRA97-1, BRA97-2, BRA98/ this leakage is often caused by the fact that concrete is missing in the ridge of the shell and reinforcement is not embedded by concrete. Due to the water pressure the sealing is pushed into the reinforcement and locally damaged. To find these locations, the shell thickness should be determined, and no other technique than impact echo measurement seems to be more suitable, as shown below.

The impact echo technique is schematically described in figure 7, details see /MAL91, KRI95/. A stress pulse is introduced into the tunnel shell by mechanical impact on the surface. This stress pulse propagates into the concrete along waves. The waves will be reflected by changing the medium, for example concrete-air, concrete-sealing, respectively. The echo will have a spectrum of frequencies. For concrete, low frequencies, such as those caused by an impact, are sufficient. The frequency of the echo is analysed by using the Fast Fourier transform technique. From the main frequency, the thickness of the shell can be calculated by:

$$d = \frac{v_s}{2f}$$

(1)

where

- $d$ = thickness
- $v_s$ = sound velocity of concrete
- $f$ = main frequency

In figure 8 a typical signal of an impact is shown. Analysing this signal results in a plot as shown in
figure 9. The first peak at about 4 kHz results from the frequency of the measuring system, the second peak at about 10 kHz gives a thickness of the shell of about 0.2 m.

Before measuring a grid of dots has been painted on the surface of the concrete shell. Spacing of the dots in both directions has been chosen to 0.4 m. From the experience it is known that problems mainly occur nearby the ridge. Therefore, the length of the concrete surface perpendicular to the tunnel has been chosen between 6 to 8 m depending on the cross-section of the tunnel. Due to the normal length of a tunnel block of about 10 m, 400 to 500 measurement points have been checked for each block. After drawing the grid on the concrete surface the impact was released at each point.

Figure 7. Scheme of impact echo technique

Figure 8. Signal of an impact caused by a hammer

Figure 9. Result of a frequency analysis
Figure 10. Impact echo measurement nearby a leakage

Figure 10 is typical for the situation while measuring. Two or three blocks were connected together and one file was created for one row in the direction of the tunnel. The data have been stored on the computer. Using this system of coordinates it is very simple to find a single point later on in the tunnel. Having found a region of less thickness a finer grid is used to get more information about the shape of Concrete
this region. The spacing of the dots then is chosen to 0.1 m.

Figure 11 shows the thickness of the concrete shell for two tunnel blocks. In block 38 nearby the joint at the ridge the concrete thickness is less than 0.25 m instead of the desired thickness of 0.4 m. A second measurement using the finer grid has been carried out. The result is shown in figure 12(a). The measured thickness has been confirmed by drilling some cores. The cavity in the concrete caused the damage of the sealing because it was pushed into the reinforcement due to the water pressure. Knowing the shape of the cavity one can carry out the next steps for repair, e.g. injection, very precise. In the first step cores were drilled for injection of cement paste. The result of this injection as determined by an additional impact echo measurement is shown in figure 12(b). The injection was not complete, but about 90% were filled with cement paste, resulting in a highly reduction of the water flow. A second injection has been carried out using a paste made of cement with higher fineness. The result is shown in figure 12(c). After this injection the tunnel was dry.

Figure 11. Results of a thickness measurement, grid with 0.4 m spacing

Figure 12. Detail of figure 11 nearby the joint, grid with 0.1 m spacing
Conclusions

In this contribution three examples for non-destructive testing methods have been described. From this, it can be seen, that NDT has developed in a quite good manner, defects/damage caused by different incidents can be detected and the quality of repair controlled, respectively. Nevertheless, application of NDT-methods needs a lot of experience. The interpretation of measured values often is not totally sure due to the inhomogeneity of concrete structures. Sometimes defects may be detected by signal analysis which are not real. Therefore only an engineer with high experience can decide about the validity of the measured values.

Literature


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Introduction

This paper presents an overview of the problems that can be identified in buildings with concrete structures. The selection is made from the practical point of view of maintenance. Since concrete is a very durable material, especially when not exposed to outside conditions, the only serious problems that have to be faced inside the buildings result from fire damage. All the remaining problems happen outside.

Fire damage

Two aspects of fire damage must be considered in relation to maintenance:
- determination of the fire-resistance
- refurbishment and repair of the structure after a fire

For both aspects, it is important to understand what happens to a concrete structure during a fire. The fire-exposure is calculated as a typical temperature development in a room (Fig 1.). Different schemes are applied to different situations. The structure is assumed to carry a typical load. As a function of the heat capacity and heat-conductivity of concrete, the reinforced structure also follows a delayed temperature development. The bonded water of the concrete causes the temperature to remain around 105 °C for some time. After all the water has evaporated the temperature in the concrete cover rises again. The temperature of the steel reinforcement follows the temperature of the concrete cover. Fire-resistance calculations are based on the temperature of the reinforcement. At a higher temperature, the steel will flow at a lower stress level (Fig 2.). Floors and beams are expected to collapse when the steel reaches a temperature at which it flows under stress that is related to the typical load. The fire-resistance is defined as the time between the start of a fire and this point. In addition, there are criteria of maximum bending and maximum temperature at the unexposed surface. The most generally required fire-resistance in buildings is 60 min or 90 min. This is a reasonable time to evacuate before the threat of collapse is to be expected. For different structures like highway tunnels, a fire-resistance of up to 120 minutes is required. This time is valid even with much more serious fire-exposure.

Because of its high specific weight, concrete has a relatively high heat capacity. Therefore, it is used as fireproofing material in offshore structures. This is also the reason why normally no special fireproofing is required in buildings with concrete structures.

Only 10 to 15% of fires develop so severely that structures may exhibit signs of collapse, like large scale bending and/or cracks. Of course, the repair of such damage is unlikely to be feasible. Most concrete structures do not show these severe signs after a fire, but more often there is scaling at the surface. This damage can be caused when fast temperature development has occurred. The bonded water can 'explode' which causes internal cracking. This effect is also known from the refurbishing technique of 'fire blasting'. Another cause can be the use of large quantities of water during the fire fighting process, which gives a temperature-shock at the surface.

To determine if this internal cracking has occurred, one can knock the surface with a hammer. In practice, a rebound-hammer ('Schmidt-hammer') has also proven to be a good tool. It gives a nice resonance in the structure.
With a hand on the surface, one can clearly tell the difference between a part with the cracking and a sound part.
The depth of damage can easily be determined by hammering out a piece of concrete or by taking core samples at several points.

When a structure has not collapsed, the standard reinforcement steel has hardly suffered from the fire. However, cold-stretched pre-stressed steel should be checked since when high temperatures have occurred, it can lose its high strength and stress permanently. This problem must also be addressed in prefabricated floor elements that are cast at the so-called 'long bank'. Short samples can be tested in a laboratory for their stretching behaviour.
After fire, concrete structures can be repaired well by using sprayed concrete. First all the de-laminated concrete must be removed. By using the pull-off test before applying the sprayed concrete, it is possible to determine whether the substrate has been adequately prepared.

**Corrosion in concrete in retrospect**
Since 1980 many of the problems caused by damage to concrete have been studied and explained. At the turn of the millennium, there is a lot of experience of inspection, diagnosis, selection and the execution of repair measures. We have come to the point where we know enough about the processes and materials to be able to follow a true long-term maintenance regime instead of a 'one-off solutions for unique problems'.

Further research is needed to optimise materials and techniques both for this long-term maintenance approach and for environmental and health reasons. Of course there is always a need for true fundamental research to come up with new approaches in testing and repair.

**Carbonation**
Except in marine environments, most concrete damage is caused by carbonation. Carbon dioxide penetrates the concrete and reacts with calcium hydroxide, which causes the pH of the concrete cover to decrease from 12 to below 9. When the carbonation has reached the steel bars the concrete loses its protective function and steel can start corroding. Since the corrosion-product takes up to six times the volume of the original steel, the concrete cover is cracked or de-laminated.

For some time, it was expected that all not protected concrete would eventually suffer from this process. Now we know that in moderate climates the progression of the 'carbonation-front' is asymptotic. It keeps pace with the depth of drying out. Inside a building, the 'moisture-front' and carbonation process passes relatively quickly through the cover. However, because the carbonation process needs water, not much of the calcium hydroxide is consumed during this passage. Only when concrete is very porous inside can carbonation penetrate right through in time. But then again, there is not enough moisture in the concrete to prolong the corrosion-process.

Outside damage will only occur when the carbonation has reached the steel. Even then, it has been found that prolongation of the corrosion depends on the fluctuations in the moisture content of the concrete around the steel (figure 3). If the cover is homogeneous and thicker than 20 - 25 mm, even in a carbonated zone the steel corrodes only very slowly and not much damage is to be expected.

![Figure 3: Different zones in the concrete cover with carbonation and moist fluctuations](image-url)
Repair and the prevention of carbonation-initiated corrosion damage can be prescribed after a good survey of cover thickness, carbonation depth and the amount of de-laminated concrete. All de-laminated concrete must be removed and the corrosion must be cleaned from the steel. The spots of damage can be repaired with cement mortars, applied by hand or by spraying.

To prevent further damage, the aim should be to ensure that the steel is at least in a zone where no moisture fluctuation occurs. This can be obtained by hydrophobic impregnation, a permeable coating or an extra concrete layer. These measures keep the fluctuations more to the outside of the cover. The choice mainly depends on other criteria like aesthetics and accessibility. A good colour coating improves the commercial value of a building. However, when scaffolding is very expensive, the costs for repeating hydrophobic impregnation (5 to 10 years) or the coating (10 to 15 years) may make the choice of a more expensive extra cover with sprayed concrete the most feasible. Further maintenance should concentrate on maintaining the function of the protective measures.

One more observation is that when a building of, for instance 25 years old, is treated for the first time, the weakest spots have become obvious by natural selection. In most of the rest of the concrete, the steel is probably in a zone that is not carbonated and/or has no moisture fluctuations. The carbonation-progress has most likely reached the asymptote and will never get deeper.

**Chloride**

Chlorides usually play a role in concrete damage in buildings in marine environments. In non-marine environments chloride-related damage can be found where salt is used for de-icing walkways. A classical case happened in Wormerveer, where a cantilever walkway collapsed due to corrosion of the steel under the gutter (figure 4).

![Figure 4: Collapse of a walkway due to chloride-initiated corrosion.](image)
Furthermore, calcium chloride can be used as an accelerator for the curing of the concrete. In the seventies, this happened in Holland on quite a big scale. Most often, it was used in prefabricated elements to boost production. At present ground floor elements are a hot topic that is also treated separately in this congress.

The assessment of chloride-initiated corrosion is less straightforward than carbonation-initiated corrosion. The same goes for the treatment. With moderate chloride contents - up to 1% m/m of cement weight - parallels in corrosion behaviour have been found with carbonation-initiated corrosion. With these chloride concentrations, it is found that when the cover is of good quality and at least 20 to 25 mm thick, the corrosion is very slow and real damage can be delayed for a long time. When there is a combination of carbonation and chloride around the reinforcement, corrosion can be aggressive.

One problem is that most of the time the concentrations of chloride in various parts of the structure vary greatly and so does the moisture content. Because the corrosion-products from chloride-initiated corrosion are solvable in water it may be a long time before damage occurs in the concrete. When it does occur is may be quite serious: deep pits may be formed at local spots. Assessment of a structure when there is the suspicion of a chloride problem must be done more intensively and carefully. Drilling-powder samples can be taken for analysis of the chloride content and half-cell measurements carried out to locate possible corrosion spots. Still the human eye is the most effective of selecting the places where there is the biggest threat.

Places with a real loss of steel diameter should be repaired. The concrete cover must be removed locally to free the corroded steel bar. When traditional repair is chosen, the steel must be cleaned thoroughly and immediately afterwards coated with a polymer coating. This prevents a different electrical polarity of the steel in the fresh concrete from increasing corrosive action in neighbouring steel that is still in chloride-contaminated concrete.

This traditional repair of chloride-initiated corrosion can still be feasible, if it is proven that the chloride concentrations are moderate and only local circumstances are really aggressive. Of course, additional measures should be taken to prevent more chloride penetration, but a moisture control must also be carried out.

Although the ‘natural selection’ of worst spots is apparent in cases of chloride contamination, one cannot trust that once the worst spots are treated, no further damage is to be expected, as is more often the case with carbonation.

Therefore, every few years a strict program of inspection should be executed. In addition, electrochemical monitoring with half-cells is an interesting option, though it is difficult to select the next spot where corrosion will appear.

When there are higher chloride concentrations, traditional repair is not enough. Then, in many cases, cathodic protection is the most feasible option to preserve the structure in the long term. If this is option is chosen, only the places with real de-laminations of concrete should be repaired. For the repair, no polymer coating must be applied on the rebar. Once a cathodic protection system is installed a follow-up by monitoring and adjusting is necessary. In particular anode materials and measuring equipment like half-cells will need maintenance and replacement in time. Therefore, when higher chloride concentrations are detected, no “one off” measurement is sufficient.
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Problems and possibilities is the subtitle of this congress

What the problems and possibilities are has not yet become clear but we do know that they relate to concrete structures. Concrete is used in a very wide area from roads, viaducts and bridges to buildings and for a variety of miscellaneous purposes that together fall within the province of urban heritage/building maintenance. In this contribution, we restrict ourselves to buildings and within that broad classification, to residential buildings, although the ideas that have been developed can also be applied in other areas. Because the subject is now concrete residential buildings, it involves the adaptation of concrete structures to make them suitable for dwellings. The original concrete structure may also have had a residential function but not necessarily so. There are many examples in which warehouses, office buildings, churches and even silos have been transformed into residential buildings.

The present number of residential buildings in the Netherlands is around 6.5 million. These include about 1.5 million ‘traditional buildings’ constructed before 1945. In the context of housing, after 1945, the huge demand for dwellings could no longer be satisfied by building in the traditional manner. Various industrialised housing construction systems came onto the market so that the smallest possible number of men could build the greatest possible number of housing units/dwellings in a very short time. Up to 1970, traditional houses were still built being built but industrialised building made by far the greatest contribution to the addition of almost 2 million homes to the housing stock. After 1970 with the pronouncements of ‘the club of Rome’ and the energy crisis that followed, buildings were much better insulated than they had been previously. Different types of dwelling, building materials and structural details were needed. In the period after 1970 a further 3 million dwellings were constructed.

In addition to the developments in residential building, there have been many demographic changes; such as fewer children per family, fewer one and two-parent families, increased life span and greater financial capacity. These changes are expressed in other requirements, with the general desire for different types of housing, larger rooms, greater luxury and increased comfort. Therefore it is necessary to adapt houses to meet these demands and up to the present, the shell of the building has been seen as the boundary within which alterations can be made. To some extent, this is understandable because the original new houses were suited to their purpose and they were kept in good repair by maintenance. Because over the years, there have been great changes in the demands and regulations have been added relating to energy, noise, social safety and sustainable building, maintenance measures are no longer adequate and renovation must be carried out (figure 1).
From the maintenance period, we have learnt that it is the shell itself that affects the cost of maintaining the building. For this reason, it is no longer logical to see the shell as the boundary within which changes can be made. Indeed, from now on it must be a condition that the integral costs and yields can be related to the changed model, which permits alterations to be made within the shell and extensions beyond the present limits of the shell.

From the 1970s, the renovation of dwellings and more especially entire residential neighbourhood came into prominence. A characteristic renovation technique was to combine dwelling units and in this way to increase their size. Within the shell of the building, three adjacent units could be converted into two dwellings. The costs of this exercise were limited but there were only two tenants to pay them in place of three and the total cost of the renovations must be divided over two dwellings. By adding the desired extra living space outside original building volume it is possible to keep every dwelling unit intact, the total cost is spread over three units and there are three tenants. The calculations for a project show that in comparison to the combination of dwelling units, the extension of the shell by placing the necessary new facade brings lower costs and higher incomes per unit.

Because we restrict ourselves to industrially constructed buildings with a concrete load bearing structure, it is wise to look at the idea behind the design as background to the industrial building method. For one thing, the architectural movement 'the new building' chose to separate the residential and working environments. A positive effect of this was the garden city with many communal public spaces and no necessity to live in an area with relatively high industrial pollution. One consequence partly stimulated by the fact that the government permitted specific systems to be used to construct large contingents of dwellings, was the attendant urban sprawl. Monoculture!

Now there is a new movement towards the combination of residential and working environments and by replacing residential blocks by towers and low-rise buildings to create more variation in an area. The industrialised building system itself is being developed on the basis of costs and not of use or future value. Because of this, components that could form flexible interior structure become parts of the bearing construction. The industrialised concrete building systems developed in this way have no flexibility so that when the question of adaptation to new functions, with the modern requirements that this brings, arises, they are more likely to be destined for demolition.

When it comes to concrete, in the past the choice of concrete in the past was based on strong and weak points of the material in comparison to brick, steel or wood.

Strong point in favour of the choice of concrete were:
- the durability;
- hygienic considerations,
- fire resistance,
- design freedom,
- the relatively high compressive strength,
- the mass (acoustics).

After thirty years, we can now question the strong points. Our doubts relate to:
- The durability in relation to the corrosion problems that have been established, whatever their cause may be;
- The relatively high compressive strength, which is low compared to that which can now be achieved;
- the mass, that is indeed big but which needs a thickness of wall or floor that is 210 mm thick to satisfy the acoustic demands. In the 1970s the stipulations in relation to strength and stiffness for buildings were guidelines for the design, as a result of which in many cases floors and walls could be thinner and the favourable acoustic effect no longer attained.
Weak points that worked against the choice of concrete were:
- the weight (the mass);
- the contraction;
- the creep;
- the cost of future demolition.

It is interesting that now some of the unfavourable aspects such as the cost of future demolition fit in with current desires for durability and also increase the feasibility of renovation projects. After all, if existing buildings are replaced by new buildings, the high cost of the demolition of the existing buildings must be taken into account. The mass of the dead load of concrete means that higher changing loads have much less effect on a concrete structure than on lighter bearing structures of steel and wood.

In general, there is much to be said for the principle of keeping existing concrete buildings and changing them to satisfy new functional demands. Moreover, new insight into the constructional behaviour and safely, indicates that existing concrete structures are often more reliable than was originally calculated. This will be discussed later when the making of great savings is under consideration.

**Blocks of gallery flats constructed in in situ concrete**

Blocks of gallery flats constructed by using the in situ concrete system, which have been built in large numbers in The Netherlands and throughout Europe, have been chosen to provide an example of making changes within the shell of the building and by changing the shell of the building.

The area between the buildings is usually large because the residential development was designed on the basis of the garden city idea that was associated with ‘the new building’.

In the flats chosen as demonstration model, the foundations are on piles over which foundation beams are laid and a in situ concrete floor on the ground. The skeleton consists of in situ concrete floors and walls. The rooms on the ground floor are the same height as those on the upper storeys, which makes it easier to change their functions. A double wall is used for dilation joints so that each part of the building is independent. The floors above the ground floor are constructed at the same time, as the walls by using narrow tunnel shuttering the widest floor spans is 4 metres. The thickness of the floors is 120 mm, while the vertical slabs are 180 mm thick. The cantilever beams are made of prefabricated concrete. There are two types, one columnar with a cantilevered,
projecting corbel and one that is constructed as a simple corbel. Both elements are either mounted in or on the tunnel formwork or attached to the bearing construction by reinforcements. The stability of the building in both transverse and longitudinal directions is provided by the concrete walls.

Figure 3:

Very generally stated, the problems of the existing blocks of gallery flats arise from:
- the massiveness and uniformity of the blocks of gallery flats, which lack human scale;
- the lack of social control so that in and around the flats there is more criminality than near other types of dwelling;
- an extreme form of uniformity so that even from a short distance ones own dwelling cannot be identified;
- in consequence of this, the residents are not proud of their home and vandalism is accepted;
- passages through the flats are dark and narrow, which create an atmosphere of danger;
- no clear area of transition between private and public area so that the private area starts behind the front door and little attention is paid to the public area;
- the plans of the dwellings, with bearing partition walls that do not permit the flexibility that is needed for changes to meet the new wishes.

Figure 4

No human scale  Human scale  Human scale
In the chosen example, an attempt is made to achieve presence human scale by leaving out the gallery on the first floor. By making this constriction, a different zone is provided on two storeys that can serve to create human scale. The disadvantage of this solution is that the lack of supervision in this area provides opportunities for undesirable activities to take place precisely the zone where there is no possibility for social control from the galleries. To solve this problem it is necessary to make changes. Social safety, especially at the ground floor level, can be improved by including dwellings at this level. The residents then acquire the function of become the 'social eyes', which leads to a reduction in criminality. There is people's behaviour is under social observation.

Figure 5:
Top: Original design of the “hoogoord flat”
Bottom: New design for the “hoogoord flat” by F. Verheyen

Having dwellings on the ground floor, while at the same time maintaining the storage space of the residents in the flats on the upper floors requires extra space. The most obvious solution is to let the two lowest storeys project beyond the walkways.

From the technical point of view, there are two options for this:
- the independent construction of the new extensions on their own foundations. This solution is simple but attention must be paid to possible pipelines lying in front of the flat. The extensions add so little load on the subsoil that a shallow foundation suffices for them. If the subsoil has little bearing capacity, as is usually the case in the west of the Netherlands, the extension must be constructed on piles.
- the changes in the concrete structure involve the removal of the galleries and balconies on the second storey and replacing them by a concrete floor with insulation and a finishing layer.

Concrete

41
- The suspension of the extension from the bearing walls. In this case, no extra piles are needed. The reinforcements of the consoles that support the galleries can be retained and taken up in the new extension of the walls.

