State of the Art Report on new technologies to monitor, conserve and restore the materiality of modern buildings in a compatible, durable and sustainable way

Project: CONSECH 20

Working Package 2 – Task (i)

Version 03

Date: November 07, 2019

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Introduction

Concrete is a 'young' material, used in Europe from the middle of the 19th century onwards. Initially, concrete was a secondary material, used mainly for planters and small objects, which developed as a primary construction material with the second industrial revolution by the end of the 19th century. Short after, concrete became one of the first global construction materials and thousands of buildings were built worldwide.

As in any ground-braking new technology, the first concrete buildings were experimental. There was no past experience or a solid scientific knowledge behind these pioneering structures. As a result, different design flaws can be encountered in early concrete constructions such as lack of sufficient concrete cover to reinforcement, use of non-suitable materials in the mix design, such as materials with high contents of chloride, low concrete and steel strength (f_c<20 MPa / f_s=220 MPa), sparse and poorly anchored stirrups, lack of stiffness, low ductility, or an overestimation of durability capacity. Additionally, the corrosion damage encountered in most early concretes leads to loss of rebar sections, embrittlement of steel, bond deterioration and cover delamination. Especially in seismically active zones, the lack of knowledge of the overall response of early concrete structures to seismic excitation led to lower than required reinforcement ratios of the structural elements and wrong design practices.

By the 1970s, concrete repair had become a major issue, but until the late 1980s little research was carried on concrete repair and conservation. It has also taken years for the conservation practice to merit cultural heritage (CH) concrete buildings a historical, technical and cultural status.

Concrete conservation techniques have evolved during the last decades, although there is still justifiable caution about different methods of repair. This is due to different factors, such as long-term impact in historic buildings, and general lack of sensitivity and knowledge on types of damage, causes and repairs.

One of the challenges is, therefore, to accurately predict the ongoing threats to a (reinforced) concrete structure and how this will respond to these threats, and then to determine what type of repairs are necessary.

This report aims at supporting the conservation of CH concrete buildings by providing a concise and still comprehensive state of the art on the most common:

- Damage processes affecting concrete structures and related damage types (Chapter 1Damage Processes and Damage Types);
- Techniques for survey and monitoring of the state of conservation of concrete buildings and structural assessment (Chapter 2);
- Materials and techniques for conservation and rehabilitation of (historic) concrete buildings (Chapter 3).

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1 Damage Processes and Damage Types

As part of the conservation process, identifying the type of damage and what is the main cause, or causes, is an essential key for a successful intervention. In addition, using a common and standardized terminology will help to unify the condition on the different case studies under investigation in this project.

Once the causes of damage are known, two other factors should be addressed: how much of the building or the structure is affected, and how fast the damage is progressing (Andrade and Martínez, 2009). In this sense, one of the aims of this working package is to establish and validate the relation between types of damage and damage processes.

1.1 Damage processes

1.1.1 To concrete as a whole

1.1.1.1 Damage caused by structural loads (Structural damage)

The nature can be diverse: accidental (such as seismic, wind, or impact), differential settlements, displacements, or overloading.

The general damages types are related to cracks due to shear, bending, anchorage fracture, or fatigue fracture.

1.1.1.2 Fire

Obvious signs of distress are cracking and spalling of the surface, caused by differential thermal expansion of successive layers of the concrete and by internal pressure developed by moisture in the concrete turned into super-heated steam. Secondly, the material properties change.

Heating concrete above about 300°C reduces its compressive strength; the reduction is approximately linear with temperature, with all strength being effectively lost at about 1000°C. Similarly, there will be a reduction in the strength of any reinforcing steel, which may have a significant effect on the performance of the structure (The Concrete Society TR78, 2008).