By suspending walls on the existing construction, there is an important increase in the range of options. The forces that must be taken into account are shear forces along the old concrete, which create relatively small stressed, and the tensile forces and compression forces to transfer moments from the newly added part to the existing concrete. The tensile force that needs to be taken up is equally small, despite the relatively large mass of the extensions.

The present constriction is the same width as the galleries, which are usually very narrow. With a few exceptions, they vary between 1.35m and approx. 1.5 m. wide. If a relatively wide extension is desired, for example up to 3 metres we must investigate whether the assumption that the forces and stresses remain relatively small, is true.

Figure 6:

The suspension option for the extension of the ground floor causes the greatest loading that we can imagine. If the extension continues to a greater height on the facade surface, the loading and the moment per unit area will decrease.

<table>
<thead>
<tr>
<th></th>
<th>Dead weight</th>
<th>Shear force</th>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall</td>
<td>5.3m * 3m * 0.18m * 24 kN/m² = 68.7 kN</td>
<td>68.7 kN * 1.2 = 86.4 kN</td>
<td>86.4 kN * 1.5m = 130 kNm</td>
</tr>
<tr>
<td>Roof plate</td>
<td>4m * 3m * 0.2m * 24 kN/m² = 57.6 kN</td>
<td>57.6 kN * 1.2 = 69.1 kN</td>
<td>69.1 kN * 1.5m = 62 kNm</td>
</tr>
<tr>
<td>Ground floor</td>
<td>4m * 3m * 0.12m * 24 kN/m² = 34.6 kN</td>
<td>34.6 kN * 1.2 = 41.5 kN</td>
<td>41.5 kN * 1.5m = 62 kNm</td>
</tr>
<tr>
<td>Facade panels</td>
<td>5.3m * 4m * 1 kN/m² = 21.2 kN</td>
<td>21.2 kN * 1.2 = 25.4 kN</td>
<td>25.4 kN * 3m = 76 kNm</td>
</tr>
<tr>
<td>Face beam</td>
<td>4m * 0.5m * 0.12m * 24 kN/m² = 5.7 kN</td>
<td>5.7 kN * 1.2 = 7 kN</td>
<td>7 kN * 3m = 62 kNm</td>
</tr>
<tr>
<td>changeable weight</td>
<td>2.84m * 3m * 2.5 kN/m²</td>
<td>60 kN * 1.5 = 90 kN</td>
<td>90 kN * 1.5 = 135 kNm</td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>319 kN</td>
<td>528 kNm</td>
</tr>
</tbody>
</table>

shearing strain = 319 kN / 180 mm * 5000 mm = 0.3 N/mm²

Tabel 1: statical calculations
The shearing strain remains so low that only one treatment such as the rough making and addition a few short anchors is needed to maintain the joint between the old concrete construction and the new in situ concrete part.

The tensile force to transfer the moment is approx.: \( 528 \text{ kNm} / (-0.8 \times 5.3m) = 125 \text{ kN} \)

By using FeB 500 steel reinforcement with \( f_s = 435 \text{ N/mm}^2 \)

\[ 125 \text{ kN} / 435 \text{ N/mm}^2 = 290 \text{ N/mm}^2 \]

These tensile forces can be transferred to the existing bearing construction in two ways. It can be concentrated, with only joints such as long or short adhesion anchors in the walls or spread, in which case part of the tensile force is transferred via the floors.

Naturally, the additional load on the foundation system must also be taken into account.

In view of the stiffness of the walls the so called calculable value of the additional loading, if it is suspended symmetrically on the building must be equally distributed over all the piles.

This means twice 319 kN divided by \( n \). If the additional load is suspended on the south side of the building. For the outermost piles this is approx. \( 1.75 \times 319 \text{ kN}/n \) all in all, a load that can be supported up by this type of building.

In addition to factors such as transfer of forces there is also deformation due to contraction and creep. Contraction and creep can be dealt with two ways. The first is to pour the new concrete directly against the existing structure as a result of which this is subjected to increased pressure and free deformation of the new concrete prevented, so that tensile stresses are introduced. The use of finely spaced reinforcement netting ensures that cracks remain small. The other method is to design in such a way that the newly added concrete construction can deform freely. Ways of doing this include using a saddle on the underside with reinforcement on the upper side, which thus has the possibility to deform over a small area without this creating extra stress. This gives an impulse to the option of using prefabricated extensions.

**Wider corridors through the blocks of flats, so that the present narrow, dark, inconveniently arranged and therefore dangerous routes can be replaced.**

This means where the corridors are located the vertical slab construction of the two lowest layers must be replaced by a columnar structure. Since there are piles under the vertical slabs, the columns are positioned in such a way that they are located between two piles or precisely over a pile. The system is very simple. Where the columns need to be placed, part of the slab is sawn out creating the conditions to fit the new prefabricated or in-situ manufactured columns. The columns need not contribute to stability because sufficient other elements remain to provide this. For example, there are too many transverse concrete walls of the block of flats, while in the longitudinal direction sufficient monolithic construction remains. The junction with the foundation beam, however, may cause a shear force problem. There is usually without adaptation no solution for this problem.

![Figure 7:](image)

*Concrete*
The easiest option is to shape of the columns at the junction with the foundation in such a way that the load can be transferred directly to the piles. Even then, it is still possible that the tensile reinforcement under the foundation beam is insufficient. It is possible to reach the foundation beam by making holes in the ground floor. By using an adhesive-bonding agent to attach reinforcement, the required tensile reinforcement can be reached.

Figure 8:

The junction with the walls of the upper storey may develop excessive stresses. Tensile forces that may develop can most probably be taken up by the distribution reinforcement in the floors. If there is insufficient existing reinforcement for this, reinforcement can be attached by using an adhesive bonding agent.

For placing dwellings on the ground floor, the suspension of walls on the existing structure provides a valuable addition to the range of possible options. Nevertheless, other possible means of doing this such as the addition of piles and the independent construction of the extension. The financial aspect determines whether the addition of piles or the drilling in of anchors is more economical.

Figure 9: Internal galery and change of facade

If on the other hand, we want to make extensions on the higher floors, we shall quickly come to the choice of suspension as the most direct manner of transmitting forces. In principle, the walkways are a type of suspended structure. The only change is the increase in the shear force and the moment.

**Change of galeries**

Galleries are often draughty and poor options for private use since they mainly serve as walkways for people who want to reach their homes. For greater privacy, to prevent people from walking very close to the windows of the flats there are three possible variations for the widening of walkways. The first is to make a big inside corridor, which always provides access for three...
flats. An example of this has been executed as ‘new building’ in the residential building “DWL of the architect Pi de Bruyn, in Rotterdam. Every third floor is extended out to the facade. The part above this is a space extending over two storeys that can be considered as a semi-private area. Because the kitchens look out over this private area, permanent social control is possible so the flats can be considered safe. From the point of view of construction, it is very easy to find solutions and is fully in accordance with the approach to extension at ground floor level.

A second variant assumes that only some of the flats are extended, together with the addition of a lift. In this case, vertical divisions are created to counterbalance the strongly horizontal character. In consequence, the appearance of the block of flats is greatly changed. Part of the gallery disappears, so the access system is one with porches. It is now possible to recognise one’s own home from street level. The extra lifts makes this option very expensive.

If a different method is chosen to construct the extension, the extra lifts are not required. The second escape route must then be via the balconies on the rear side of the building the existing staircase must be adapted in such a way that the stairs run from floor to floor.
The third variant widens the galleries on every second floor. The dwellings become maisonettes. The bedrooms continue under the extension and are thus less exposed to direct sunlight, which in any case is not desired there. By building an extension on the access part, a semi-private area can be created. Because the in situ concrete part of the wall is much higher than the original consoles, the upper reinforcement of the console can be used for the new wall. Extra anchors and the roughening of the walls are necessary to ensure good joints. Here too consideration can be given to letting the tensile force be transmitted not only through the walls but also via the floors. The monotony of the block of flats is now clearly broken up and these measures have given it a stronger character. Maisonettes can contribute to the desired variation in the types of dwelling available and naturally, that such a solution need not necessarily be used for the entire building.
It is necessary to make holes in the floors, raising the question of whether the remaining part of the floor can function with the reinforcement that is in it. If that is not the case, the addition of reinforcement by using adhesive bonding is an obvious option.

**Making two rooms into one**

A great problem in the housing stock constructed in the immediate post-war period and up to the 1980s results from the use the relatively small free spans. The was caused by false feeling for economy which the future use of the buildings was not taken into consideration and which makes changing this type of bearing construction very difficult. The narrow deep rooms are no longer acceptable.

![Diagram of room layout](image1.png)

**Figure 13:**

In the walls there are small 2.1 m high door openings. The height from floor to ceiling is 2.8 m so 0.7 m remains above the door opening. The shear stress for concrete that is not reinforced is one of the boundary conditions for increasing the size of the openings. Assuming a useful height of only 0.6 m, a wall thickness of 0.18 and a calculation value for the horizontal shear shearing strain of 0.65 N/mm² a shear force of approx. 70 kN can be absorbed. With the existing floor spans of 3.6m, a floor thickness of 0.12m with a finishing layer of 0.05 meter sand-cement and an imposed load of 1.5 kN/m² this gives a calculable value of 25.7 kN/m and allows a free opening of more than 5 metres. These measurements are not usually needed. Even so assuming a free span of 5 metres, the maximum support moment is ca. 60 kNm. Still assuming an interior lever arm of 0.6 m a tensile force of 60kNm/0.6 m = 100 kN is necessary. Depending on the type of steel chosen, there must be between 280 en 450 mm² of steel on the upper side of the beam and half of this value on the underside.
If insufficient reinforcement is in the floor and underside of the beam, the amount that is lacking must be added. The reinforcement can be bonded onto the construction floor and onto the sides and bottom of the beam. Protection against fire can be provided by a covering of shotcrete.

 existing structure  openings every two floors same structural system for the floors  openings every floor different structural system for the floors

Figure 14:

The 0.6 meter high beam serves as a visual separation. For good use of space, solutions must be sought which permit the entire beam to be sawn away. A free span for a 0.12 m thick floor could amount to 3.6 meter. The big problem is that there is scarcely any reinforcement in the floor running in the direction of the walls. How can this be solved? On the upper side the finishing layer of the floor can be removed reinforcement added and instead of a new finishing layer a compression floor of self-compacting concrete, which becomes very flat, can be laid. To ensure the reliability of the connection between the old floor construction and the new pressure layer it is recommended that short adhesion anchors are used. On the underside of the floor, if there is a flat ceiling, no possible solution can be found on the underside of the floor. Naturally reinforcement must be fixed to the underside by using adhesives and then protected against fire by a layer of shotcrete. There will always some increase in thickness/thickening.

A variation on this is to make the opening on the floor side 3.6 metres and chamfer it above the height of 2 m. Now it is possible to find a solution without the use of bottom reinforcement. The bevelling reduces the free span to 2.2 metres. The moment for the upper reinforcement can be laid in the concrete topping. The support moment is now $\frac{1}{12} \times 25.7 \text{kN/m} \times 2.2 \text{m}^2 = 10.37 \text{kNm}$. The moment of span is only half of this, that is 5.2 kNm. Assuming a contributing strip width of 2 metres the bending tensile stress in the concrete is $5.2 \times 106 \text{kN/m}^2 W \text{met} W = \frac{1}{6} \times 2000 \times (170)^2 = 0.06 \text{N/mm}^2$. There safety factor of 10 against collapse. In spite of this, builders will have some ungrounded anxiety about not using any reinforcement under the floors in the direction of the walls. Questions arise if there is any possibility of there being discontinuities gravel pockets and the like. Although the method of construction makes this unlikely, the real effect is that if a crack occurs in the middle of a span there is a small increase in the support moment. The support moment can increase by a maximum of 10.37 kNm to 1.5. 10.37 kNm=15.55 kNm is less than 300 mm^2.
The amount of reinforcement that must be added to these types of building to achieve large-scale interventions is small. The possibility to make better use of existing buildings is now within our grasp. With regard to the safety of buildings, we should make creative use of it.

Postscript
From this paper it appears that to date there are still untried ways to adapt concrete structures to meet the desire for new uses. Naturally there are also impediments or extra points that need attention. Such an impediment is formed by walls that were added later without reinforcement. It may then be necessary to reinforce these walls at a later date by using bonded laminates of steel or other material. Probably it is easier to make the tension connections directly with the floor elements next to the wall, although the wall must still bear the tensile forces. Some further research into this is certainly needed, because owing to the variation in deformation between the old floor and the new one, some shear forces will also have to be borne by the floor. Apparent impediments can thus be removed. Extra attention must be paid to the shrinkage of the new concrete. It may be assumed that the existing construction is worked-out in terms of shrinkage and creep, so temperature and humidity deformation can be neglected since the construction is subject to inside climatic conditions. The fresh new concrete will shrink and creep owing to hardening and time effects. Two alternative are needed for this. The first is to provide the new walls with a finely distributed shrinkage mesh reinforcement so that cracks remain small. The second is to use a so-called ‘shrinkage reduced high performance concrete’ so that the maximum degree of shortening remains as small as possible.
Abstract
Our urban heritage consists of a considerable amount of reinforced concrete structures. From the designers point of view, the constructional resistance (load bearing capacity) should not change during the lifetime. Practically this may not be true. If deterioration processes will lead to loss of structural properties, structural safety needs to be evaluated. Unless one can be certain the structural degradation can be stopped effectively, these structures should be monitored. Non Destructive Testing (NDT)-methods can provide useful tools to assess structural integrity, and monitor deterioration. In this paper an inventory of methods is made, focusing on deformation, cracks, corrosion and moisture.
Practically this assumption is incorrect. In positive sense, concrete will gain strength in time, due to ongoing hydration. On the other hand, due to deterioration processes, structural integrity may decrease. Therefore, once deterioration processes lead to loss of constructional resistance, the structural safety should be evaluated. The parameters should be assessed from the actual structure.

The constructional resistance of a concrete structure can decrease by loss of constructional resistance of concrete, or loss of constructional resistance of the steel reinforcement. Familiar examples of degradation processes that cause loss of constructional resistance of the concrete are delayed ettringite formation and alkali aggregate reaction in concrete. Both reactions are disintegrating the concrete. Concrete constructional resistance can also drop due to cracks in concrete, due to unexpected local tensile stresses within the concrete. The most common form of loss of steel constructional resistance is the corrosion of steel. Corroding steel leads to loss of reinforcement. The corrosion products may cause internal stress in the concrete, which leads to spalling of the concrete cover, or may (in case of chloride induced corrosion), show bleeding of corrosion products at the concrete surface.

**Deterioration assessment and monitoring**

A inspection should be aimed at defining the extent of deterioration. Depending on the effectiveness of the rehabilitation measures, there may or may not be a need to monitor. Monitoring is a periodic or continuous inspection.

Inspection and monitoring can focus directly on the constructional resistance, by assessment of the response of the actual structure to loads and deterioration processes. Examples are: periodic visual inspection, periodic deformation measurements, or controlled load-deformation testing. Inspection and monitoring can also focus on parameters which provide information about the amount of deterioration and the deterioration process. Examples are: radar measurements (finding cracks or deteriorated areas in the concrete), potential mappings (finding corroding area's), moisture measurements (AAR and delayed ettringite formation will only occur at sufficient availability of water; steel corrosion speed depends on the amount of moisture) and concrete resistance measurements (indirect moisture measurement).

In this paper several (semi) non destructive techniques for inspection and monitoring are discussed, which may be useful to make a constructional assessment, focusing on the following aspects:

1. deformations
2. loss of concrete integrity and cracks
3. moisture and concrete resistance
4. corrosion of steel

**Deformations**

Deformations can occur for different reasons. Failures in design or construction, ground settlement, excessive loads, mechanical impacts, etceteras. In case of steel corrosion, deformations can be a warning sign, indicating the structural integrity may be at stake. Most structures will have "warning ability", i.e. the structure will deform excessively before collapse will take place. In these cases deformations will lead to reinforcement yield and cracking of the concrete. Expansion of concrete because of AAR and delayed ettringite formation can cause structure deformations. Mostly these expansions lead to cracking of the concrete surface.

**Loss of concrete integrity and cracks**

The concrete integrity can change if local or general changes occur in the modulus of elasticity or the concrete strength. Concrete tensile strength can change or disappear if the concrete cracks. Different types of cracks can be distinguished:
Micro cracks in the concrete cover
Mostly caused by poor workmanship during construction. Drying shrinkage, thermal effects during hydration, high water-cement ratio and bad mixing are common causes. These cracks may decrease the durability of the reinforcement protection by the concrete cover. Bending moments may also cause micro cracks. These are usually perpendicular to the reinforcement, and not considered to be of influence on the durability.

Macro cracks in the concrete cover
These cracks are usually perpendicular to the concrete surface. They can also be caused by poor workmanship during construction. Other familiar causes are: spalling of the concrete due to reinforcement corrosion, deformation cracks, cracks due to internal concrete expansion (AAR and delayed ettringite formation), and cracks due to thermal effects. Macro cracks in the concrete cover can be a threat to the reinforcement.

Micro cracks in the concrete mass
Common causes of micro cracks in the concrete mass are poor workmanship and deterioration processes like AAR and delayed ettringite formation. Concrete containing extensive micro cracks can be a good environment for AAR, since the cracks can transport moisture and alkali to the reactive aggregate.

Macrocracks in the concrete mass
Macrocracks in the concrete cover may proceed into the concrete mass. Furthermore, delamination parallel to the concrete surface may appear due to main tensile stresses in the concrete, exceeding the concrete tensile strength. AAR and delayed ettringite formation may also cause internal macro cracks, parallel to the main plain of reinforcement. Macro cracks in the concrete mass can decrease the shear capacity the structure, and (in lesser extent) the moment capacity.

Moisture and concrete electric resistivity
Moisture and concrete electric resistivity measurements are related. The resistance of concrete depends on the ability of ions to transport through the moisture in concrete cracks and pores. The more the concrete pores are filled, the easier ions can transport, and the lower the concrete electric resistivity. Moisture in concrete is of importance for most deterioration processes in concrete:
* ingress of chlorides into the concrete and carbonatation are moisture related;
* the corrosion speed of steel in concrete depends on the moisture content;
* the expansion rate of the concrete due to delayed ettringite formation and AAR depends on the moisture content

Corrosion of steel
Corrosion of steel is an electrochemical reaction. Along a corroding reinforcement bar anodic areas will form, were steel will be oxidised to positive steel ions, and cathodic areas where water and oxygen will be reduced to negative hydroxide ions. From the anode to the cathode electrons will flow through the steel. From the cathode to the anode negative ions will transport through the concrete pore solution. The electric field in the concrete can provide information about the steel corrosion. The local loss of reinforcement diameter depends on the anode-cathode ratio. In case of a local anode and a big cathode (pit corrosion) the local corrosion speed will be much higher than when the ratio is smaller.
Several NDT methods for inspection and monitoring of reinforced concrete

In the appendix in table 1 an inventory of non destructive techniques is presented, for the evaluation of the 4 aspects discussed above. Table 2 shows for each method of monitoring some advantages and disadvantages. In the following paragraphs several NDT-methods have been described in more detail.

Vibrating wire settlement sensors [17]

General

The principle of this method is that of communicating vessels (figure 2). Two floaters are placed in reservoirs, partly filled with de-aired antifreeze solution. The reservoirs are connected a tube, connecting the fluids. The floaters are each connected to a wire. A vibrating wire pressure transducer measures the strain in the wires, which is translated in the relative displacements between the two floaters. Since the actual measurement is electric, the method is well suitable to use remote control. In windy locations the upper air spaces are connected by a second tube, to ensure uniform atmospheric pressure. Temperature sensors compensate thermal effects.

![Vibrating wire settlement sensor diagram](image)

Figure 2: vibrating settlement sensor [3 and 17]

Execution and interpretation

The reference sensor must be placed at a fixed point (e.g. no vertical displacements). The other can be placed at the vertically deforming structure. The fluid levels must be placed at roughly the same height. The transducers are connected to a readout box. The system is particularly suitable where high resolutions are required. Changes as little as 0,02% of the range are detectable. The maximum range is 18 meter.

Radar measurements

General

Radar technology is an electromagnetic method to detect delamination or debonding between materials (i.e. bridge decks) by mapping changes in reflection amplitude at a layer boundary (debonding) or by identifying planar reflections within an otherwise homogeneous material (delamination). Reflection of radar waves are the result of internal properties of the material (permittivity, magnetic permeability). Radar is an effective method to measure delamination of i.e. asphalt-covered decks, internal cracks parallel to the structure as well as the thickness of the cover. Moisture in the cracks may distort the
Moisture measurements are possible only up to restricted depth. The use of dual frequency radar allows more accurate characterisation of the defects. The performance of high frequency radar antennas in stead of antennas in the range of 1 GHz and 1.5 GHz for detecting i.e. ducts below a mesh of reinforcing bars is under development and seems promising (see figure 3). New technologies that permit assessment of bridge decks at traffic speed combined with automated signal processing and imaging are currently under development.

![Figure 3: Photograph a test specimen including the tendon ducts below reinforcing bars. The radargrams show the measurement results for each antenna.](image)

**Execution and interpretation**

Digital radar systems enable rapid survey coverage and comprehensive mapping of defects in highway and bridge structures. Modern radar systems incorporating high frequency antennas, high capacity digital storage and automated location referencing systems enable rapid and comprehensive data acquisition. Data processing can improve the clarity of collected data and use of analysis software can produce powerful plan views, including 'slices' through the structure to examine particular depth windows. There is, however, still a need for objective interpretation of these signals. Interpretation of the data requires consideration of the following aspects [21]:

*Concrete*
* The presence of thin layers of material close to the boundary or zone under examination can result in additional reflections;
* Variations in the density or moisture content of the materials to either side of the depth window can result in variations in the reflection amplitude similar to those caused by delamination;
* The presence of reinforcement or other metallic materials distorts the measurements in deeper layers.

**Impact Echo**

**General**

Impact-echo is an elastic wave method and has several applications in the assessment of different concrete deterioration processes. Impact-echo has recently become more widely employed in civil engineering thanks to the availability of higher-speed data acquisition systems and computers which, at the data analysis stage, permit to perform advanced signal processing and frequency analysis. The principle of impact-echo method is based on multiple reflections of an acoustical wave impact between the surface and any internal reflector. Reflected waves are picked up by a transducer at the receiver position - next to the impact - so that compression wave arrivals will be dominant. A waveform emerges in the time domain by the first and subsequent reflection arrivals. Different structural element shapes produce different responses when subjected to impact. This method can be used to detect delamination in concrete with or without overlays. Other applications include characterisation of surface-opening cracks, measurements of concrete pavement thickness, detection of voids in grouted tendon ducts, and analysis of interfacial bond quality in concrete. A standard procedure for measuring the longitudinal wave speed and the thickness of concrete plates has been submitted to the ASTM.

![Figure 4: Principle of impact-echo testing][24]

**Execution and interpretation**

Applicability of impact-echo can be found in a wide variety of concrete structures. Impact-echo is a wave propagation-based technique which uses frequency domain analysis for data interpretation. Frequency spectrum analysis is performed on the waveform obtained from a mechanical elastic impact applied on the surface of the concrete element. By applying a local impact on the surface of the test object, a transient stress pulse is generated, which propagates into the concrete as compression, shear and surface waves. The compression and shear waves, which travel through the material, are partly reflected by any internal interface or discontinuity such as reinforcements, ducts, defects, delaminations. These waves are almost completely reflected if the second material is air, such as in the presence of a void or at the external boundaries of the element under investigation. The interpretation of

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time domain data can be time consuming and uncertain if the geometry of the test object is complex and/or if the reflecting parts are located close to each other. Therefore, analysis of the data is performed in the frequency domain after plotting the frequency spectrum of the waveform (a graph of amplitude versus frequency components obtained by performing a Fast Fourier Transformation on the time domain waveform). The depth (d) of each reflector would be calculated by dividing the wave velocity (v) by the measured frequency of the echo signal (f): \( d = \frac{v}{2f} \). (see figure 4)

**Acoustic emission**

*General, execution, interpretation*

Acoustic emission can be described as a technique of ‘listening to the noise’ that is produced by the structure. Acoustic emission is a NDT-method based on the analysis of elastic waves generated by the sudden release of stored elastic energy in the material. Therefore, there must be some process taking place in the material or structure like propagation of micro-cracks (for instance due to AAR or ongoing hydration), in order to detect a signal. In case of passive acoustic emission, no signal is used to initiate cracks. For active acoustic emission, an impulse is applied to the concrete surface, and the response is measured. A property known as the Kaiser effect is used to detect crack propagation through acoustic emission. The Kaiser effect states that emission of acoustic waves occurs only if a load surpassing the previous maximum load is applied to the structure. Therefore the acoustic emission waves depend upon the past history of the structure. But, for concrete structures, the Kaiser effect may not be a reliable indicator of the loading history.

To make AE measurements, one or more acoustic sensors should be placed at the concrete surface (figure 5).

The technique can detect rebar corrosion in concrete (resulting in local cracking of the concrete) earlier than other methods (such as half-cell potential measurements) though no field experiments have yet been documented [23]. Techniques for identifying crack location, type, and orientation by the analysis of acoustic emission is under development in laboratories. Signal distortions due to traffic vibrations may restrict application in bridge inspections.

![Figure 5: AE sensors on masonry and on concrete](image)

**Concrete resistivity measurements**

*General*

If potential measurements indicate that there is a high probability of active corrosion occurring, concrete resistivity measurement can be subsequently used to estimate the rate of corrosion. Concrete resistivity values have been indirectly related to the corrosion rate of the steel reinforcement from practical experience. Concrete moisture content and hence the electrolytic resistivity of concrete are important parameters influencing the corrosion process of the reinforcing steel. According to [8] the corrosion rate is considered to be inversely proportional to concrete resistivity. Depending on environmental exposure
conditions and concrete quality and composition, resistivity may vary between 10 to 105 (m. A lower water to cement ratio or the addition of blast furnace slag, fly ash, silica fume or other supplementing materials result in an increase of concrete resistivity.

**Execution**

For existing structures the determination of concrete resistivity is carried out by using the four point Wenner technique (figure 6). This method has been adapted from that used for geotechnical surveying and involves passing an alternating current between the outer pair of four equispaced probes in contact with the concrete surface. The ratio between the voltage measured between the inner probes and the applied AC current is transformed into concrete resistivity using a geometry factor. However, the inhomogeneity of the concrete material, pronounced moisture profiles in the concrete cover, i.e. due to pre-wetting of the concrete surface, and the presence of highly conductive reinforcing steel bars may influence the results.

In addition to proprietary versions of the four-probe system, less accurate but cheaper two probe systems are also available.

![Figure 6: Concrete resistivity measurement using a Wenner four probe](image)

**Interpretation**

The following interpretation of resistivity measurements from the Wenner four-probe system is empirical and refers to depassivated steel (L+B):  
- > 200 kohm * m: low corrosion rate  
- 100 - 200 kohm * m: low to moderate corrosion rate  
- 50 - 100 kohm * m: high corrosion rate  
- < 50 kohm * m: very high corrosion rate  

If the resistivity of the concrete is measured in conjunction with potential mapping, it is claimed that this can be used as an aid to assess the severity of the problem. A large potential gradient associated with a low concrete resistivity will normally result in a high corrosion rate.

**Multi ring electrodes**

**General**

Multi ring electrodes (MRE, figure 7) are developed to determine moisture distribution in concrete.

The time and depth depended distribution of electrolytic resistance (depending of the water content of the concrete pores) can be determined by AC current measurements between metal rings. Depending of the use, different types of MRE are available. If one is interested in the moisture profiles in a concrete

Concrete
cover (for instance in case of steel corrosion, or for the evaluation of the effect of an applied coating) small multiring electrodes are available, measuring between the surface and the reinforcement [18]. Larger MRE's may be used if one is interested in the moisture content in the concrete bulk [1] (for instance in case of AAR).

**Execution**

For monitoring of an existing concrete structure, a hole needs to be drilled perpendicular to the concrete surface. The MRE is placed in the centre of the hole (figure 8). Next, the hole is injected carefully. The void between the MRE and the concrete should be 4 mm or larger, depending on the distance between two rings. The grain size should be adjusted to the void width, in order to get the drilled hole fully injected. The conductivity of the injection mortar should be roughly equivalent or lower than the surrounding concrete. Measurements are always taken between two neighbouring rings.

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**Concrete**

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Interpretation

One should be careful translating a concrete resistance into a moisture content. Although a moisture profile can be derived from the electronic resistance, exact interpretation of moisture content is only possible if the properties of the injection mortar and the concrete are exactly known. One should be aware, that due to ongoing hydration of the injection mortar the resistance may change. Furthermore, the difference in pore structure between the concrete and the injection mortar may cause a difference in moisture content between the mortar and the concrete. If the bore hole is not fully injected, moisture may gather in the void, corrupting the measurement. "Short-circuits" due to reinforcement close to the bore hole may influence the measurement, if the void between the MRE and the concrete is small in relation to the distance between two rings.

Time Domain Reflectometry (TDR) [4 and 5]

General

Originally the Time Domain Reflectometry (TDR) was designed to detect faults in electronic systems, e.g. transmission lines and cables. A voltage pulse is injected into the cables. The pulse propagates along the cable as an electromagnetic wave. The pulse shape and transit time depend on the cable properties, length, and the termination of the cable where the pulse is reflected. Moisture measurement in porous materials using Time Domain Reflectometry is based on the principle of dielectrical constant measurement. It is a well-known method for measurement of moisture in soils, and is recently been adjusted to be applied on solid building materials. The new sensors, shaped like a fork with two rods, are longer than those used for soil. A pulse generator is transmitting electromagnetic waves into the rods. The wave is reflected at the end of the rod. The wave travel time is measured (figure 9).

![TDR principle](image)

Figure 9: TDR principle [5]

This transit time is dependant on the dielectric constant of the surrounding concrete. The dielectric constant of concrete depends on the moisture content.
Execution and interpretation

The two rods from the TDR-probe can be inserted in two small holes in the concrete surface. TDR measures reflections both in the cables and in the rods, depending on the pulse amplitude (voltage). The reflected TDR-pulse is scanned by the “sampling method”. Each point of the pulse signal is measured as a voltage value at a distinct time. Based on the voltage signal the transit time is derived graphically (figure 11). From the probe transit time the moisture content can be determined (figure 12). The interpretation might be difficult, because the transit time in the probe is very small.

Alternatively the Time domain Reflectometry with Intelligent Micro module Elements (TRIME)-method can be used. This method is a specially designed TDR-technique to measure material moisture. The difference with conventional TDR-measurements is, that the measurement of the TDR-pulse is realised by direct time measurements, at distinct voltage measurements (figure 13 and 14)
The principle of NMR is based on the interaction of a transmitted radio frequency (rf) field, and the magnetic properties of an atomic nucleus (mostly hydrogen atoms, since it possesses a significant magnetic moment) in a high static magnetic field. NMR measures the proton density and mobility in a sample volume. In concrete both the hydrogen in binding state as hydrogen bearing water can be determined at different depths in the sample. Until recently, NMR could only be applied for small samples in laboratory conditions. Moisture in concrete cores could be studied, by moving a concrete core through a magnetic field. New possibilities seem to appear, now a new version of NMR, One Side Access NMR (SA-NMR) is under development (figure 15).

OSA-NMR generates a rf magnetic field from a coil, placed at the concrete surface. The rf field from the surface coil at a frequency will only excite protons in a small sensitive volume, in which the hydrogen protons will be in resonance condition. In this manner, moisture profiles in the outer centimetres of the concrete can be derived. Alternatively, OSA-NMR can be used to determine concrete permeability. Some OSA-NMR inspection units are currently in prototype stadium (figure 16).
Execution and interpretation

To measure moisture profiles, the unit containing the coil is placed on the concrete surface. By measurement at different NMR resonance frequencies a moisture profile can be derived. If OSA-NMR is used to determine the concrete permeability, a pressure driven experiment must be performed. Water is pressed into the concrete at 4 bar for at least a half an hour. Afterwards the change in moisture profile in time is followed. A permeability coefficient (K) is derived from the change in moisture profile.

Potential mapping

General

The objective of potential measurements on reinforced concrete structures is to identify areas of active corrosion without disrupting the concrete cover to the reinforcing steel. This electrochemical technique allows for a rapid and cost effective overview of the condition of a structure with respect to corrosion of the embedded steel reinforcement. Although half-cell potential mapping is a simple and widely used technique, it does not give any indication of the extent or intensity of corrosion. The electrochemical potential characterises the state of a metal in its environment, thus it allows to distinguish between actively corroding and passive rebars. The potential of the reinforcing bar embedded in concrete is measured as a potential difference against a reference electrode (half cell) which has a defined, constant and reproducible potential. Thus experimentally measured potentials differ with the type of reference electrode used. In practice the following reference electrodes are used: Cu/CuSO4 (CSE), Calomel (SCE) and Ag/AgCl (SSE). Therefore reported potentials should always be referred to the reference electrode employed. Steel in concrete can adopt different potentials dependent on the electrochemical condition, the humidity conditions and the chloride content. Pronounced variations in the measured potential may occur due to the difference in corrosion state between actively corroding and passive rebars. The potential difference between active and passive steel may amount to 500 mV (figure 17), setting up a macrocell with a current flow between adjacent anodic and cathodic areas. Consequently, an electric field in the concrete is developed which is identified by pronounced potential gradients measured at the concrete surface (figure 18).
Execution
Potential measurements can be performed with a single electrode (point measurements) or with one or several potential wheel electrodes (potential field measurements). For simplicity the description is related to measurements with a single electrode. (figure 19)

A suitable reference electrode, frequently Cu/CuSO₄, is put on a wet domestic sponge soaked in a solution of detergent onto the concrete surface and connected to a high impedance voltmeter. The other
input is connected to one of the rebars of the reinforcement network, making a metallic, electrically sound contact. A necessary prerequisite is that the reinforcement network has to be completely electrically continuous. This can be checked prior to potential mapping by measuring the electrical resistance between remote sections of the steel network. In addition, a good electrolytic contact between reference electrode and concrete surface is essential. This can be achieved by using a sufficiently wetted sponge. However, severe wetting may change the local exposure conditions to such an extent that a significant potential drift occurs during measurement. This situation has to be avoided since it renders the measurement results useless for reliable interpretation.

**Interpretation**

The results obtained from potential mapping need careful interpretation in order to determine correctly which areas of the structure are actually corroding. Due to the concrete cover the reference electrode cannot be placed at the steel/concrete interface and this means that the measured potential is not the “true” potential. In general, corroding areas on rebars are identified by the most negative potential values. The criteria according to C876-91 were devised empirically from chloride-induced corrosion of cast-in-place bridge decks in the USA.

According to [8] the following exposure conditions may be associated with typical ranges of potentials:

<table>
<thead>
<tr>
<th>Range of potentials of carbon steel in concrete (vs. CSE)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>wet concrete without oxygen</td>
<td>-1.1 V</td>
</tr>
<tr>
<td>humid, chloride contaminated concrete</td>
<td>-0.6 V</td>
</tr>
<tr>
<td>humid, chloride free concrete</td>
<td>-0.4 V to +0.1 V</td>
</tr>
<tr>
<td>humid, carbonated concrete</td>
<td>0 to +0.2 V</td>
</tr>
<tr>
<td>dry, carbonated concrete</td>
<td>0 to +0.2 V</td>
</tr>
<tr>
<td>dry concrete</td>
<td>0 to +0.2 V</td>
</tr>
</tbody>
</table>

Normally, very negative potential values are indicative of active corrosion, however, some caution has to be exercised for continuously wet or water-saturated concrete. The limited availability of oxygen may result in very negative potentials although the corrosion rate is insignificant. Occasionally this situation is encountered in very wet concrete contaminated with chlorides in bridges and submarine structures with potentials in the range -0.6 to -1.1 V versus CSE.

For passive steel the potential is largely determined by the oxygen content of the concrete. In normal, moist concrete when the oxygen supply is good the passive potential is usually in the range -0.2 to 0 V versus CSE whereas in very wet concrete with restricted oxygen supply the passive potential can be as low as about -0.6 volt versus CSE.

Corrosion is best located from iso-potential contour maps by considering the magnitude of the potentials and qualitatively assessing the potential gradients.

It has been found that there are no absolute potential values to indicate corrosion hazard in a structure, in contrast to the interpretation given in the ASTM C876-91 that relies on a fixed potential value of -350 mV SCE. According to [8] measurement results have to be examined statistically to distinguish between corroding and passive areas. Therefore the potential distribution over all the structure’s surface must be known to assign a certain probability of corrosion to a certain region.

A first estimation of the corrosion state may be made by looking at the potential gradients. The potential depends on a number of factors, including concrete moisture content, chloride content, temperature, carbonation of the concrete, and cover depth.

However, a physical investigation is essential to evaluate if there is a reasonable correlation between rebare corrosion and anodic areas as identified by potential mapping. Any correlation between potential and corrosion rate is fortuitous and is often due to holding other variables constant in laboratory tests.
Linear polarisation measurements

General
Non-destructive techniques to determine the corrosion rate are needed in order to detect steel passivation, to predict the residual life of a structure and to monitor the efficiency of a repair system. Potential mapping and resistivity measurements are both indirect methods for assessing the rate of corrosion, but there has been much recent interest in developing a means of performing perturbative electrochemical measurements on the steel itself to obtain a direct evaluation of the instantaneous corrosion rate. The method is based on the Stern-Geary equation which implies that the corrosion rate, most frequently identified by the corrosion current density, $i_{\text{corr}}$, is inversely proportional to the resistance against polarisation at the free corrosion potential, $E_{\text{corr}}$. This so-called polarisation resistance, $R_p$, is calculated from the ratio between a potential shift ($E = (E - E_{\text{corr}})$, and the applied external current $I_{\text{app}}$. The corrosion current density, $i_{\text{corr}}$ (mA/m$^2$), is then calculated from the expression $i_{\text{corr}} = B/(R_pA_{\text{steel}})$ where $B$ is a constant using 26 mV for actively corroding steel and 52 mV for passive steel, and $A_{\text{steel}}$ is the steel surface area.

Execution
The measurement is performed with sophisticated equipment comprising a reference and a counter electrode, together with a variable low voltage DC power supply. (figure 19)

![Figure 20: Set up for determination of the corrosion rate by linear polarisation using a guard ring (Gecor6)](image)

Both the reference and the counter electrode (usually a stainless steel disk) are placed on a wet pad onto the concrete surface and a current is passed from the counter electrode through the concrete cover to the reinforcing steel directly below it. The resulting change in potential must be kept to less than approximately 20 mV for the equation to be valid. Adequate compensation for the voltage drop over the concrete cover is crucial for the correct interpretation of the results. As for potential mapping and resistivity measurements, electrical continuity of the reinforcement network is essential for successful application. However, as a consequence of continuity, the area of the steel section constituting the area of steel sensed by the counter electrode is not well defined. Assuming a 1:1 relationship between counter electrode and area of steel polarised can lead to severe overestimation of the actual corrosion rate. To overcome this problem the set up of one of the commercial equipment's comprises a primary counter electrode surrounded by a ring shaped secondary counter electrode termed a guard ring.
The following broad classification for rebar corrosion have been developed from field and laboratory investigations for equipment’s with a guard ring device:

- **passive condition**: \( \text{icorr} \leq 1 \text{ mA/m}^2 \)
- **low to moderate corrosion**: \( 1 \text{ mA/m}^2 < \text{icorr} < 5 \text{ mA/m}^2 \)
- **moderate to high corrosion**: \( 5 \text{ mA/m}^2 < \text{icorr} < 10 \text{ mA/m}^2 \)
- **high corrosion rate**: \( \text{icorr} \geq 10 \text{ mA/m}^2 \)

However, it should be emphasised that the results obtained for laboratory specimens may be in error by a factor 2; for in-situ measurements the order of magnitude seems to be more appropriate to qualify the actual corrosion rate. Moreover, the results refer to a corrosion current density averaged for the steel surface under study. For situations with strong variations in corrosion attack along the rebar, e.g. for chloride-induced corrosion, this average density may severely underestimate the local rate of attack.

In view of the large dimensions of real structures, some doubt has been expressed regarding the reliability of the method for conditions conducive to macocell corrosion. In these situations corrosion rate measurements may erroneously identify passive steel as actively corroding [15].

### Acknowledgements

During 1998 and 1999, the Dutch ministry of transport, public works and water management is working on a project of monitoring of AAR in concrete. In this scope, a lot of effort has been put in studying and inventorying ways to measure deformations, moisture and cracks in concrete. Much of the work was done in co-operation with TNO, which was also asked to make a literature study of the monitoring of AAR [2]. Many of the items discussed in this paper, are derived from this work. From TNO, I would like to recognise Ton Siemes and Jeanette Visser, for their outstanding work. Furthermore, I would like to recognise Harry van Gaanderen, from the Dutch contractor BALM, for his work done on making a monitoring plan for 16 AAR affected bridges [3]. Many new ideas came from this plan.