1.1.2 To concrete material

1.1.2.1 Alkali-Silica-Reaction (ASR)

Alkali silica reaction (ASR) in concrete is a deleterious chemical reaction between alkali hydroxides in the pore solution and reactive silica found in some aggregates. The reaction results in the formation of a hydrophilic gel (ASR gel) that swells in the presence of moisture. This causes expansion and cracking of concrete structures; the surface cracking can leave the concrete exposed to other deterioration mechanisms such as corrosion and frost action. As with many chemical reactions, higher temperatures will increase the rate of reaction, leading to more rapid development of distress in warmer climates (E. Giannini, K. Folliard, J. Zhu, O. Bayrak, K. Kreitman, Z. Webb, 2013).

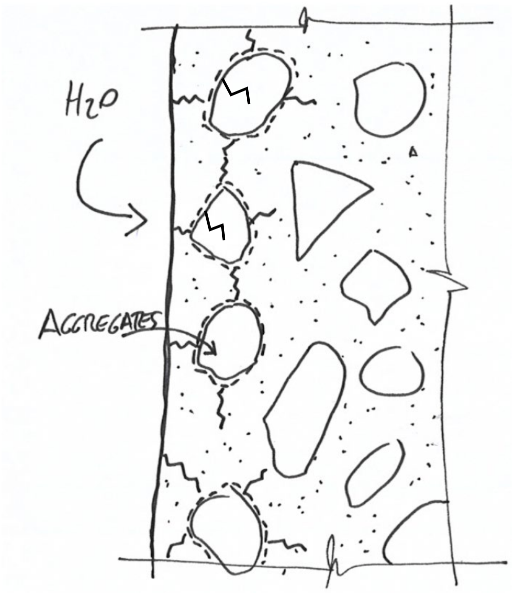

Figure 1. Schematic illustration of ASR

1.1.2.2 Delayed ettringite formation (DEF)

DEF is associated with the delayed formation of the mineral ettringite which is a normal product of early cement hydration. DEF has been linked with elevated temperatures (above 65 °C) of fresh concrete, either due to an internal cause such as hydration of cement in mass concrete, or to an external cause such as steam-curing, a method often used in pre-casting (Larosche, 2009).

DEF in concrete is a form of sulphate attack. The source of sulphates can be external (concrete in contact with sulphate-containing soil and water), or internal (present in the concrete mix such as contents sulphate-rich aggregate, or excess of added gypsum in the cement or contamination).

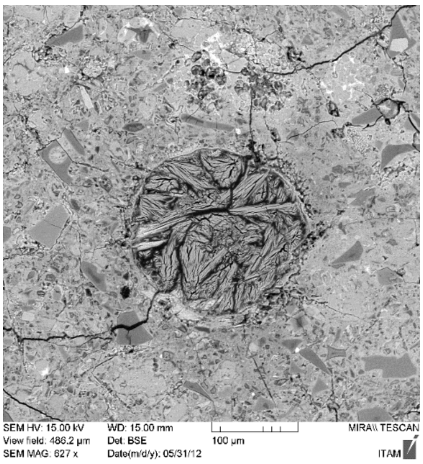

Figure 2. Microscope image of ettringite in old concrete (Crisitana Lara Nunes)

The hydration of cement and formation of calcium silicate hydrate(C-S-H) is greatly accelerated as the curing temperature increases. Once temperatures return to "normal" levels experienced by concrete in service, thermodynamics favours the formation of ettringite. Trapped sulphates may be released from the C- S-H and react with water and monosulphate to form ettringite; this can lead to deleterious expansion and cracking of the concrete (E. Giannini, K. Folliard, J. Zhu, O. Bayrak, K. Kreitman, Z. Webb, 2013)

DEF can cause similar damages as in ASR (expansion and cracking), the main difference can rely that ASR can also crack the aggregate.

1.1.2.3 Formation of thaumasite

The thaumasite form sulphate attack (TSA) is another form of sulphate attack, similar to DEF. It is a unique distress mechanism in ordinary Portland cement (OPC) concrete in which thaumasite formation alters the primary binder, calcium silicate hydrate (CSH).