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### Appendix

**Inventory of NDT-methods for inspection of reinforced concrete structures**

Monitoring aspects:

1. deformations
2. concrete integrity and cracks
   - 2a. Micro cracks in the concrete cover
   - 2b. Macro cracks in the concrete cover
   - 2c. Micro cracks in the concrete mass
   - 2d. Macrocracks in the concrete mass
3. moisture
4. corrosion

<table>
<thead>
<tr>
<th>Category</th>
<th>inspection / monitoring method</th>
<th>aspects</th>
<th>principle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual/optical Methods</td>
<td>Visual inspection</td>
<td>1, 2b</td>
<td>Visual examination of the structure, general impression.</td>
</tr>
<tr>
<td></td>
<td>Surveyor</td>
<td>1</td>
<td>Relates horizontal and vertical positions on the structure to a reference point, using an optical instrument.</td>
</tr>
<tr>
<td></td>
<td>Crack loupe</td>
<td>1, 2a, 2b</td>
<td>Visual inspection of cracks through a loupe, quantifying the crack with</td>
</tr>
<tr>
<td></td>
<td>Crack mapping</td>
<td>1, 2b</td>
<td>Manual measurement of cracks. Periodically drawing a crack map of a part of the construction on a transparent plate.</td>
</tr>
<tr>
<td></td>
<td>Photo / video monitoring</td>
<td>1, 2b</td>
<td>Comparing different images of the structure in time</td>
</tr>
<tr>
<td></td>
<td>Holographic interferometry</td>
<td>1</td>
<td>Holographic images are constructed using reflections of a laser beam. 3D image can be made, and followed on video.</td>
</tr>
<tr>
<td></td>
<td>Moiré method</td>
<td>1</td>
<td>A grid is made on a structure. A photo (using laser light) can show the deformed grids at different times.</td>
</tr>
<tr>
<td></td>
<td>Demec points</td>
<td>1, 2b</td>
<td>3 demec points are placed around a crack. Periodic measurement of the distance between the points provides information about change in crack widths and deformation along the crack surface.</td>
</tr>
<tr>
<td></td>
<td>Invar reference bar</td>
<td>1</td>
<td>3 or 4 gauge points are placed in a triangle or a square at some distance apart (for example: 1 meter). The distance between these points is measured periodically, using a bar that is invariant for changes in temperature and moisture.</td>
</tr>
<tr>
<td></td>
<td>Optic glass fibres</td>
<td>1</td>
<td>Optic glass fibres are paved on the surface or in a structure. Expansion of the fibre leads to a change in light transmission.</td>
</tr>
<tr>
<td></td>
<td>Optic glass fibres with multiple reflectors</td>
<td>1</td>
<td>The optic glass fibre with reflectors is placed on the material surface. The time it takes for a short light impulse to travel up to each reflector and back is measured.</td>
</tr>
<tr>
<td></td>
<td>Infrared thermography</td>
<td>2a, 2b, 3</td>
<td>Surface defects and moisture influence heat transport in the material. This results in temperature variations, which can be visualised using infrared thermography.</td>
</tr>
<tr>
<td>Elastic wave methods</td>
<td>Ultrasonic pulse velocity</td>
<td>2</td>
<td>Loss of structural integrity is evaluated by the assessment of the dynamic modulus of elasticity.</td>
</tr>
<tr>
<td>----------------------</td>
<td>---------------------------</td>
<td>---</td>
<td>---------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Pulse echo</td>
<td></td>
<td>2</td>
<td>Piezoelectric crystal is vibrated by pulse generator. Receivers measure reflections, refractions and wave conversions in the reflected signal.</td>
</tr>
<tr>
<td>Impact echo</td>
<td></td>
<td>2</td>
<td>Mechanical impact is applied to the material surface. Further like pulse echo</td>
</tr>
<tr>
<td>Active acoustic emission</td>
<td></td>
<td>2</td>
<td>Pressure waves are applied to the material. Like pulse echo. Receivers at different places on the structure can assess detailed views of the internal structure, using tomography.</td>
</tr>
<tr>
<td>Ultrasonic pulse propagation</td>
<td></td>
<td>2b</td>
<td>A transmitter and a receiver are placed on both sides of a crack. From the measured travel time the crack depth can be derived.</td>
</tr>
<tr>
<td>Spectral analysis of surface waves</td>
<td></td>
<td>2</td>
<td>Dispersion measurements of Rayleigh waves between two receivers placed at some distance from the transmitter, are transformed in depth profiles of material integrity.</td>
</tr>
<tr>
<td>Passive acoustic emission</td>
<td></td>
<td>2,4</td>
<td>Like active acoustic emission, though no signal transmitter is used. New formed cracks are registered.</td>
</tr>
<tr>
<td>Vibration method</td>
<td></td>
<td>2</td>
<td>Assessment of own frequencies of a structure using a vibrator and an electronic audio oscilloscope. Mechanical vibrations are measured using acceleration transducers at the material surface.</td>
</tr>
<tr>
<td>Mechanical impedance measurement</td>
<td></td>
<td>2</td>
<td>An impedance sensor, consisting of a vibrator, impulse sensor and acceleration sensor is applied to the material surface. A wide range signal spectrum is used. The local impedance as function of the frequency is derived.</td>
</tr>
<tr>
<td>Vibrating wire gauges</td>
<td></td>
<td>1,2</td>
<td>Measurement of crack growth and expansion. A snare is placed in a tube with a coil. The snare can be excited by a magnetic field in the coil. Its own frequency is a measure for the expansion.</td>
</tr>
<tr>
<td>Electric methods</td>
<td>Electric resistance</td>
<td>3,4</td>
<td>Measurement of electric resistance between electrodes, under altering current at 10 Hz - 10 kHz</td>
</tr>
<tr>
<td>Multiring sensors</td>
<td></td>
<td>3</td>
<td>Assessment of profiles of electric resistance in concrete, by placing a series of metal rings in the concrete. Measurement of resistance between each two adjacent rings</td>
</tr>
<tr>
<td>Dielectric constant</td>
<td></td>
<td>3</td>
<td>Measurement of dielectric constant using electrodes under high frequencies altering current.</td>
</tr>
<tr>
<td>Electric impedance spectroscopy</td>
<td></td>
<td>3,4</td>
<td>Measurement of electric impedance using electrodes under varying frequencies of altering current</td>
</tr>
<tr>
<td>Strain gauges</td>
<td></td>
<td>1, 2</td>
<td>Thin wire placed over a crack or on the concrete surface. If deformations take place, or the crack changes size, the resistance in the wire will change</td>
</tr>
<tr>
<td>Vibrating wire settlement system</td>
<td></td>
<td>1</td>
<td>Principle of communicating vessels. Two floaters are placed in reservoirs, partly filled with fluid. The reservoirs are connected by a tube. The floaters are each connected to a wire. A vibrating wire pressure transducer measures relative displacements between the two floaters.</td>
</tr>
</tbody>
</table>
The probable areas of corrosion are determined from measurements of concrete potentials, related to the reinforcement potential.

The polarisation resistance is determined from the potential shift and an applied external current. From the polarisation resistance the corrosion current is derived.

AC altering voltage perturbations are applied in a single frequency, and an AC density measurement is taken together with two higher harmonics.

Small constant current perturbation is applied to the steel reinforcement. The potential transient response is analysed.

Spontaneous, random fluctuations in current flow between two electrochemical isolated bars are recorded, together with the fluctuations in potential of one of them, measured against a reference electrode.

Hydrogen atoms in water are excited in their resonance frequency using a magnetic field. From the response the moisture content can be derived.

A nucleus and holder containing a coil is connected to the material surface. At surface deformations the coil will change length, and the magnetic induction field changes.

A transit time measurement of an electromagnetic wave in two parallel metal rods. The transit time is dependant on the dielectric constant of the surrounding concrete.

Reflection of radar waves because of internal properties of the material (permittivity, magnetic permeability).

Change of microwaves by moisture. Average moisture content can be determined, assuming the water is equally divided in the material.

Transmitter emits a gamma radiation at the material surface. A receiver registers radiation passing through the material or reflections, depending on the location of the receiver.

Radiation transmitter emits röntgen and gamma radiation at one surface of the material. A light sensible plate is placed on the other side. A photographic view of the inner of the structure is obtained.

A neutron beam is applied to the concrete. Absorption of neutrons mainly takes place in the water molecules. This causes a gamma radiation, which is measured.

Surface deformations, for instance by cracks, causes a spring, connected to the surface, to be stressed or pressed. The stresses can be measured.

Wooden dowels are placed in a hole in the material. After some time the relative humidity in the wooden dowel will be that of the surrounding material. The relative humidity is measured in the dowel.

Table 1: Inventory of Non Destructive Techniques (NDT) for inspection and monitoring (mainly derived from [1] and [2])

<table>
<thead>
<tr>
<th>Method</th>
<th>Technique</th>
<th>Code</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Potential mapping</td>
<td></td>
<td>4</td>
<td>The probable areas of corrosion are determined from measurements of concrete potentials, related to the reinforcement potential.</td>
</tr>
<tr>
<td>Linear polarisation resistance</td>
<td></td>
<td>4</td>
<td>The polarisation resistance is determined from the potential shift and an applied external current. From the polarisation resistance the corrosion current is derived.</td>
</tr>
<tr>
<td>Harmonic analysis</td>
<td></td>
<td>4</td>
<td>AC altering voltage perturbations are applied in a single frequency, and an AC density measurement is taken together with two higher harmonics.</td>
</tr>
<tr>
<td>Galvanostatic pulse transient analysis</td>
<td></td>
<td>4</td>
<td>Small constant current perturbation is applied to the steel reinforcement. The potential transient response is analysed.</td>
</tr>
<tr>
<td>Electrochemical noise</td>
<td></td>
<td>4</td>
<td>Spontaneous, random fluctuations in current flow between two electrochemical isolated bars are recorded, together with the fluctuations in potential of one of them, measured against a reference electrode.</td>
</tr>
<tr>
<td>Magnetic methods</td>
<td>Nuclear magnetic resonance</td>
<td>3</td>
<td>Hydrogen atoms in water are excited in their resonance frequency using a magnetic field. From the response the moisture content can be derived.</td>
</tr>
<tr>
<td>Inductive displacement meters</td>
<td></td>
<td>1</td>
<td>A nucleus and holder containing a coil is connected to the material surface. At surface deformations the coil will change length, and the magnetic induction field changes.</td>
</tr>
<tr>
<td>Electromagnetic methods</td>
<td>Time domain reflectometry</td>
<td>3</td>
<td>A transit time measurement of an electromagnetic wave in two parallel metal rods. The transit time is dependant on the dielectric constant of the surrounding concrete.</td>
</tr>
<tr>
<td></td>
<td>Radar reflection</td>
<td>2,3</td>
<td>Reflection of radar waves because of internal properties of the material (permittivity, magnetic permeability).</td>
</tr>
<tr>
<td></td>
<td>Microwave sensors</td>
<td>3</td>
<td>Change of microwaves by moisture. Average moisture content can be determined, assuming the water is equally divided in the material.</td>
</tr>
<tr>
<td>Nuclear methods</td>
<td>Radiometry</td>
<td>2</td>
<td>Transmitter emits a gamma radiation at the material surface. A receiver registers radiation passing through the material or reflections, depending on the location of the receiver.</td>
</tr>
<tr>
<td></td>
<td>Radiography (röntgen, gamma)</td>
<td>2</td>
<td>Radiation transmitter emits röntgen and gamma radiation at one surface of the material. A light sensible plate is placed on the other side. A photographic view of the inner of the structure is obtained.</td>
</tr>
<tr>
<td></td>
<td>Neutron-gamma technology</td>
<td>3</td>
<td>A neutron beam is applied to the concrete. Absorption of neutrons mainly takes place in the water molecules. This causes a gamma radiation, which is measured.</td>
</tr>
<tr>
<td>(semi) Mechanical methods</td>
<td>Mechanical crack meters</td>
<td>1,2b</td>
<td>Surface deformations, for instance by cracks, causes a spring, connected to the surface, to be stressed or pressed. The stresses can be measured.</td>
</tr>
<tr>
<td>Other</td>
<td>Wooden dowels</td>
<td>3</td>
<td>Wooden dowels are placed in a hole in the material. After some time the relative humidity in the wooden dowel will be that of the surrounding material. The relative humidity is measured in the dowel.</td>
</tr>
<tr>
<td>Category</td>
<td>inspection / monitoring method</td>
<td>advantage</td>
<td>disadvantage</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>--------------------------------</td>
<td>----------------------------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>visual/optical methods</td>
<td>Visual inspection</td>
<td>Simple to do. Possible to relate different observations.</td>
<td>Superficial, time consuming when performed detailed or regularly.</td>
</tr>
<tr>
<td></td>
<td>Surveyor</td>
<td>Simple to do, well developed</td>
<td>Time consuming</td>
</tr>
<tr>
<td></td>
<td>Crack loupe</td>
<td>Simple to do</td>
<td>Scaffolding may be needed. Local measurement</td>
</tr>
<tr>
<td></td>
<td>Crack mapping</td>
<td>Simple to do</td>
<td>Superficial, time consuming. Must always be performed by the same person to have comparable results.</td>
</tr>
<tr>
<td></td>
<td>Photo / video monitoring</td>
<td>Simple to do</td>
<td>Superficial, labour-intensive, very low resolution for crack survey</td>
</tr>
<tr>
<td></td>
<td>Moiré method</td>
<td>2D surface technique. Strains can be measured</td>
<td>Currently rarely used any more.</td>
</tr>
<tr>
<td></td>
<td>Demac points</td>
<td>Easy to do, cheap</td>
<td>Local measurement. Labour intensive.</td>
</tr>
<tr>
<td></td>
<td>Optic glass fibres</td>
<td>Relatively simple.</td>
<td>In experimental stage. Relatively expensive. Measurements in one line.</td>
</tr>
<tr>
<td></td>
<td>Optic glass fibres with multiple reflectors</td>
<td>More measurements in one line possible.</td>
<td>In experimental stage. Relatively expensive. Measurements in one line.</td>
</tr>
<tr>
<td></td>
<td>Infrared thermography</td>
<td>Highly developed. Quick and simple, low cost</td>
<td>Only surface cracks. Dependence on variations of moisture not clear.</td>
</tr>
<tr>
<td>Elastic wave methods</td>
<td>Ultrasone pulse velocity</td>
<td>Well developed, quick and simple, sensitive to local changes.</td>
<td>Influences of compression zone and reinforcement difficult to eliminate.</td>
</tr>
<tr>
<td></td>
<td>Pulse echo</td>
<td>Well developed for tomographic use.</td>
<td>Under development, including software.</td>
</tr>
<tr>
<td></td>
<td>Impact echo</td>
<td>Well developed for tomographic use.</td>
<td>Under development, including software.</td>
</tr>
<tr>
<td></td>
<td>Active acoustic emission</td>
<td>Internal cracks / structure can be determined</td>
<td>Low resolution. Very specialised. Still in development</td>
</tr>
<tr>
<td></td>
<td>Ultrasonic pulse propagation</td>
<td>Easy to learn. Well developed.</td>
<td>Sparsely used in practice.</td>
</tr>
<tr>
<td></td>
<td>Spectral analysis of surface waves</td>
<td>Good interpretation of material properties</td>
<td>Specialists needed for interpretation. High initial costs. Still in development</td>
</tr>
<tr>
<td>Method</td>
<td>Description</td>
<td>Advantages</td>
<td>Disadvantages</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>-----------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------</td>
<td>--------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Passive acoustic emission</td>
<td>Internal cracks / structure can be determined. Cracks occurring during monitoring can be determined. Reasonably developed.</td>
<td>Very specialised. Resolution unclear.</td>
<td></td>
</tr>
<tr>
<td>Vibration method</td>
<td>Very sensible for structural changes. Well developed</td>
<td>Difficult with complex geometrics.</td>
<td>Difficult to use in the field.</td>
</tr>
<tr>
<td>Mechanical impedance measurement</td>
<td>Well developed. Simple to use. Very sensible for structural changes.</td>
<td>Difficult with complex geometrics.</td>
<td></td>
</tr>
<tr>
<td>Vibrating wire gauges</td>
<td>Well developed, simple.</td>
<td>Local measurement</td>
<td></td>
</tr>
<tr>
<td>Electric methods</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Electric resistance</td>
<td>Well developed, simple and cheap.</td>
<td>Unequivocally is bad in concrete. Cracks or reinforcement may corrupt the measurement. Profiles are only possible in the cover zone</td>
<td></td>
</tr>
<tr>
<td>Multi-fibre sensors</td>
<td>Reasonably well developed, relatively quick and cheap, reasonably unequivocally</td>
<td>The measurement is done in the injection grout instead of the concrete. Bad injection may corrupt measurements. Properties of the grout (in time) should be known exactly.</td>
<td></td>
</tr>
<tr>
<td>Dielectric constant</td>
<td>Imaginary and real signal can be measured</td>
<td>Under development</td>
<td></td>
</tr>
<tr>
<td>Strain gauges</td>
<td>Well developed, quick, simple, cheap.</td>
<td>Local measurement</td>
<td></td>
</tr>
<tr>
<td>Potential mapping</td>
<td>Easy to do. Well developed.</td>
<td>Semi NDT. Measurements are useless if reinforcement is not continue. Delamination may corrupt measurements. Measured value not absolute. Reference needed</td>
<td></td>
</tr>
<tr>
<td>Linear polarisation resistance</td>
<td>Easy to do. Well developed.</td>
<td>Doubtful interpretation for lifetime prediction. Assumption equally divided corrosion over measured area.</td>
<td></td>
</tr>
<tr>
<td>Harmonic analysis</td>
<td></td>
<td>Under development</td>
<td></td>
</tr>
<tr>
<td>Galvanostatic pulse transient analysis</td>
<td></td>
<td>Under development</td>
<td></td>
</tr>
<tr>
<td>Electrochemical noise</td>
<td></td>
<td>Under development</td>
<td></td>
</tr>
<tr>
<td>Magnetic methods</td>
<td>Nuclear magnetic resonance</td>
<td>Less sensible for salt concentrations and cracks.</td>
<td>Limited depth. High initial costs. In experimental stage.</td>
</tr>
<tr>
<td>------------------</td>
<td>---------------------------</td>
<td>------------------------------------------------</td>
<td>--------------------------------------------------</td>
</tr>
<tr>
<td>Inductive displacement meters</td>
<td>Well developed, quick, simple, cheap</td>
<td>Local measurement</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Electro magnetic methods</th>
<th>Time domain reflectometry</th>
<th>Well developed technique for moisture measurements</th>
<th>Little experience with concrete.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Microwave sensors</td>
<td>Reasonably developed. Fast and simple. Not sensible for salt concentrations.</td>
<td>Unsuitable for high moisture contents. Depths up to 25 mm. Commercial equipment may not yet be available.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Nuclear methods</th>
<th>Radiometry</th>
<th>Well developed. Practically usable. Easy to learn</th>
<th>Specialised personal needed. Safety procedures necessary. Exemption needed.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neutron-gamma technology</td>
<td>Moisture can be measured in concrete up to 90 mm. depth</td>
<td>Not yet in use for concrete. Specialised personal needed. Safety measures must be taken.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(semi) Mechanic methods</th>
<th>Mechanical crack meters</th>
<th>Well developed, easy to do, cheap, fast</th>
<th>Local measurement.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Invar reference bar</td>
<td>Easy to do, cheap.</td>
<td>Possibly need scaffolding. Measurements must always be made by one person to be reproducible. Not precise when measured in difficult position.</td>
<td></td>
</tr>
</tbody>
</table>

| Other | Wooden dowels | Cheap, easy to do. Well developed. | Relative humidities restricted to 85-100%. Each individual probe must be calibrated. Relation humidity wood and that of the concrete is not univoquive |

Table 2: Advantages and disadvantages per monitoring method (mainly derived from [1] and [2])
Potential measurement has been in use for years for detecting corrosion in the reinforcement of concrete constructions. Particularly in the event of chloride damage, corrosion maps prove to be an indispensable aid. Recent developments in measurement technology allow rapid and large-scale production of maps illustrating the distribution of corrosion throughout the entire construction.
As an example, this presentation provides information on a method used to tackle the very widespread problem (in the Netherlands) of corrosion in flooring elements due to the chloride content of the concrete. The combination of potential measurements and a new system of galvanic protection of the elements is proving to be highly efficient and cost-effective.

Damaged Concrete (General Introduction)
An estimated 80% of all damaged concrete is the result of reinforcement corrosion. Under normal circumstances, the rebars in the concrete are protected by a layer of oxide impenetrable to oxygen, the so-called passivation layer. This protective layer is only formed in a highly alkaline environment. Sound concrete has a pH of 12-13, due primarily to the presence of calcium hydroxide. When the concrete dries, carbonic acid and oxygen can penetrate through the pores to the depth of the rebars. Carbonic acid and calcium hydroxide react with each other to form calcium carbonate, resulting in a lower alkaline level in the environment. The passivation layer is lost through this process and the metal can start rusting under the influence of the oxygen and water present. This process is called "carbonation". The volume of the rust product is 8 to 10 times greater than that of the original steel. The pressure built up in this manner causes the concrete to crack and the cover to spall.

Even when considerable damaged concrete is visible, in most cases there is sufficient rod diameter left to ensure the cohesion of the construction. This corrosion process gives a clear visible warning. In the event of insufficient coverage or poor concrete quality, substantial damage may arise within less than 10 years.

Figure 1: Carbonation damage recognisable from the high volume of rust, loose pieces of concrete and the brown colour of the rust product
Moreover, the chloride present in the concrete may break through the passivation layer and cause corrosion. In this event, a rust product is formed with a volume similar to that of the original metal. By the time the damage becomes visible on the surface, the rebars are often already considerably corroded. This pitting may endanger the safety of the construction although hardly any visible warnings have been observed.

Chloride can enter concrete by means such as the sea wind in coastal areas or the de-icing salts scattered on roads and bridges. In many cases of damage, the chloride was added to the concrete mix. Until the 80s, calcium chloride used to be added into the concrete mix as an accelerant for setting of the concrete. Concrete elements with added chloride have been used in thousands of buildings. A current problem in our country is posed by the so-called “Kwaaitaal floors” constructed with elements containing large quantities of accelerator. The relatively warm and humid environment in the crawl space under the basement floors provides ideal circumstances for corrosion to arise. A recent inventory carried out by the government reveals this problem to be present in at least (and probably even more than) 110,000 homes.

Figure 2: Rust bleeding is an indication of chloride damage
Figure 3: Here the metal has disappeared; the rust product is nearly black in colour.
Figure 4: Dangerous pitting in reinforcement elements essential to the construction.
Figure 5: ‘Kwaaitaal’ floors: the reinforcement in the underside of the ribs is entirely corroded, which may cause floors to collapse.
Potential measurements

Particularly in the event of chloride damage, there is little or no warning via visually observable damage. Thus, other methods are required to investigate the corrosion process. Moreover, measuring chloride concentrations via the analysis of concrete samples does not provide good insight into the problem due to the irregular distribution of the chloride mixed into the concrete.

Corrosion of the reinforcement bars creates variance in potential between spots on the steel surface that sacrifice themselves - the pits in the anodic part - and the spots that are actually protected - the cathodic part. Parts of the rebars look sound while every ten or twenty centimetre there are spots that have been seriously affected by pitting.

The potentials in these electrochemical circuits can be measured on the surface of the concrete with a half-cell, usually a copper electrode in a copper sulphate solution (CSE), or silver in silver chloride, is used along with a sponge for the contact. With a good quality multimeter whose other pole is grounded to the reinforcement, the potential in respect of the reinforcement can be read out. In the event of corrosion caused by chloride, variances in potential ranging from -200 to -500mV are found in respect of CSE. In the event of carbonation, these values are in the range of -100 to -250mV.

To evaluate the damage, it is important to have an understanding of the distribution of the anodic and cathodic parts in the construction. For this purpose, the measurement will have to be effected in a grid, preferably over the entire surface. To do this quickly and efficiently, our company uses the “Bloodhound”. This automated measuring equipment measures also the resistance between steel and electrode. The data are stored as computer tables and later turned into maps by means of special software. This software analyses distributions and gradients and calculates the chance of active corrosion for all spots. This is represented in coloured maps. Green and blue zones indicate the cathodic part of the construction and red and yellow indicate active corrosion, the anodic spots. In this manner, hundreds of square metres can be mapped within a few hours. The maps can be viewed and printed on location immediately by means of a laptop PC.
As is the case with all non-destructive test methods, the interpretation must be calibrated. At a number of spots, the results are compared with the actual situation. It may be assumed that different coloured zones in the corrosion maps correspond to similar situations in the actual concrete. Based on these maps, the bad spots can be marked on the surface, allowing targeted repairs to be carried out. There is no longer any need to cut into the concrete more than really necessary. And, importantly, all bad spots are found, even those where no damaged concrete is as yet visible.

For traditional chloride damage repair, these maps provide indispensable information to ensure durable repair of the construction. Potential measurements are also frequently used for the evaluation of safety aspects of chloride damage in concrete constructions, such as finding pit corrosion in the supporting rebars in beams and consoles.

Last year, dozens of objects were investigated in this manner. Using the Bloodhound equipment, it proved to be possible to carry out measurements straight through the asphalt layer on many bridges. The measured data turned out to correspond entirely with the actual corrosion damage visible after removal of the asphalt. With bridges in particular, the advantages of this measuring procedure in planning repair work are obvious.

Due to the relatively low costs involved and the detailed picture which these potential maps produce, this method proves to be very suitable for "monitoring" existing constructions, especially when chloride is present in the concrete. By carrying out the same measurements periodically, a clear picture of developments in the corrosion process will unfold. When "monitoring" is required, the first thing considered is usually to insert electrodes in the construction. In addition to cost effectiveness, the advantage of making periodic potential maps is that a total view is provided.

This measurement method is also successfully applied to map problems with pre-stressed cables. In one of the larger pre-stressed concrete constructions, a bridge of 1000m in length, many cables proved to be insufficiently injected. Joints and anchors in the road deck show leakages allowing moisture transport in the cables. Pit corrosion on the cables has been detected in various locations. The risk of breakage is unacceptably great. The inner walls in which the cables are located have been mapped with a 30x30 cm grid. The cables through which moisture is being
transported are clearly distinguishable. On the maps with the resistance measurements, the moisture spots in the surrounding concrete are clearly visible. The potential maps show corrosion of the normal reinforcement around the cables.

Even for simple investigations into concrete chloride content, these maps are actually indispensable. How else would it be possible to determine whether samples are taken from a bad or good zone?

Example: method for repairing damaged concrete floors (repair and protection of chloride-contaminated flooring elements)

To tackle the massive problem of chloride-contaminated flooring elements in our country, HTC-EcoReMain has developed a new repair method which can achieve substantial cost savings compared to the usual repair methods. Potential measurements form an essential part of this approach.

Photo-12: Prestressed cables through which moisture is being transferred visible in potential and resistance maps.

Concrete

81
The corrosion of the reinforcement in the elements is stopped by means of cathodic protection. Each element is fitted with a galvanic anode which is sacrificed in favour of the reinforcement. Zinc or Zinc Aluminium alloy is applied to the ribs. Electrical contact with the reinforcement is required to allow the protective current to flow. With heavily damaged elements, the anode material is arc-sprayed straight onto the ribs. In the event that the damage is still limited, a 250( strip of zinc is glued to the ribs with a conductive hydrogel. These extremely cost-effective techniques provide sufficient protection from corrosion for 15 years or longer and are carried out under insured warranty.

In the production process, chloride was mixed into approximately half of the elements, which are now randomly dispersed among residential houses. Potential measurements allow detection of the bad elements, even if damage is not visible.

With this combination of applying galvanic anodes to each element and selecting bad elements by means of potential measurements, substantial cost savings can be achieved. The cost of this approach will be about half the cost of conventional repair methods.

The commercial and social interests involved can be inferred from the fact that 110,000 or more residential houses still have to be repaired and protected.

**Summary**

Potential measurement in itself is not new, but as it is possible only now to produce corrosion maps cost-effectively and on a large scale, the use of these methods proves to be a most attractive and indispensable aid wherever chloride is involved.

* Localisation of corrosion problems allows targeted treatment, resulting in less surface reconstruction and providing more certainty as to durability.

* Periodic monitoring provides insight into the progress of the corrosion over the years, making it possible to determine the right moment to take action.
* Construction safety can be guaranteed by inventorying pit corrosion in essential and vulnerable reinforcement elements.

* Moreover, variance in moisture and salt content in the concrete can be measured; in this manner, water transport in pre-stressed cables has been mapped.

* Potential measurements enable substantial cost-effectiveness to be achieved, as shown by the example of the ‘Kwaaitaal’ floors. Potential measurements make targeted application of galvanic anodes possible, thus reducing costs by half in comparison with traditional methods.

Information:
These potential maps are produced by (among others):
ConCorThe BV: (31) 76 5220978
Adviesburo BEJAN: (31) 591 362268

The systems for galvanic protection of flooring elements are implemented by:
HTC-EcoReMain BV: (31) 76 5651881

Photo-16: Maps are made for each house indicating the elements to be treated.
REHABILITATION OF PREFABRICATED CONCRETE FLOOR SLABS CONTAMINATED WITH CHORIDES, BY MEANS OF EXTERNAL POST-TENSIONING CABLES

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Introduction

In the sixties and seventies, the Dutch building industry was working at an extremely high production level. In those years, there was a great need for new houses and all kinds of new public buildings like schools, sport-centres, shopping centres, and hospitals.

In order to supply this need from the market high-speed production techniques were developed and many parts of the buildings were prefabricated, so the Dutch concrete industry produced also many prefabricated elements, like precast concrete floor slabs.

To supply this high demand several concrete floor producers used calcium chloride in their process as an accelerator for the hardening process. With such a measure, the daily production capacity could be doubled. In those years the use of such accelerating products was widespread and not restricted. It was in 1974 that the Dutch building regulations first limited the use of calcium chlorides in reinforced concrete.

Due to the high groundwater level, as well as the clayey and peaty composition of the Dutch soil many buildings in the Netherlands are founded on piles supporting reinforced concrete foundation beams. Prefabricated floor slabs are laid directly on these beams and covered with a concrete overlay. Under these floors an enclosed space (crawl space) remains, which is very often used to lead cables, sewer-pipes and other services.

In this crawl space, with an average height of 40 to 60 cm, in most cases the relative humidity is high (above 90 %) owing to the high ground water level. Such a high humidity, however, is the main condition for corrosion processes to take place.
Occurrence of the problem

Ten to 15 years later many of the houses and other buildings that were built in this period later showed the effects of severe corrosion of the reinforcement, subsequently followed by cracking and spalling of the concrete cover. Many investigations repeatedly pointed to chloride induced corrosion of the reinforcement as the main cause for this problem. A well-known example of this problem in the Netherlands is caused by the so-called “Kwaaitaal” floor elements.

In these a combination of the high chloride content (0.5 - 2.0 % relative to the cement mass), a high relative humidity, often a poor concrete compaction around the main reinforcement bars, and the use of lightweight aggregates, severe corrosion appeared. In many cases, the main reinforcement was completely affected by pitting-corrosion. The bearing capacity of these elements is then decreased so far that their safety can no longer be guaranteed.

About 3 years ago this became a national problem as many individual house-owners were confronted with the fact that all legal terms for claiming this damage had been exceeded. At that moment, the
national government initiated a research programme in this field. An inventory of this problem in the private market was made in order to investigate the need for governmental support for the stricken private owners.

**Structural concept for repair**

It has been shown that traditional concrete repair methods like manual repairs or shotcrete, sometimes combined with measures to decrease the relative humidity in the crawl space, and the construction of wooden or steel supporting beams under the damaged floor elements do not give the required result or durability. Sometimes the corrosion process was still going on at a significant level, or settlement of the supporting structure led to extreme bending of the floors, resulting in cracks in the walls above, and loosening of tile floorings and other elements.

Fifteen years ago BIM had already developed a method for the structural repair of these kinds of floors. It was based on the principle that the existing corrosion process would be permitted to go on forever, while the structural function of the reinforcement was completely taken over by external high-quality tensile steel cables.

![Figure 4: The BIM post-tensioning cable system.](image)

This structural concept was based on the principle that by fixing the cables underneath the floor in a parabolic way, upward forces would be created as an external load to the floor. A well-calculated cable force and specified dimensions of special supporting blocks make it possible to influence the internal stress in the concrete elements in such a way that the security against collapse is provided and no cracking will be possible.

**Theoretical model**

The design of this external post-tensioning cable system is based upon two design limits:

1. At the moment when the cables are tensioned, their tensile strength is maximal. The upward load is then also maximal. When there is no external load on top of the floor (the floor is carrying only its own weight), in order to avoid any cracking the tensile strength in the top of the floor must be lower than the allowable tensile strength. Of course, the total upward load, initiated by the cable strength, must always be lower than the total weight of the floor itself.
2. As a function of time, the cable force slowly decreases owing to relaxation of the steel, elastic deformation of the floor, creep and shrinkage of the concrete and supporting materials and other factors. In the long term, when the external load on top of the floor is maximal, the security against collapse must stay within legal limits (safety factor at least 1.7).

In most cases these design limits result in a cable force of 70 to 100 kN per cable, while the distance between two supporting blocks will be about 80 to 120 cm. The height of the supporting blocks normally varies from 35 to 120 mm, but it can be adjusted if necessary.

**Practical aspects**

The materials to be used for this system are high quality steel anchor plates and wedges, and a 7-string high tensile steel cable, Ø12.9 mm FeP 1860. The cables are packed in grease into a 25 mm wide polyethylene tube.

In order to fix the cables, it is necessary to anchor them at both sides. Usually the cable has one blind anchor at one side of the building, while at the other side the cable is tensioned and fixed.

Figure 5: Components of the tensioning system.

In order to avoid doing any work inside the building (above floor level), several standard solutions were developed for the fixing of the cables underneath the floor or in the brickwork facade of the building.
To lead the cables under the floor, an additional polyethylene tube is pulled into position first. The space between the two tubes is filled with a cement-grout mortar as soon as the cables and supporting blocks are in the right positions. The alkalinity of this mortar forms an extra protection against corrosion in case the tubes should be damaged.

Because of the tensioning of the cables, a compressive force is introduced in the longitudinal direction of the elements. Although this compressive force is relatively low (maximum 4 to 5 MPa) it could lead to significant deformation of this the structure. This can be the case when the concrete that fills the gap between the elements is of very poor quality. It is then necessary to strengthen this poor quality concrete by means of grout injection before tensioning.

A big advantage of this repair method is that all activities take place inside the crawl space or outside the building. The floor can be used as usual, even during the execution of the work. Existing pipes and cables can normally stay where they are, while it is always possible to direct the cables around them.

**Quality control**

All materials to be used are produced and mounted under a strict quality control regime, checked by independent external auditors. Cables and auxiliary materials are also certified products. During the core drilling through the concrete that fills the gap between the elements, the quality of this poured in situ concrete is visually inspected. If this quality is too bad, this concrete will be strengthened by grout injection.

During the tensioning of each cable the elongation is monitored in order to be alert to unexpected deformations. After the tensioning of the cables, the theoretical elongation is compared with the measured one before cutting of the cable is permitted. Problems that have appeared during a 15-year period of experience have all been evaluated and have led, if necessary, to adjustments in the structural concept or in the method of quality control.
It is not only "Kwaaitaal" floor elements that have been rehabilitated by using this method. Other types of chloride contaminated floor systems in the Netherlands, like "Monoliet" and "Manta" floors have also been treated, while several projects were also executed where reinforced brickwork floor elements were damaged by reinforcement corrosion. So far in the Netherlands, more than 250,000 m² of ground level floors have been repaired in houses, industrial buildings, hospitals, institutes for elderly care, sport-centres, and schools.

Another application of the system is the improvement of the bearing capacity of industrial floors. As long as the limitations of design are not exceeded, the post tensioning cable system can upgrade the allowable floor load to 10 kN per m². In this way several industrial projects were carried out in the Middle East as well on a smaller scale in some office buildings in the Netherlands.

Nevertheless, the great experience with this method has shown it to be less suitable for individual private house-owners. As the total costs of the system are mainly provided by the extent of anchoring blocks or beams, the relatively small surface to be repaired makes this method for these projects not cost-effective enough. For larger scale projects like schools, hospitals, sport- or shopping centres and even blocks of houses, the post tensioning cable system proved the most interesting structural repair method.
STRENGTHENING OF REINFORCED CONCRETE BY FIBER REINFORCED PLASTICS

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Abstract
The application of fiber reinforced plastic materials (FRP) to renewal of civil infrastructure and to construction of new reinforced concrete structures has increased significantly in the recent years. The main research interests that are driven by the application of new construction materials is oriented to durability of FRPs in combination with traditional materials and to studies on behavioral mechanisms of structures where FRPs are applied. Some aspects of advantages and problems related to use of FRPs are briefly discussed based on literature survey. An example of experimentally supported study on flexural behavior of reinforced concrete beams strengthened by steel and FRP strips is also presented. Two types of strengthened beams of different span to depth aspect ratio have been tested. The different failure mechanisms have been observed in cases of strengthening by steel or by carbon fibre reinforced plastic stripes. The results of experimental research help in explanation of differences between the behaviour of structures strengthened on traditional way by steel stripes and on contemporary way by CFRP ones.

Keywords
Fiber reinforced plastic, retrofitting, durability, reinforced concrete beams, strengthening, steel plates, CFRP plates

Introduction
The development of repair and strengthening technique in the last decade has been very much influenced by availability of composite materials on market. The traditional techniques of external strengthening of reinforced concrete structures based on application of steel jackets and plates have been recently modified and adjusted to the properties of fiber reinforced plastic (FRP) materials. That is the first phase of development of the new technology that is started from the knowledge obtained by the use of previously developed techniques suitable for another type of material. The new techniques entered in the construction practice about ten years ago in the most developed parts of the world namely in Japan, USA and Canada. The FRP is applying in Europe with delay of several years with advantage of learning from problems that have been identified during the use of strengthened structures in Japan and USA. The first applications were oriented to the bridge strengthening and later on also to the high rise and other building constructions. The main reason is because the studies from 1992 clearly show that a large number of US bridge inventory are reaching the end of their initial design life (Karbhari, V.M., 1996). The application of FRP is not limited only on strengthening of reinforced concrete structures but also on the strengthening of masonry and even timber structures. The main reason of strengthening is to increase the earthquake resistance of structures. However, there is also a wide range of applications where the structural systems are strengthened to increase their load bearing capacity due to increase of traffic or dead load action.

The main problems are arising from uncertainties related to the durability of FRP components and their response on environmental impacts. The another potential source of problems is weakness in understanding of behavioral mechanisms of strengthened structures. Both the design and the construction practice can be influenced by lack of knowledge and experiences even when professionals are experienced in application of steel jackets and plates. Learning from the laboratory or on-site tests is helpful in
solving problems that are arising from misunderstanding of behavioral mechanisms. The long term response of FRP material attached to reinforced concrete or another structural material can not be satisfactory reproduced in short time. However, there are indications that the durability problems are related to the decay of certain types of fibers due to chemical attack in combination with decay of polymer matrices due to temperature changing and moisture absorption effects. Even more problems can arise from decay and corrosion of materials in adhering area or changes of adhesive that influences its cohesive strength. However, that is still very simplified observation of problem that influences the effectiveness of strengthening techniques.

The confinement effect of FRP that is wrapped around the concrete core is completely different that the effect of steel stirrups. The difference in load bearing mechanism can also be observed in comparison of flexural behavior of beams and slabs strengthened by steel or FRP plates. The main objective of this paper is to discuss on these differences using according to experiences gained from experimental research.

Figure 1: The examples of strengthening of beam-column connection in Thousand Oaks, California utilizing the Tyfo SEH 51 System (left) and increasing of shear and flexural strength of beams in Singapore by the same Tyfo System (above). The Tyfo System is developed by The Fyfe Co., LLC, Montreal, Quebec, Canada.

Advantages and problems related to fiber reinforced composites
Fiber-reinforced polymer-matrix composite materials are usually called fiber reinforced plastics (FRP). They have a number of advantages when compared to traditional construction materials such as steel
and concrete. FRP offer excellent corrosion resistance to environmental agents as well as the advantages of high stiffness-to-weight ratios when compared to conventional construction materials. Such ratios for carbon-fiber-reinforced composites (CFRP) are 10 to 15 higher than those of steel. Other advantages of FRP include low temperature expansion, good fatigue performance and damage tolerance, non-magnetic properties, ease of transportation and handling, low energy consumption during fabrication of raw material and structure and most of all the tailorbility. Glass (GFRP), aramid (AFRP) or carbon (CFRP) fibers are commonly used reinforcement in FRPs. The matrices are usually thermosets as polyester, phenolic, polyamide and epoxy. The main reason for limited use of these materials has been the high price if compared on the basis of unit price. However, this comparison is not to be taken into account without other components of structural costs. The lightweight of FRP reduces transportation expenses and enables prefabrication of components that reduce time at job site. The high corrosion resistance makes the life cycle costs lower than they are with conventional materials. However, the cost is expected to be driven down in future by increasing demands.

Currently, FRP is applied as external and internal reinforcement, for environmental protection of structures and for construction of new structures and structural elements. External application of bonded FRP plates is used for strengthening of concrete, steel and wood beams, concrete slabs and masonry walls. FRP tubes are used for external strengthening of concrete and timber columns. External unbonded reinforcement is used in form of prestressing tendons and cables. Internal reinforcement is used in forms of internal bonded FRP rods or internal unbonded prestressing tendons. New structures constructed by FRP are mainly bridges, bridge decks and stair towers as well as structural elements as hybrid beams and columns and FRP connections. Beyond cost issues the most significant technical obstacle preventing the extensive use of FRP materials is lack of long term durability and performance data comparable to the data available for more traditional construction materials. Decaying influences on embedded and external FRP and bonding material are generating within the concrete and attacking from surrounding as it has been summarized by Karbhari (Karbhari, 1996) and is shown in figure 2.

![Figure 2: The influences on long term durability and performance of FRP according to (Karbhari, 1996)](image_url)

In the past decades many studies in the area of creep, stress corrosion, fatigue and environmental fatigue, chemical and physical aging and natural weathering were completed. However few of them were aimed at applications for the construction industry. Recently the interest for studying the behavior of FRP materials to be applied in strengthening of existing or constructing of new infrastructure increased tremendously. Most of studies are concerned with short-term behavior where a material or structure is
loaded for a short period of time to learn about behavioral mechanisms of composed structures. Even the very limited number of studies has been carried out in the area of long-term behavior Liao et al (Liao et al, 1998) completed the detailed overview of existing research reported in 402 references published since 1949. From diagram in Figure 3 is can evidently seen the growing interest in durability issues of FRP. The intensive research had been carried out in early eighties and early nineties with culmination in publishing on 1983 and 1995 when 59 papers and reports in English had been published. The literature overview does not cover publications in other languages. It is important remark because great advance in this area is achieved in Japan.

Figure 3: The intensity of research in durability expressed in term of publishing of research results (Liao et al, 1998)

The review covers a large area of different effects and phenomena related to long-term behavior of FRP, as they are summarized below.

Diffusion of aqueous fluid into composite materials may result in reversible and irreversible changes of constituents (fiber, matrix and fiber/matrix interphase region). The effects of fluids on mechanical properties can be identified as fluid sorption process, fluid damage in FRPs, effects of fluids on strength, modulus and fracture toughness of FRP. The long-term action of externally applied load causes the irreversible material deformations due to the phenomenon known as creep. To describe this phenomenon and its effect on FRP several theoretical models have been developed. The research done in this area based on applications of Findley's power law model, Boltzman-Voltera model and some other models that are oriented on creep of laminated composites.

The studies of creep of GFRP and CFRP were mostly oriented on action of tensile loading. Some research have been done in field of effect of moisture and temperature on creep of FRP, on effect of physical aging and ultraviolet radiation on creep. Recently, the studies of creep of entire FRP structures are coming into research interest. Since the acceleration test methods are known from testing of other types of materials they are used also for testing of FRP materials. The most widely accepted accelerated test method is the time-temperature-stress superposition (TTSSP). The method is based on the assumption that the effects of changing temperature and/or stress on the time dependent behavior of a material are equivalent to a shift in the actual time scale for the measurement. The especially important area of research of durability is stress rupture and stress corrosion of FRP because this phenomenon can be a
The fatigue and environmental fatigue are two of very important influences on long-term durability of FRPs. The term fatigue usually implies cyclic fatigue when a periodic mechanical load is imposed. Acting of the environmental influences during cyclic loading causes the environmental fatigue effects. The area of research interest in this area are studying of damage mechanisms, the effect of constituents on fatigue performance of FRP and the effect of connectors in connected components. The fatigue data are mostly presented in form of SN curves. These are relations between amplitude of the applied stress (S) and the number of cycles to failure (N). The increasing of frequency of cyclic load has been found to have a modest influence on lowering of fatigue life of FRP. There are only a few studies on long-term fatigue of composite structures. Some more research has been done to study the environmental fatigue of FRP but the problem is demanding more intensive and extensive research in this field. The fatigue tests are the enabling development of life prediction methods. It has been found that the fatigue performance can be extended with combination of two or more types of reinforcing fibers that in this way create hybrid composite.

The natural weathering of FRP may affect in great extend the durability of polymer matrix and even the fibers. Very important aspect of durability is the durability of bond between FRP and concrete surface. It is critical to the overall durability of structure that had been strengthened by bonding of composite or steel plates.

The wider use of fiber reinforced composites in design and construction practice is considerably limited by lack of design guidelines and codes. The first steps to solution of these problems are the specifications and guidelines issued by companies that had developed repair and strengthening systems. These guidelines are based on state-of-the art available from research work at the time of their issuing. More general approach can be traced in the cases where a larger group of researchers and engineers create more general guidelines. The good example are the design guidelines of FRP reinforced structures established in 1993 by a group of Japanese experts (Sonobe et al, 1997). The guidelines are one of the final outputs of the research committee on fiber reinforced plastic reinforced concrete structures organized under Japanese Ministry of Construction's research and development project (1988-92) entitled: Effective use of advanced construction materials.

The new European codes that are related to strengthening and repair of buildings in seismic regions (Eurocode 8, ENV 1998-1-4, 1996) do not yet include the FRP strengthening. However there are a number of paragraphs in this code dealing with strengthening of reinforced concrete structures by applying of steel plates and jackets. Knowing the differences between the behavioral mechanisms of structures strengthened by steel and FRP the ENV 1998 1-4 can, to certain extend, serve as guidance for design of FRP strengthening of reinforced concrete structures. Therefore the comparative testing of structural elements strengthened by steel and by FRP is needed to learn about the differences in behavior of structures strengthened by steel and FRP.

**Strengthening of beams and slabs**

The efficient upgrade of flexural strength of reinforced concrete structures by externally bonded steel plates has been introduced to Slovenian practice on 1979 when insufficiently reinforced newly built reinforced concrete bridge deck was strengthened (Bostjancic et al, 1982; Znic & Tercelj, 1982). After that more than 100 reinforced structures have been strengthened by steel plate bonding in Slovenia without any deficiency observed by later inspection. Recently, the new generation of plates made of
carbon fiber reinforced plastic (CFRP) is introduced in practice. Although it is well known that carbon fiber reinforced plastic plates can replace steel ones there are some differences that should be taken into account in strengthening design. Therefore, the research project on comparison of steel and CFRP plate bonding has been started recently. There are many papers reporting the experiences on flexural behavior obtained by experimental and analytical research that give a good insight in flexural behavior of plate bonded structural elements. Some of them are listed in references of this paper. The intention of herein-presented research was to verify the ability of Slovenian contractors to introduce the CFRP plates widely in practice and to learn about the differences between the behavior of steel and FRP strengthened structural elements.

**Description of tests**

Two different flexural elements were designed to represent a beam and a strip of the slab. The second one is considered as a flat beam. The dimensions of the beam were 200 mm wide by 300 mm deep by 3250 mm long (figure 4). The flat beams were of the same length as beams. Their cross-sectional dimensions were 800 mm wide and 120 mm in depth. The specimens were designed according to ENV 1992 (Eurocode 2). The steel reinforcement ratio $A_s/b_d$ was chosen in amount to allow the external reinforcement to be added without over-reinforcing both types of specimens. It avoided premature brittle failure of the concrete in compression. The steel reinforcement ratio of beams was 0.56% and of flat beams 0.40%. The internal flexural reinforcement consisted of ribbed steel bars with 400 MPa minimum yield stress. Shear reinforcement of beams consisted of ribbed steel bar 6 mm in diameter, placed at 100 mm distance in the 1/3 of length at both sides and with 150 mm distance in the 1/3 of length in the middle of the beam.

Altogether fourteen specimens were constructed, seven of each type. The concrete used for construction of specimens had compressive strength of 25 MPa. Three of each type was strengthened by steel plates and three of them were strengthened by CFRP plates. The mild steel plates ($f_y/f_u = 240/360$ MPa, $E = 210$ GPa) were 4 mm thick and 50 mm wide. The CFRP plates ($f_u = 2400$ MPa, $E = 150$ GPa) were 1.2 mm thick and 50 mm wide. The epoxy glue was 2 mm thick in all cases. Each beam was strengthened by one plate (Figure 4). Two plates (Figure 4) strengthened each flat beam. One beam and one flat beam remained were not strengthened and served as reference specimens. Plates were attached on specimens by epoxy resin on the same way as it is case in practice. Since the steel plates are usually nailed every 500 mm on building site to enable fast and efficient pressure on glued surface the same was done in the case of steel plated specimens. Specimens were tested by displacement controlled hydraulic actuator INSTRON 250 kN. The external load acted on 1/3 of the span as it is shown in Figure 4. Displacements were measured by LVDT-s at mid-span of specimens. The distribution of strains over the depth of beams

![Figure 4: Geometry and instrumentation of test specimens](image-url)
was measured in mid-span and 1/6 of span by dilatometers (D1 through D8). Strains on lower concrete surface and plate surfaces were measured at mid-span and at the ends of plates by strain gauges (S1 through S9). The layout of measuring points on beams and flat beams are shown in figure 4. The legends in diagrams refer to measuring points presented in figure 4.

**Flexural behavior of specimens**

The comparison of the deflection curves of strengthened and reference specimens presented in Figure 5 gives insight in level of flexural strength upgrade achieved by plates. Several differences between effect of steel and CFRP plating can be observed from diagrams. Using the steel plates higher increase of stiffness and ductility can be achieved than in case of CFRP plating. It was also observed that test results of beams are less scattered, what can be the influence of greater robustness of beam cross-section. The higher ductility of steel plated specimens can be partly influenced by dowel effect of steel nails used for fastening of steel plates during fitting them on the beam surfaces. Diagrams clearly shows the appearance of first cracks and therefore changing of flexural stiffness.

Flexural strength of beams increased in average for 53% because of added steel plate, while CFRP plates caused average increase of strength for 35%. The reference beam failed due to action of external force of 86.5 kN. In the case of flat beams steel plates increased their strength in average for 97.6%. CFRP plates increased their strength for 72.5%. The reference flat beam failed at external force of 36.5 kN. Reduction of flexural stiffness due to development of flexural cracks occurred at approximately same external loading in the case of unreinforced and plated specimens. Plates delayed development of cracks what resulted in different post-cracking stiffness. Inner reinforcement started to yield at the same magnitude of deflections of all beams. Steel plates were more effective then CFRP ones what resulted in higher stiffness of steel plated specimens.

![Graph comparing deflection curves](image)

*Figure 5 Comparison of deflection curves of beams (a) and flat beam (b)*
Figure 6 Comparison of strain distribution over the depth of reference beam, steel plated beam and CFRP plated beam

Distribution of strains over the depth of beams was measured at mid-span and 1/6 of span of specimens to obtain information on validity of Bernoulli’s hypothesis. It was confirmed using the set of results presented in figure 6 both for specimens strengthened by steel and by CFRP plates. Analysis of strain development in plates and on concrete surface helps in understanding of different failure modes that developed in tested specimens. Strains developed on the surfaces of plates and on the lower concrete surface are presented in figure 7 (a) and (b). It is clearly seen the function of plates in delay of crack development. In figure 7 (c) is schematically shown that the yield limit was achieved in steel plates while CFRP plates remained in elastic range. While for steel plates the stiffness had fallen to almost zero level after reaching yield point of steel and thus peeling stresses within the concrete were not further increased. The stiffness of the CFRP plates remained unchanged and thus provokes sudden delamination at the critical load.

Figure 7: Comparison of strain curves of beam (a) and flat beam (b). Strains were measured on plates and on concrete on lower surface of beam.

Concrete
Failure modes

Three basically different failure modes were observed. All of them are known from the experiences of other researchers (Ritchie et al, 1991) and (La Tegola et al, 1998). In the case of all three steel plated beams the failure was caused by end-of-plate shear peeling (figure 8 (a)). It developed due to exceeding strength of the concrete below the plate. Nails that were used for plate bonding limited the area of peeling development but did not affect the magnitude of failure forces. Failure of CFRP plated beams developed due to debonding or delamination of plates below the concentrated force acting on the upper surface of beam (figure 8 (b)).

Figure 8: The failure mechanisms that developed in the cases of: beams strengthened by steel (a) and CFRP (b) plates and flat beams strengthened by steel (c) and CFRP (d) plates.

Steel plated flat beams failed due to debonding or due to delamination of plates in relatively limited mid-span area. Development of failure (figure 8 (c)) was influenced by yielding of steel plates (figure 7 (b) and (c)). In the case of CFRP failure was caused by delamination or/and debonding of plates (figure 8 (d)). The development of failure in the cases of CFRP plated flat beams and flat beams was sudden in contrary to more ductile development of failure in cases of steel plated flat beams.

Figure 9: Comparison of the analytically obtained peeling (a) and shear (b) stresses at the end of steel and CFRP plates bonded to beams and flat beams.
Development of strains at the end-of-plate region can be predicted analytically by the expressions that were proposed by Täljsten (Täljsten, 1997). Calculated distribution of peeling tensile stresses and shear stresses at the end-of-plates are presented in figure 9. Stresses were calculated taking into account the geometry of specimens and plates and mechanical properties of materials. The acting force that was taken into account is in every observed case equal to failure force. It is clearly seen that the combination of peeling and shear stresses gives the highest value in the case of steel plated beams where failure actually occurred due to end-of-plate shear peeling. It is interesting that steel plated flat beams would probably failed on the same way if the failure would not developed by debonding due to mid-span yielding of plates. From figure 9 it can also be concluded that CFRP plated beams and flat beams are not sensitive on end-of-plate failure. In their cases far more critical is development of failure due to debonding or delamination in the region of extensive flexural deformations of structure.

Conclusions
The recent advance in application of fiber reinforced plastic in construction practice has increased research interest in this topic. The main directions of development are oriented to strengthening of existing structures and in construction of new reinforced concrete structures. Composite materials have proven as material with a number of advantages due to their composition and resistance on high loading and corrosion. However, there are still many unsolved problems related to long-term behavior and durability, detailing the structural elements, design of new type of structural systems, lack of appropriate codes and training of contractors. The new material seems to be promising and helpful for solving many problems related to structural lifetime extension. The research efforts started about two decades ago should be continued on the basis of available experiences. Even European research and building practice is adopting the new material and techniques with a delay comparing to USA, Canada and Japan the positive and negative experiences may help in developing of successful methods to be applied in practice.

References


Possibilities and problems for post-installed anchors used for changing concrete structures

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Summary
From sustainability point of view, extending the lifetime of a structure can be regarded as the best opportunity there is, when its no longer suitable for the purpose it was originally built for. For many dwellings built in an industrialised way during the 1960’s, the changed demands for residential buildings ask for important adaptations in order to make them suitable for future use [1]. Furthermore, also warehouses, office buildings and even churches are sometimes transformed into residential buildings. In the Netherlands for the load-bearing structure of residential buildings, a concrete framework is mostly used. For the adaptations of the structures it will be necessary also to alter the load-bearing structure or to connect new parts to it [1]. A possibility to do this is to partly demolish the concrete, connect the original reinforcement with new reinforcement and to pour a new part of concrete against it. As alternative to this, it might be more convenient to use prefabricated elements connected to the concrete frame by means of post-installed anchors. The knowledge about and reliability of post-installed anchors has increased considerably during the last decade. In this paper recent developments in this fastening technology will be discussed and some considerations will be given as far as the possibilities for post-installed anchors in adapting the concrete load-bearing structure, is concerned.

Introduction
Sustainability is an issue of great interest nowadays. More and more it is realised in the building industry, as it is also in other industries, that economics should not be the only governing parameter, but that also the care for our environment must play an important role. There is an increasing consciousness that we should not pass the problems of our actions today to future generations. In that respect, many governments are therefore actively promoting policies aimed at reducing the use of primary resources. As far as buildings are concerned, the best opportunity from a sustainable point of view is to extend their lifetime.

Due to the many demographic changes such as fewer children per family, fewer two-parent families, increased life span and greater financial capacity, the requirements for types of housing have strongly changed. Verhoef [1] discussed this aspect and concluded that it is necessary to adapt houses in a way that the shell of the building is no longer regarded as the boundary within alterations have to be made. Going beyond these traditional boundaries directly means that new structural elements have to be connected to the original structural concrete framework.

Whether it will be possible to add new structural elements to the outside of a concrete framework, depends on the structural capacity of the existing framework. In any case, a design for the intended new situation has to be made and it may result in the conclusion that the intended changes are not possible or only possible after strengthening of the existing structure. When the structural capacity of the existing structure is sufficient for the intended adaptations, the question arises how to connect the new structural elements to the existing framework. In this paper some considerations will be given regarding that
aspect. Special attention will be given to the possibilities and problems for applying post-installed anchors in this respect. A problem of short post-installed anchors for this application might be the relatively small thickness of the elements in which the anchors have to be placed. Reference will be made to an example for adding concrete elements to the concrete framework as it is given by Verhoef [1]. In Section 3 attention will be paid to this aspect. In the next paragraph first attention will be paid to some recent developments with regard to the fastening technology.

Recent developments in fastening technology

General
Concrete is a material that is very suitable to resist compressive stresses, while its resistance to tensile stresses is rather poor. That is the reason why steel (reinforcement or prestressing elements) is required in most concrete structures. As a result of this, many people think that in concrete structures we do not rely on the concrete tensile strength, which is not correct as will be discussed later. For post-installed anchors, like torque-controlled expansion anchors, undercut anchors and bonded anchors, their tensile capacity is fully depended on the tensile properties of the concrete. Because of this fact, still some scepticism as far as the application of such anchors in structural connections is concerned, can be encountered.

With respect to the role of the tensile capacity in concrete structures, the fact that it is a relevant parameter in for instance the shear and punching shear capacity, and bond between steel and concrete, can be mentioned. Furthermore, it is known that it is not only the tensile strength that determines the behaviour of concrete under a tensile loading. After the introduction of the non-linear fracture mechanics for concrete in the late 70’s it was recognised that also the fracture energy, G_t, which is by definition the amount of energy that is required to create one unit area of crack, is an important parameter. However, in most Design Codes in which many empirically based formulas are used, this parameter is not applied. Here it is mentioned because theoretical and experimental investigations showed that the failure load of short anchors depends more on the Young’s Modulus, E, and fracture energy G_t of the concrete than the tensile strength [2]. In the empirical formulas that are being used to determine the tensile capacity of anchors, the influence of these properties E and G_t is represented by the compressive strength f'_c 0.5.

In the 80’s the knowledge about the behaviour of anchors in various circumstances, was still limited. Manufacturers gave values for the average strength and proposed to apply an overall safety factor of about four or five in order to determine the allowable load. Reduction factors were given to account for the influence of adjacent anchors or boundaries of the concrete element. Since then many investigations have been performed. Especially in Germany and in particular in the group of professor Eligehausen from the University of Stuttgart, much research was done. For instance the effect of a crack on the capacity of anchors was investigated and introduced in the design. As far as the international developments in this field are concerned, the activities in CEB (since the merging with FIP in 1998 now fib) and EOTA can be mentioned. A Task Group of CEB first published a state-of-the-art report [2] and than started to work on a design guide. A general part and the parts for post-installed expansion anchors, undercut anchors and cast-in-place headed anchors were published in 1996 [3]. On parts for other types of anchors, like bonded anchors, work is still going on. As far as the determination of the relevant properties of anchors is concerned, much work is done by EOTA, of which something will be explained in the next section.

A comprehensive test programme for post-installed anchors

By EOTA (European Organisation for Technical Approvals) a guideline (ETAG) [4] is produced that should be followed in order to obtain a European Technical Approval (ETA) for metal anchors for use in concrete. Several parts of the ETAG are ready. These are the parts for:
torque-controlled expansion anchors
* undercut anchors
* deformation-controlled expansion anchors

Work in this area for other types of anchors or anchors for use in other materials, like masonry, is going on. The part for bonded anchors is almost ready by now. In figure 1 three types of post-installed anchors are shown.

Figure 1: Schematic representation of three types of post-installed anchors.

According to the ETAG [4] the manufacturer can choose between various options. In general it means that if he wants to get as much as possible out of it, the most extensive test programme has to be performed. On the other hand, if he excepts that his anchor can only be used in non-cracked concrete while furthermore only one characteristic strength, irrespective of the concrete grade and loading direction is used, the simplest test programme is applicable.

For details about the various tests that have to be performed the reader is referred to [4]. Here only attention will be drawn to the two types of tests that are distinguished in the ETAG. These are “tests for suitability” and “tests for admissible service conditions”.

Tests for suitability are required for various reasons of which the following are mentioned here:
* Anchors should be not too sensitive to deviations from the manufacturer’s installation specifications including aspects like:
  - cleaning of the drilled hole
  - applied torque moment
  - striking reinforcement during installation
* Anchors should be not too sensitive to variations in the properties of the concrete in which the anchor is installed.
* Anchors shall function properly under sustained loads or loads with varying magnitude (is not fatigue).

In the testing of a particular anchor, variations on the installation procedure required by the manufacturer, which can occur on site, have to be taken into account. Furthermore, tests in low and high strength concrete (characteristic cube strength of respectively 25 and 60 MPa) have to be performed as well as tests with sustained loading and tests with repeated loads. The results of the various tests have to fulfil certain requirements. If these requirements are not met, the characteristic strength of the anchor will be reduced. Furthermore, based on the test results an additional partial safety factor $\gamma_2$ (installation safety...
factor), that has to be applied in the design procedure, is determined. This factor, as well as other information about the anchor, like the characteristic strength for the various failure mechanisms, is given in the ETA (European Technical Approval). Finally it shall be mentioned that gross errors in anchor installation are not covered by the Guideline. Examples of such gross errors are using a drill bit with a wrong diameter (+ 1 mm) or not cleaning the bore hole in the case that it is required by the manufacturer.

Tests for admissible service conditions have to be performed in order to derive performance characteristics of the anchor that are required in the design of a fastening. The number of these tests is limited to only that, necessary to confirm whether the behaviour of the anchor falls within current experience. With respect to current experience it can, for instance, be mentioned that the effective depth of the anchor and the strength of the concrete determine the strength for the failure mechanism “concrete cone failure”, as will be discussed later.

**Design method for fastenings with post-installed anchors**

A design guideline for fastenings with post-installed anchors in concrete is given in [3] and [4]. Except for some minor differences it concerns one and the same design method in both documents. Again, for details about the design method the reader is referred to the original documents. In the following some main lines of the design method are presented.

For the anchors, various possible failure modes have to be taken into account for tension and also for shear. In figure 2 and 3 the failure modes for respectively a tensile and a shear loading are shown. In the figures 2 and 3, as well in following figures, a cast-in-place headed anchor is also shown, but for post-installed expansion and undercut anchors the same failure modes are valid.

For a single anchor loaded in tension or in shear, the characteristic strength for the various failure modes can be determined. For the design of fastenings in practice that information is not enough, because for a number of failure modes neighbouring anchors or edges of the concrete element in which the fastener is placed, reduce the capacity. The main principle that is used in the design method to account for this phenomenon will be explained for the failure mode “concrete cone failure”.

![Failure Modes](image)

**Figure 2**: Schematic representation of the failure modes that have to be taken into account for a tensile loading.
Figure 3: Schematic representation of the failure modes that have to be taken into account for a shear loading.

the effective depth, based on many experiments, was derived [2]. For the characteristic strength of concrete cone failure in the design method, the following relation is used for torque-controlled expansion anchors [3]:

\[ N^o_{Rk,c} = 7.5 \cdot f_{ck}^{0.5} \cdot h_{ef}^{1.5} \quad (1) \]

In which:

- \( N^o_{Rk,c} \) is the characteristic strength for concrete cone failure of an anchor that is not influenced by other anchors or edges of the concrete element;
- \( f_{ck} \) characteristic concrete compressive strength
- \( h \) effective depth (see figure 4)

Figure 4: Definition of the effective depth \( h_{ef} \). It should be remarked that for the bonded anchor the characteristic strength is based on an average shear strength along the embedded anchor instead of a formula comparable to equation 1.
The procedure to account for the influence of other anchors and of edges is as follows. For one anchor a cone in the shape of a pyramid of which the depth is equal to $h_{ef}$ and the base is equal to $3h_{ef} (= s_{cr,N})$ is assumed. This results in an area $A_{c,N}$ on the concrete surface which is equal to $9.h_{ef}^2$. In order to calculate the resistance of a group of anchors or of anchors near a concrete edge, similarly the area $A_{c,N}$ is calculated. Examples for the calculation of $A_{c,N}$ in respectively the case of a group of two anchors and the case of two anchors near an edge, are shown in the figures 5 and 6.

![Diagram](image)

Figure 5: Definition of the area $A_{c,N}$ that is used in the design method for calculation of the resistance of a group of anchors near edges of the concrete elements

![Diagram](image)

Figure 6: Example of the calculation of the area $A_{c,N}$ for a group of two anchors with an effective depth of 100 mm ($A = (1.5h + 100 + 1.5h) * (1.5h + 1.5h)$).
In order to calculate the strength of the group or of an anchor near an edge, the characteristic strength for one anchor (equation 1) is multiplied by a factor $\psi_{AN}$ which is defined as:

$$\psi_{AN} = \frac{A_{CN}}{A_{cN}}$$  \hspace{1cm} (2)

For the examples in the figures 6 and 7 respectively a value of 1.33 and 1.11 is calculated for the factor $\psi_{AN}$. This means that the capacity of the group of two anchors is respectively only 33% and 11% more than the capacity of a single anchor that is placed far from other anchors and far from concrete edges. The previous examples were only given to demonstrate the methodology that is used in the design guide [3,4] for taking group effects and influences of concrete edges into account. It should be remarked that besides the factor $\psi_{AN}$ it may be necessary to apply some more multiplication factors (see [3,4]). Furthermore, it can be mentioned that for the failure mode "concrete edge failure" under shear a similar approach is followed.

**Importance of execution**

For a reliable fastening with anchors the following three aspects play an important role:

1) The properties of the anchors under the various circumstances should be known;
2) A design method should be available that ensures that the design of fastenings with various configurations of anchors results in a safe construction;
3) The execution, which means the placing of the anchors, should be done properly.

For anchors with an ETA and designed according to the available design guides, as discussed before, the requirements with respect to the first two points may be assumed to be fulfilled. With respect to the execution, so far there is not so much organised. The importance of this aspect may be obvious. When gross errors like e.g.:

* a too large bore hole diameter;
* no cleaning of a bore while it is required;
* a wrong torque moment is applied

occur on site due to insufficient quality control, than the end is an unsafe fastening despite a very good anchor and a good design. This is one of the reasons why a Dutch recommendation [5] did not allow to apply a fastening with one anchor. According to that recommendation it was furthermore required that
the fastening could resist the loads applying a load factor of 1.1, while assuming that one anchor had failed. Such a rule is not applicable in [3,4]. In a draft for an update of the mentioned Recommendation [5], this rule is replaced by a sentence in which the designer is reminded at the fact that the structure should have a second possibility to carry the loads when one anchor fails.

Connecting new structural elements to the outside of an existing concrete framework

As a demonstration, Verhoef [1], made a calculation for an extension of a flat. It was assumed that the skeleton of the flat consists of poured concrete floors with a thickness of 120 mm and walls with a thickness of 180 mm. For the extension it was calculated that the shear force between the old concrete and the new poured concrete part is 319 kN, resulting in an average shear stress of 0.35 N/mm². Furthermore it was calculated that due to a bending moment, a tensile force of approximately 125 kN has to be transferred to the existing structure. When using FeB 500 steel reinforcement, applying a material factor of 1.15, this results in a required cross-sectional area of about 290 mm².

According to the traditional way of designing concrete structures the reinforcement of the old and new concrete has to be connected with sufficient overlap length so that the tensile force can be transferred between the old and new reinforcement. In that case, yielding of the steel reinforcement is the criterion and for transferring a tensile load of 125 kN one bar with a diameter of 20 mm would already be enough.

In order to connect the reinforcement with sufficient overlap length, there are in general two possibilities. The first one is to demolish a part of the existing structure. The other one is to drill holes in the existing structure and to apply bonded anchors. Compared to the short bonded anchors (usually having an effective depth of about 10 times the diameter) this implies long bonded anchors (embedment length of about 30 times the diameter or more). In the Netherlands there are no regulations for long bonded anchors in drilled holes. When it is assumed that these anchors behave similarly to reinforcing bars embedded in concrete than the Concrete Code [6] can be used. However, questions that often arise are whether a threaded rod should be regarded as smooth or ribbed reinforcement. According to the Dutch Concrete Code [6] there is a difference of a factor two in the bond strength for respectively ribbed bars and smooth bars. In general the question is what anchorage length has to be applied in case of long bonded anchors, irrespective whether it concerns bonded reinforcement or bonded threaded rods. In fact the possible consequences of variations in the installation procedure, as discussed before for the short anchors, should also be considered. Is it always possible to fill the bore hole completely? For vertical holes it is not so much a problem, but for horizontal holes this is more questionable. Based on the various questions arising from practice there is to the author’s opinion, enough reason to do some research on this aspect.

In the previous discussed solutions for the creation of a connection between the old and new concrete the theory of reinforced concrete is used, which means that yielding of steel in the structure takes place before concrete failure. By that, there is a warning effect. Although a connection with short anchors, based on the fastening technology will probably not be used very quickly in the described application, with some exploratory calculations it will be shown, that anchoring on the head side of walls and floors according to the fastening technology is not easy.

The concrete wall has a width of 180 mm. This means that for an anchor with a depth greater than 60 mm its capacity will be reduced by the edges of the concrete. According to equation (1) the characteristic resistance for an effective depth of 60 mm and an assumed concrete strength class C25 is about 17 kN. Due to the for anchoring relatively thin wall the use of an anchor with a greater effective depth does not
result in a significant increase of the capacity. If, for instance, an anchor with a two times longer effective depth (120 mm) is used, the capacity increases with a factor 2.8 \( (=2^{1.5}) \) according to equation 1, while at the same time a factor 0.5 has to be applied according to equation (2). As a result, the doubling of the effective depth in this case results in an increase of the capacity of only about 40\% (from 17 kN to 25 kN). It can further be remarked that in case more anchors are applied above each other, the distance should be large enough otherwise the increase in capacity is again very limited due to overlapping concrete cones.

The above examples are based on the application of expansion or undercut anchors. Bonded anchors may perform better under the given circumstances because the distance at which these anchors influence each other or are influenced by concrete edges, is smaller. Suppose a bonded anchor is used with a diameter of 16 mm and a characteristic value for the average bond strength of 6 MPa. Then the characteristic strength of an anchor with an effective depth of 60 mm is 18 kN \( (=6\pi d h_{ef}) \). The base of the concrete pyramid \( s_{er,N} \) (see figure 5) for bonded anchors may be taken equal to \( 2h_{ef} \) instead of the \( 3h_{ef} \) for the expansion anchors. As a result, doubling the effective depth from 60 mm to 120 mm now results in an increase of 50\% from 18 kN to 27 kN (first an increase of the capacity by a factor 2 and than a decrease due to the edges by a factor 0.75). As it can be seen, for bonded anchors the effect of overlapping cones is less, but the on the other hand the increase in capacity with increasing effective depth is less.

The above examples illustrate that use of short anchors for connecting new structural elements to the head surface of walls and floors is probably not very effective due to the relatively small thickness. This holds true for a tensile force. Probably, short anchors are more effective for shear forces. Nevertheless, when applying more anchors it may still be possible to anchor the load of 125 kN, although the fact that the lever arm changes should also be taken into account. Before a connection as discussed in this example is used in practice it is recommended that first further research is performed. In that respect it can be mentioned that this particular application with anchors in a thin concrete structure differs from the fastenings in most of the experiments that have been used as basis for the design method.

**Conclusion**

The knowledge about the behaviour of fastenings with short anchors in concrete has increased considerably during the last decade. This has resulted on the one hand in a European Technical Approval Guideline for the assessment of the behaviour and post-installed anchors. On the other hand a design guideline for fastenings with short anchors in concrete has become available.

For the adaptation of concrete structures the use of post-installed anchors for connections between new concrete elements and the existing structure may be one of the various possibilities. For that, it will be necessary to use the existing knowledge for the behaviour of post-installed anchors and of fastenings made with it.

For the connection of new concrete structural elements to the concrete frame of an existing structure, as discussed in this paper, applying reinforcement according to the reinforced concrete technique is the most common solution. To avoid the demolition of concrete in the existing building, the use of long bonded anchors is a technique that is already being used and will probably be advantageous. However, the knowledge about these long bonded anchors is still limited and therefore further research in this field is recommended. When using post-installed short anchors like torque-controlled expansion anchors or short bonded anchors, the relatively small thickness of the walls limits the capacity of the anchors. Using anchors with a larger effective depth will for the same reason only result in a minor increase of the capacity of the anchors. Nevertheless, also for this application in combination with prefabricated elements there may be possibilities in the future. Therefore further research in this direction is recommended.
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PROTECTION OF CONCRETE BE WATER REPELLENT MORTARS OR IMPREGNATION

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Abstract
In many cases, service life of reinforced concrete structures is severely limited by carbonation of the covercrete or by chloride penetration until the steel reinforcement. Today, concrete with high resistance with respect to chloride penetration can be produced by internal water repellent treatment. This would, however, not necessarily be an economical solution if the entire mass of concrete in a structural element had to be internally treated. An interesting alternative is to separate the assignments of concrete in structural reinforced concrete elements. The load bearing capacity of the element is thus provided by conventional structural concrete according to this concept. The required durability, however, is guaranteed by a protective cement-based layer. It is shown in this contribution that a protective layer with internal water repellent treatment provides concrete elements with optimal curing conditions in the first place. The same protective layer acts later as an efficient chloride barrier. This type of cement-based coating can be designed in such a way that chlorides never penetrate through the layer to reach the underlying load bearing structure during the planned service life. The durability of reinforced concrete structures can be considerably improved and can also be accurately designed by the application of an appropriate and optimized protective layer or by deep impregnation.
Keywords: Concrete, durability, cement-based protective coating, chloride barrier, internal water repellent treatment

Introduction
If cement-based materials are exposed to water a series of corrosive processes can take place. One dominant process or a combination of different processes may eventually limit the expected service life. The corrosive attack of water with respect to concrete can be subdivided at least into three different types.
First, pure water in permanent contact with cement-based materials acts as a solvant. The binding matrix consisting of Ca(OH)\textsubscript{2} and CSH-gel is gradually dissolved by hydrolysis [1]. The rate of dissolution can be considered to be a realistic indication of long-term durability of concrete in moist environment [2]. Second, gases of the environment may be dissolved in the aqueous pore solution of concrete. In this way, acids are formed, for instance by dissolution of CO\textsubscript{2} and SO\textsubscript{2}, which react rapidly with the hydration products of Portland cement. In the third type of corrosive attack water acts essentially as a vehicle and transports dissolved compounds, such as chlorides, into the porous system of cement-based materials. Capillary suction is the driving force for this mass transfer.
It is obvious that all three types of corrosive attack just mentioned act from the surface of a structural element. Water repellent treatment always means a strong interference with the humidity exchange of a porous material with its surrounding. Capillary suction, in particular, may be drastically reduced. Concrete is a cheap mass product and internal water repellent treatment would be prohibitively expensive if applied to the entire volume of a structural element. Traditional surface treatment of concrete with a water repellent agent leads to a very small penetration depth. The penetration depth can be considerably increased by new application technologies [3, 4]. An interesting alternative is the application of a mortar layer with internal water repellent treatment. This method of applying a protective coating has proven to be an efficient surface refining process.
In this contribution, the modification of a few selected properties, such as rate of drying and uptake of salt solution by internal water repellent treatment, is outlined. The influence of water repellent protective coatings and deep impregnation on durability will be discussed.

**Preparation and basic characterization of specimens**

In order to study the influence of an internal water repellent treatment on properties of concrete, a standard mix of fresh concrete has been prepared. The composition of this reference concrete is given in table 1.

<table>
<thead>
<tr>
<th>Component</th>
<th>Content in kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand, 0-4 mm</td>
<td>829</td>
</tr>
<tr>
<td>Small Aggregates, 4-8 mm</td>
<td>368</td>
</tr>
<tr>
<td>Coarse Aggregates, 8-16 mm</td>
<td>645</td>
</tr>
<tr>
<td>Portland Cement CEM I 42.5</td>
<td>350</td>
</tr>
<tr>
<td>Water ( W/C = 0.5)</td>
<td>175</td>
</tr>
</tbody>
</table>

Table 1: Composition of the reference concrete

1 % and 3 % related to the cement content of the following water repellent agents have been added to the fresh mix:
- calcium stearate
- siloxane emulsion
- silane emulsion A
- silane emulsion B

Silane emulsions A and B are similar but produced by two different companies. Properties of fresh and hardened concrete with and without water repellent agents have been determined [5, 6]. The addition of water repellent agents influences the workability, the rate of liberation of heat of hydration, stiffening characteristics, strength, fracture energy, and modulus of elasticity. In addition, the pore size distribution is significantly modified.

In this context, the parameters which indicate the moisture exchange of a concrete specimen with its surrounding are of primary interest. Drying of concrete can be realistically described by non-linear diffusion theory. Diffusion coefficients have been determined from results of drying experiments by inverse analysis. In Fig. 1, typical results as obtained for the reference concrete are shown. In this case, the diffusion coefficient is initially high, that means at elevated moisture content and then drops as moisture is lost due to drying. In the same diagramme, the diffusion coefficient observed on concrete with 3 % of siloxane emulsion is shown. In contrast to results obtained on reference concrete, the diffusion coefficient of concrete with water repellent agent is practically constant over the humidity range under investigation. This indicates that moisture transport is based on one simple mechanism in the latter case.
Figure 1: Diffusion coefficient of a reference concrete (Ref) and concrete prepared with 3% of siloxane emulsion related to the cement content (WRA) as function of relative humidity.

Water absorption by capillary suction can be approximately described by the following simple law:

$$W(t) = A\sqrt{t} \quad (1)$$

$A$ in Eq. (1) is a material parameter which has to be determined experimentally. The values of $A$ for the different types of concrete are compiled in Table II. It can be seen that capillary suction is reduced to values between 6.3 % and 15.3 % as compared with the reference concrete by the addition of different water repellent agents.

<table>
<thead>
<tr>
<th>Type of water repellent agent added</th>
<th>Water absorption coefficient $A$ [kg/m$^2$ h$^{1/2}$]</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard concrete (no addition)</td>
<td>1.31</td>
<td>100</td>
</tr>
<tr>
<td>Calcium stearate</td>
<td>0.083</td>
<td>6.3</td>
</tr>
<tr>
<td>Siloxane emulsion</td>
<td>0.200</td>
<td>15.3</td>
</tr>
<tr>
<td>Silane emulsion type A</td>
<td>0.084</td>
<td>6.4</td>
</tr>
<tr>
<td>Silane emulsion type B</td>
<td>0.112</td>
<td>8.5</td>
</tr>
</tbody>
</table>

Table 2: Capillary water absorption coefficient $A$ of the reference concrete and of mixes with 3 % of different water repellent agents.
Assignments of a protective coating

General
In many modern engineering disciplines, refining processes of the surface play a decisive role. The potential provided by advanced surface technology has so far, however, been widely neglected in concrete technology. Wide-spread damage of concrete structures can be considerably reduced by appropriate cement-based surface coatings.

Cement-based coatings can be specifically tailored in order to protect structural elements against imposed aggressive environmental conditions. Following the concept of separation of assignments [7], conventional structural concrete has to provide a structural element with mechanical properties to reach the required load bearing capacity exclusively. All environmental loads are then taken up by a protective coating.

In the following, two examples chosen from the huge variety of possible protective coatings are considered only.

Retardation of drying
Let us consider a structural element with a characteristic thickness of 200 mm. It is assumed that the surfaces of this element are exposed to air with a relative humidity of 60 % at 20 °C. By means of the diffusion coefficient given in figure 1 the time-dependent spatial moisture distribution can be simulated as function of drying time. Typical results for drying times of 3 days, 7 days, 14 days, 1 month, 3 months, 1 year, and 3 years are shown in figure 2. It can be seen that the moisture content in the first two centimeters is rather quickly reduced. This leads inevitably to incomplete hydration in the surface near zones. But it is just this part of the structural element (covercrete) which should protect the reinforcement from corrosion which remains finally most permeable.

Figure 2: Moisture distribution in a concrete slab exposed to 60 % RH at 20 °C as determined for different drying times between 3 days and 3 years
Sometimes, curing compounds are applied with doubtful success to prevent early drying of concrete elements. A more effective alternative may be the application of a mortar layer with internal water repellent treatment.

The drying of a coated concrete element can also be simulated. Typical results are shown in figure 3. In this case, it is assumed that the structural element built with the same concrete as the above mentioned example (see figure 2) has an initial thickness of 160 mm and is then covered on both opposite sides by 200 mm of a protective coating containing 3% of siloxane emulsion immediately after demoulding. The diffusion coefficient of the cement-based coating has been determined and is also shown in figure 1. Results shown in figure 3 clearly indicate that the structural concrete undergoes practically no drying within the first month. Hydration of cement will continue unhindered and the concrete will reach full maturity under these conditions throughout the volume.

![Figure 3: Moisture distribution in a concrete slab covered with a protective coating with internal water repellent treatment if exposed to 60% RH and 20°C at different drying times between 3 days and 3 years.](image)

**Absorption of aqueous solutions**

One of the major reasons for damage and deterioration as observed on reinforced concrete structures is the penetration of corrosive salts such as chlorides through the covercrete until the steel reinforcement. The most efficient vehicle for this salt migration is water. Structural elements in contact with aqueous solution of deicing salt or in the immediate vicinity of road surfaces covered with salt solution are particularly endangered [8, 9]. In conventional structural concrete, the penetration depth of chlorides...
increases with time and reaches 20 to 30 mm in a comparatively short period depending on the water-cement ratio and the curing conditions.

Surface impregnation of concrete elements usually leads to a penetration depth of the agent of 1 to 2 mm. This thin layer is unable to provide a long-term protection [10]. If, however, a cement-based protective coating with internal water repellent treatment is applied it might be possible to prevent chloride penetration into a structural element totally for a required period, i.e. the desired service life. In table 2, the reduced capillary absorption coefficients are given for concrete with internal water repellent treatment. Samples of these different types of cement-based materials have been exposed cyclicly to an aqueous sodium chloride solution with a concentration of 3 %. After one day of contact with the solution, samples were allowed to dry for one day. Unidirectional uptake has been observed. Results are shown in Fig. 4. After seven cycles, the capillary uptake of solution the untreated reference concrete tends to stabilize around a value of approximately 9 kg/m2. The internally treated samples absorb about 10 % only after the same number of cycles.

![Graph showing capillary absorption of sodium chloride solution into internally treated and reference concrete](image)

Figure 4: Capillary absorption of sodium chloride solution (3 %) into internally treated and reference concrete as function of time when exposed to two days cycles of wetting and drying

It is well-known that chloride solutions undergo a sort of filtering effect when they penetrate into microporous materials such as concrete. This means that the water front penetrates considerably faster and deeper than the dissolved ions [8]. Therefore, results shown in figure 4 do not allow us to estimate the penetration depth of chlorides immediately. Chloride profiles have been determined after seven cycles.
of exposure to aqueous sodium chloride solution. From the exposed samples, thin layers have been cut, these slices have been milled, and subsequently, the chloride content has been determined by ion chromatography. Results are shown in figure 5. Under these conditions, chloride ions penetrated more than 35 mm into the reference concrete while the penetration depth of most internally treated materials reaches values around 5 to 6 mm. If these cement-based materials are applied as protective coatings to reinforced concrete structures with a thickness of 20 mm these structures will be protected for a sufficiently long period of time. This means the aim to keep the structural concrete free of chlorides can be achieved.

![Chloride profiles](image)

Figure 5: Chloride profiles as determined in internally treated and reference concrete samples after 7 cycles of wetting and drying

Further research is needed to optimize internally treated protective coatings and to provide a solid basis for reliable design for a given service life.

**Surface treatment of internally treated mortars**

In sections 3.2 and 3.3 it has been shown that internally treated mortars can be applied to achieve optimal curing conditions and to prevent chloride penetration. The amount of water repellent agents added to the fresh mortar or concrete is limited because excessive dosage will influence the hardening process and the mechanical properties of the hardened material in a negative way. Therefore, it might be of interest to apply additional surface treatment to mortar or concrete treated internally with a water repellent agent before.
Samples pretreated internally with silane emulsion A have been surface impregnated at a later stage with pure silane. In figure 6, the concentration of the water repellent agent in an internally treated sample is shown. Within the accuracy of the analysis, silane concentration is constant along with the exception of the surface-near zone. The increased content of water repellent agent near the surface is due to the well-known border effect of concrete. Close to the surface, the cement content is higher than in the bulk material. It turned out that the pretreated concrete absorbed more silane as compared to the reference concrete at equal contact time.

![Graph showing distribution of active substance related to the mass of dry concrete](image)

Figure 6: Distribution of the water repellent agent in an internally treated concrete sample. In addition, the silane profile as observed in a pretreated sample after surface impregnation is shown.

In figure 6, the penetration profile of the silane is also shown. The observed penetration depth could not be reached in an untreated reference sample. As a consequence, we may conclude that internal water repellent treatment facilitates surface impregnation at a later stage.

**Deep impregnation**

Conventional surface treatment of concrete by means of water repellent agents leads to a penetration depth of a few mm only. In many cases this impregnation cannot be considered to be an effective protection of a structural element. It has been shown that in order to obtain deep impregnation a comparatively long period of contact between the water repellent agent and the porous building material is required.

In figure 7 the uptake of silane by concrete with different water/cement ratios is shown. Several hours of contact time are necessary in order to achieve a sufficient amount of absorbed agent. In figure 8 measured profiles of the absorbed water repellent agent are shown. It can be seen that even a concrete with a water/cement ratio of 0.35 can be impregnated by a silane up to depth of 5 mm if a contact time of at least three hours is maintained [4]. In this case the concrete cover becomes a powerful protective skin with respect to the ingress of dissolved aggressive compounds. The necessary contact time for a given type of concrete has to be determined prior to any impregnation [11].
Figure 7: Absorption of Silane (100%) as a function of the square root of duration of contact in hours

Figure 8: Penetration profile of silane (100%)

Conclusions

The penetration depth of water repellent agents is limited to very small values if conventional application technologies are applied. The protection of structural concrete elements therefore remains rather uncertain, and in many cases it is insufficient. If the contact time is sufficiently long, reliable results can be obtained [4, 11].

Another interesting alternative is a cement-based protective coating with internal water repellent treatment. These water repellent coatings offer a series of new protection techniques. They can be applied as efficient chloride barriers and, at the same time, they provide excellent curing conditions for the coated...
concrete. In particularly aggressive environment, the protective coating with internal water repellent treatment can be further surface impregnated with pure silanes. Silane penetration is even facilitated by internal pretreatment. The application of water repellent protective coatings allow us to build reinforced concrete structures with accurately designed long-term durability.

References

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STRUCTURAL STRENGTHENING BY GROUTING

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Introduction
It is very common practice to strengthen a concrete structure by injection. Injection material can be grouts based upon Portland cement or they can be chemical grouts. Portland cement grouts have been very much researched and are most commonly used grouts. However, in many cases Portland cement grouts cannot be used. Portland cement grouts are water-cement suspensions; they are not true solutions. The size of the cement particle limits the size of the opening into which this grout can be injected. No doubt with the use of a chemical admixture it is possible to make very fluid cement grout which increases its penetrability. Nevertheless, it is universally accepted that particulate cement grouts cannot be injected into voids that are smaller than about 3 times the diameter of the cement particles. It should also be borne in mind that the Portland cement grout has very low bond and tensile strengths and is not usually used if structural rebonding of a cracked structure is the objective. Hydration of cement grouts occurs over a long period of time. Accordingly, it is not possible to use Portland cement grouts in areas where high-speed water flow through the fracture in the rock foundation or concrete structure is required. However, they are used in other places for strengthening the building structures.

Cement Grouts
These are made with very fine cement (very high specific surface area), bentonite clay, water and superplasticizer. Superplasticizer is added for obtaining fluidity at low water to binder ratio. The fluidity is checked by injecting the grout through a column made of Plexiglas and filled with crushed brick of different size to get good packing. The time taken to fill the column is noted and the flow is measured at different time intervals (figure 1). This is done to determine the time up to which the grout will be in the fine dispersion form before becoming coagulated.

Chemical Grouts
These are true solutions, and generally do not have limitation of the void size for penetration. Some chemical grouts can be prepared with gel or setting times that vary from a few seconds to several hours. The first use of chemical grouts occurred in 1900 and the grout contained a crude form of sodium silicate. Silicates still make up the most commonly used and cheapest chemical grout preparations. This type of grout, however, forms a low strength gel and has other deficiencies, which led to the development of organic polymer grouts.

Recently, colloidal silica has been developed by Eka chemicals in Sweden, and this is also used for consolidation. It consists of very fine amorphous silica particles in dispersion form. The particle size can be 500 nm. These are very fine particles and are easily penetrate in the concrete. It fills the cracks, fissures and voids. Amorphous silica is very highly reactive pozzolan and thus reacts with the calcium hydroxide produced during the hydration of Portland cement. The excess amount makes a thick gel in the pores and capillaries. Consequently, the liquids (salt solutions) which are found in the old concrete and are the source of the chemical deterioration of the concrete are pressed out to the surface where they are wiped off by wet sponge. Thus, this grout serves two purposes: 1) Consolidation, which strengthens the structure, and 2) Desalination, which decreases the salt contamination of the concrete structure, so increasing its service life. It is completely inorganic material and is thus environmentally friendly.
The use of organic polymers such as epoxies, polyesters, polyurethanes and acrylics for chemical grouts first began in the 1950's. These materials occupy only a very small percentage of the foundation grouting market. However, they take a much more prominent position when the definition of chemical grouting includes the similar and closely related technologies of crack injection for structural rebonding and water flow control.

Here the use of polymeric resins for two types of grouting techniques is discussed:
1. Crack injection grouting
2. Grouting for foundation stabilisation or for water control

Crack Injection Grouting
Polymeric resin is injected into the cracks for purposes of structurally rebonding the cracked surfaces, to stop or control water leakage from or through the crack, or for a combination of these two purposes. The type of resin used for crack injection grouting and the grouting technique depend on the purpose of the grouting and on the nature of crack,

Crack Injection Grouting Material
Structural rebonding is usually performed with polyester, epoxy, or other types of resin that develop high bond and tensile strength. If the crack injection is done to control water leakage, either epoxy resins or polyurethane resins can be used. Epoxy resins can be used if there is assurance that the crack will not experience future movement and if the flow of water from or through the crack is relatively low. High water flows will remove epoxy resin from the cracks before it can cure or harden. Polyurethane resin reacts with water to form an expanded, flexible foam. This is a very fast reaction and is thus suitable for blocking or sealing large water flows even under moderate pressure. The density of the resulting foam can be varied by selecting the water to resin mixing ratio to suit the needs. The polyurethane foams have relatively low bond and tensile strengths and are thus not suitable for structural rebonding applications. There are various types of polyurethane resin, each having different application and physical properties. There is no standard specification for these resins such as exists for the ASTM C881 for the epoxy resins.

Crack Injection Grouting Techniques
Crack injection is done by drilling the injection ports along the crack and injecting the selected polymeric resin into the crack to the desired degree. In the crack injection process, the resin is injected resin under pressure into each port, in sequence, until the entire crack has been treated.

Grouting for Foundation Stabilisation or for Water Control
Foundation stabilisation grouting with polymeric materials has been the major attraction for the use of chemical grouting materials. This type of grouting has found uses in dam and tunnel construction, the rehabilitation of structures, new building construction and in projects specially dealing with relatively unstable soils or the control of water seepage.

Foundation Grouting Material
The most common chemical grouting materials have been the sodium silicates, the acrylamides, and the acrylates. But the acrylamides and acrylates are not so very environmentally friendly and the polyester and epoxy resins are too expensive for use, except in special cases. Polyurethane grouting was initially used to solve the leakage problem in underground sewer systems. This was done simply by injecting a mixture of water and polyurethane into the soil mass surrounding the leakage. It was noticed that the
polyurethane could also be effective in sealing the damaged or cracked area of the pipe itself.

**Foundation Grouting Technique**
The methods used for the chemical grouting are identical to those used for cement grouting. The major difference is that the gelling time is too short. Therefore, the injection is to be done under very precise and well-planned conditions unlike cement grouting. The technique is easy to understand by studying a case where it is applied. One such case, the Upper Stillwater Dam, is described here.

**Upper Stillwater Dam Repair - A Case History**
The Upper Stillwater Dam is situated about 60 km. north of Duchesne, Utah USA. It was constructed from roller compacted concrete. During the first filling of the reservoir, a continuous crack was discovered in the foundation gallery at the station 25+20. The crack also appeared on the upstream and downstream faces of the dam. As the reservoir continued to fill, the crack widened and produced an unacceptable amount of leakage into the foundation gallery and out of the downstream face. At full reservoir head the crack measured about 6.6 m. wide. About 4.9 m³/min water were leaking from the gallery and about 6.6 m³/min were leaking from the crack on the downstream face. The crack ultimately extended from the foundation to the crest and from the upstream face to the downstream face. The water leakage is shown in the figures 2 and 3. As the reservoir was drawn down, water leaking from both the gallery and the downstream face decreased to about 3 mm.

These cracks were repaired injecting polyurethane resin. It was performed in three stages. The main thing done in the first stage was to cut off the flow of water into the foundation gallery work area (figure 4).

**First stage:**
A series of relatively shallow holes (16 mm diameter) were drilled from the gallery walls to intercept the crack at depths of 0.3 to 0.9 m (figure 5). After the holes had been drilled, valved injectors known as ‘wall spears’ were placed in the holes with the valves open to relieve water in the crack. The surface of the crack was then between wall spears with wooden wedges, lead wool, or urethane soaked jute rope. Once the flow of water was controlled, the wall spears were connected to the urethane pump system and injected with resin. After injection, the wall spears were removed.

**Second stage:**
This was the major work in the crack repair. Crack intercept holes were drifted from 5.8 to 28 m (figure 6). The D-line of holes intercepting the crack (1.5 m) downstream from the face was drilled, but first to reduce the flow during drilling, the interior holes A, B, C and E were drilled. The alphabetical designation of the holes indicates the sequence of drilling and injection. The water to resin ratio varied from 0:1 to 21. Most of the injection was done at a 1.1 ratio. The closure pressure varied from 4000 to 8300 kPa). Check holes were drilled to determine if the crack had been sealed by the injection. If not, additional holes were drilled and injected until the crack was adequately sealed. In some instances the holes served as subsequent injection holes.

**Third stage:**
The final stage of repair was performed from the upstream face of the dam using a floating barge and a spider platform suspended from the top of the dam (figure 7). The crack zone located above the reservoir water level was injected (figure 8). Standard 16 mm diameter holes spaced 305 to 610 mm apart were
drilled on alternate sides of crack. Wall spears were installed and the holes were injected with urethane resin starting with the lowest hole and proceeding up the crack to the top of the dam or to the top of the crack. A similar procedure was used to repair the cracks at other stations. These cracks were not very wide, a reduced injection zone was adopted (figure 9). After injection of the cracks, and at full reservoir, no measurable leakage was detected.

Figure 1 (l): Fluidity test of the cement grouts
Figure 2 (r): Water leaking into the gallery
Figure 3 (l): Water leaking from the downstream face at full reservoir, station 25+20.

Figure 4 (r): Stage 1 injection plan at station 25+20.

Figure 5: Gallery wall crack intercept drill plan.
Figure 6 (l): Stage 2 injection plan at station 25+20
Figure 7 (r): ‘Spider’ platform and barge used to inject the upstream face.

Figure 8: Stage 3 injection plan at station 25+20
Figure 9: Zone of resin injection at station 41+10.
Collaboration between existing and newly added concrete structures

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Introduction
Buildings are often changed during their lifetime. This is so for all types of buildings, whether they are considered to be a part of the urban heritage or not.

These changes have, in most cases, to do with changing demands of the occupants of the building. With those demands will mostly be coped by just changing parts of the interior, whithout a need to change parts of the (concrete) structure.

In some cases, especially when the use of the building changes, parts of the structure have to change as well. The most common changes deal with the creation of openings in floors for installation of pipes or for new staircases or elevators. In rare cases the tolerable floorloads have to be increased.

Buildings with a concrete structure, that were erected in the sixties of this century or earlier, are mainly cast in situ. That means that in most cases it is rather easy to make openings in floors. In the case of increasing the tolerable load, the installation of additional reinforcement, eventually combined with thickening of the floorslab, is often rather simple to execute.

In cases that the loads of column or foundations increase many techniques exist to strengthen the columns and/or the foundations.

If excessive changes in use or loading appear, changes aren’t made so easy.

When newly structures have to collaborate with existing structures, both the strength and the (time-dependant) deflection of old and new make a difficult mix.

Case 1: Government building in Arnhem

Figure 1: Large openings in the floors for better communication between the collaborators at the different floors. Existing beams stay in place. 
*Photo Government Building Agency*

Figure 2: Cross-section of the office in Arnhem after renovation. Extra floors have been added.
In 1980 the station postoffice in Arnhem was transformed into an office building. Extra floors were added where enough height was available. Some of the existing floors were partly opened to allow a better communication within the building (see figure 1 and 2). The adding of the extra floors didn’t give many problems because of the large loading capacity of the existing floors. Those floors had to carry much less load as a office floor as before.

Making large openings in some floors was not difficult because the existing beams stayed in their place.

In this case we can say that all the changes in the concrete building structure could rather easy be realised.

Case 2: Telecommunication tower in Rotterdam

In 1989 the telecommunication tower in Rotterdam that had a height of 106 m has been heightened to a height of 167 m, with the possibility of an steel tower on top of 40 m high. The new height would allow higher buildings within the centre of Rotterdam. An important telecommunication path went from the tower straight through the centre of Rotterdam and on to the telecommunication tower in Alphen. Due to that path the building height in the centre of Rotterdam was limited to 60 m. After intensive studies was concluded that heightening of the Rotterdam tower was the easiest solution.

The existing tower, build in 1962, was designed to have, under maximum wind load, a maximum rotation in any direction of 0.5 degree at the top.

The heightened tower had to be designed to have a maximum rotation of 0.3 degree at the (167 m high) top. That meant that the tower and its foundation had to be substantially stiffened.

During the construction work the telecommunication had to go on without any interruption.

After a little calculating is was clear that not only the tower structure but as well the foundation had to be strengthened and stiffened.

With the assistance of a consulting architect a plan was quickly made: some extra piles around the existing foundation, some concrete added to the existing tower structure and an extra 61 m on top of the existing tower.

However, during the designing process the following problems still had to be solved.

The existing piles should always remain under pressure. The bottom of the piles was situated at only a small distance inside the loadbearing sandlayers. Coming under tensile, the piles would be torn out of the sandlayer, resulting in large deflections. Newly added piles could deal with tensile forces if they would be placed into the soil unto a much deeper level as the existing piles. After intense calculations a design of the foundation has been made where the existing piles would always remain under pressure, while the new piles under certain condition would come under same tensile. Enlarging the foundation slab under the tower, to lead the forces in the tower to the new piles was not acceptable. That would have meant that the cellar under the tower would have to be filled with concrete, to make the slab thicker. But that cellar was specially designed as an entrance for communication cables and no disturbance was accepted at all.

That is why on top of the new piles a big ring girder was positioned, that was connected with the new “rocket”-wings of the tower. In this way there was also reached an connection between old and new foundation.
Figure 3: Enlarging the foundation with 127 new piles of 23 m long. Existing were 102 piles of 19 m long. The ring girder has been completed.

photo IBC Bouwgroep

The connection between the existing tower and the added wings not only had to ensure the strength and stability but also the demanded stiffness. That is why much attention has been given to the connection between old and new concrete. There was of course a risk of cracking at the connection between old and new concrete, due to shrinkage of the new concrete.

To ensure both strength and stiffness the connection between old and new concrete was made in three ways. First bij adhesion. The cement skin of the old concrete was removed to insure a good bondage. Second, there were connecting bars glued into the existing tower. And in the third place existing windows of the tower were filled with the concrete of the added wings so that an extra connection was made.
Figure 4: Removing the cement skin of the existing tower at the connection with the new wings. Connecting bas have been glued.

*photo IBC Bouwgroep*

Figure 5 (next page): Tower ready for constructing the new wings. Existing windows will be filled with concrete.

*photo IBC Bouwgroep*

Figure 6 (next page): Constructing the wings that also connect the new foundation via the tower to the existing foundation.

*photo IBC Bouwgroep*
At the connection between the new top and the existing tower there was also a problem to solve. The reinforcement in the existing tower decreased with the height to the absolute minimum at the top. That meant that no tensile stresses could be allowed at the connection between old and new. The only possibility to allow tensile stresses would be a strengthening of the existing tower to the very top. But by doing so the construction would have entered the “forbidden zone” of the communication antennas. That is why the dimensions of the top (thickness of walls) have been chosen in a way that the weight of the top would be high enough so that no tensile stress would appear.
Figure 10: The completed tower: strengthened and stiffened, with a calculated bond between old existing and newly added concrete.

*photo IBC Bouwgroep*

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Strengthening Concrete Structures with the aid of bonded reinforcement

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Introduction

When alterations are being made to existing concrete structures, it appears that there is often sufficient reserve in the pressure zone of a cross section. In the tension zone, there is usually just sufficient reserve to guarantee the calculated load transfer. If alterations are desired, in the first place it will be necessary to ascertain whether the existing reinforcement satisfies the new demands. If this is not the case, it is possible to increase the existing reinforcing by adding extra bonded reinforcement to the concrete surface.

This extra reinforcement usually consists of thin steel strips that are bonded to the concrete by means of a 2-component epoxy resin adhesive. By means of the shear stress in the bonding joint between the concrete and steel this composite construction can serve to transmit extra tensile, compressive or shear forces. This strengthening can be used in the following situations:

* Changes in the static system resulting from structural alterations,
* Increase in the working load,
* Design or construction errors,
* Corrosion of the existing reinforcement,
* Fire damage.

This first use of this method dates from 1962; bonded reinforcement has been used in the Netherlands since 1982. After the introduction of the system, thoroughgoing research was carried out into aspects of design, material and execution.

In addition to the use of steel strips, now strips strengthened by carbon fibres are also used. These demand special attention with regard to the constructional aspects.

General behaviour of strengthened structures

The behaviour of a concrete structure strengthened by bonded reinforcement is determined by the properties and interaction between:

* Concrete
* Adhesive
* reinforcement

The bearing capacity of the strengthened structure is determined by the quality of the cement in the connection layer. To obtain good adhesion the concrete must be roughened by means of grit blasting or light bush hammering. The adhesive to be used must have good adherence properties with regard to concrete and steel. The interaction between the concrete and reinforcement must be guaranteed in both the short-term and the long-term.

Mild steel (Fe 360), provided with adequate anti-corrosive protection is usually used for reinforcement. The use of higher tensile strengths generally leads to impractical strip measurements in relation to the maximum stresses to be transmitted to the concrete. It is possible to use other materials such as a stainless steel or carbon fibres.
Strengthening with the aid of bonded reinforcement leads to:
* greater stiffness
* greater accepted loads

**Greater stiffness**
Greater stiffness is caused by:
* Bigger steel cross section,
* Steel in the outermost fibre.

The effect on stiffness is shown in figure 1.

**Greater accepted loads**
If well finished, strengthening with the aid of bonded reinforcement can be used to take up bending moments and shear or normal forces. Strengthening will be achieved in both the users phase (crack width) and in the failure stage.

**Failure behaviour**
Failure is caused by:
a. yielding of the inner reinforcement
b. yielding or the outer reinforcement
c. rupture of the joint between the reinforcement and the concrete

**N.B.**
1. Failure behaviour resulting from c. should be prevented by good finishing of the anchorage
2. Yielding of the inner or outer reinforcement is affected by structural loading during the application of the bonded reinforcement.
Materials used

Adhesive
A 2-component epoxy adhesive is used to bond steel to concrete. Adhesive based on epoxy resign is suitable because it possesses the following properties:

* good adherence to steel and concrete,
* low sensitivity to humidity
* small shrinkage during hardening
* few creep phenomena
* good resistance to ageing,
* good alkaline resistance.

Steel
In principle, steel of all types and qualities is suitable for use in bonded reinforcements. Usually, mild steel (Fe 360) is used, with a conservation that can be suited to the environment.

In special cases, other types of steel, such as stainless steel, can be used. Higher quality steels usually lead to impractical dimensions as concrete - the measure for the link - cannot transmit the higher initial forces.

For the strips the usual dimensions are:
* thickness between 5 mm and 10 mm
* width between 50 mm and 200 mm

Concrete
The shear strength of concrete at the site of the bonded joint determines the behaviour of the composite construction. The concrete must satisfy the requirements relating to quality and flatness. Adhesion testing apparatus can be used to determine the adhesion value of the surface of the concrete. With this, the characteristic adhesion-tensile strength (Öcld) can be determined. This is a measure for the characteristic adhesive -shear strength (Öcld).

To obtain reliable results the surface of the concrete must be adequately pre-treated; the tensile strength of concrete a few millimetres below the surface is considerably higher than it is on the surface (see figure 2).
Special considerations during execution

1. Pre-treatment of concrete
To ensure good bonding/adhesion with the concrete the cement skin of the concrete must be removed. The best results are reached by grit blasting. Small surfaces can be treated by light bush hammering. Various techniques have been developed to reduce dust trouble while grit blasting. After pre-treatment the surface of the concrete must be well dried and made dust free. Big irregularities are filled up with epoxy mortar before the gluing process begins.

2 Pre-treatment of steel strips
An epoxy primer is used on the sides of the steel strips that are to be covered by the epoxy adhesive. Before the adhesive is applied the steel on this side is lightly sanded and degreased.

3 Application of the adhesive
To strengthen concrete structure with bonded/adhesive reinforcement a two-component adhesive based on epoxy is used. The components, resin and hardener are mixed on site to a homogenous mass. The working time depends on the composition and temperature and is approx. an hour.
4 Mounting of the strips
After the application of the adhesive the strips are pressed against the pre-treated concrete. During the hardening period of the adhesive, the pressure that is applied must remain constant. Usually screw jacks or threaded rods are used for this.

5 Monitoring during execution
The quality of the adhesive bonding must be largely determined during the execution of the work. A number of points demand special attention since poor working will affect the total result of the strengthening operation.
In the first place, inadequate pre-treatment of the concrete can affect the interaction with the steel. The transmit of forces from the concrete via the adhesive to the steel is usually determined by the concrete. The quality and pre-treatment of concrete can be monitored by means of pulling off tests in which discs glued to concrete are pulled off the concrete. The force needed for this indicates the quality of the concrete and of the pre-treatment.
With good pre-treatment a bond stress of 1.3 N/mm² (with B15) to 2.0 N/mm² (with B30) can be reached.

In the second place, the quality of the bonding is determined by conditions during the working. Low temperatures, and/or high humidity in particular can have adverse affects on the result.
To ensure good quality the temperature and relative humidity must be regularly checked. During working the temperature may not be below 100°C and the maximum humidity is 80%.

5 Boundary conditions for construction
If the design rules are used correctly, the behaviour of the structure is indicated by the behaviour of the adhesive bond in the anchoring zone. If the finishing is correct, the tensile strength in the bonded reinforcement is transmitted to the inner reinforcement. Good design and execution can prevent premature loosening of the strip. The section where the existing construction is just sufficiently strong to

Figure 3

ensure the required security against failure is considered (see figure 3).
In this section, the tensile force in the strip is calculated. Behind this section there must be sufficient length to anchor this tensile strength. If the required anchoring length is not available broader strips can be used or a bolt can be placed in the outer end of the strip. It is recommended that the strips should be anchored in places where no bending tensile cracks occur.