Like conventional sulphate attack, TSA takes place in the presence of sulphate-bearing ground waters or soils. In the more conventional forms of sulphate attack, ettringite is the primary component formed, and most of the damage is considered to be associated with expansion and cracking induced by its formation. The amount of ettringite that can form is limited by the alumina content of the cement. However, in the case of TSA, additional forms of damage can occur since thaumasite is not limited by the alumina content of the cement. Thaumasite can continue forming as long as external sulphate and carbonate sources continue to be available (Macphee and Diamond, 2003).

TSA in concrete may cause loss of paste-aggregate bond, strength, coherence, and eventually serviceability (Hou *et al.*, 2015). TSA is not common in concrete structures, and it is rarely found in Europe.

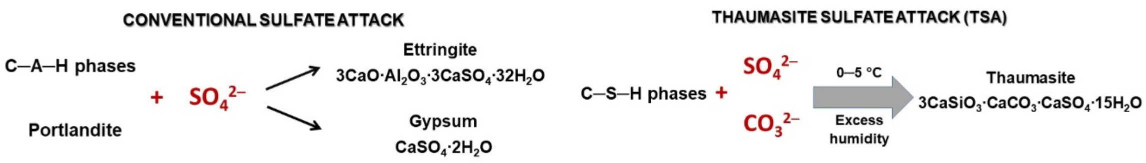

Figure 3. Conventional sulphate attack (left) and thaumasite sulphate attack (right)

1.1.2.4 Frost damage

In hardened concretes, water freezes from outside towards inside: due to freezing of part of the water in a saturated material, the not yet frozen water is put under pressure. The pressure becomes so high that the concrete eventually disintegrates.

1.1.2.5 Crystallization of soluble salt

Surface scaling due to salt crystallization, also known as salt scaling, is a form of deterioration of concrete. The damage mechanism is due crystallization pressure created by salts growing from a supersaturated solution (Thaulow and Sahu, 2004). When the pressure of the salts in the pore structure exceeds the tensile capacity of the binder, cracking can result. Over time, with cyclic events and weathering, the initial cracks can progress making the element more permeable to the ingress of other substances.

The salts present in concrete as efflorescence or crypto-efflorescence are usually sodium sulfate (Na2SO4, thenardite) and hydrous sodium sulphate (Na2SO4d 10H2O, mirabilite) (Thaulow and Sahu, 2004). Salt crystallization due to sodium sulphate generate a severer scaling than other salts such as sodium carbonate and sodium chloride. In addition sodium sulphate can introduce chemical attack on the concrete (Haynes *et al.*, 2010).

An important point relevant to old concrete is the study of Lee and Kurtins in 2017. The study points out that damage due to salt crystallization on cement pastes with high water/cement ratio (w/c > 0.4) can lead to more severe type of damage (Lee and Kurtis, 2017). Old concretes tend to have higher w/c ratios than 0.4, therefore they may be more prone to this type of damage.

1.1.3 To reinforcement

Reinforcement corrosion has been identified as being the predominant deterioration mechanism for reinforced concrete structures, which seriously affects the serviceability and the safety of the structures.

Steel bars corrode when air (oxygen) and water (humidity) are present. However, when steel is embedded in concrete it does not corrode, in fact, the concrete protects the steel against corrosion.

Non-carbonated concrete is alkaline (pH 12-13) as it contains high concentrations of soluble calcium, sodium and potassium hydroxides in pore water. The alkaline condition leads to a 'passive' layer forming on the steel surface. This passive layer is a dense, impenetrable film which, if fully established and maintained, prevents further corrosion of the steel. The layer formed on steel in concrete is probably part metal oxide/hydroxide and part mineral from the cement. A true passive layer is a very dense, thin layer of oxide that leads to a very slow rate of corrosion. There is some discussion whether or not the layer on the steel in concrete is a true passive layer as it seems to be thick compared with other passive layers and it consists of more than just metal oxides; but it behaves like a passive layer and it is therefore generally referred to as such (Broomfield, 2007).

When the concrete pH at the interface with the steel drops to levels below 8, the passive layer is dissolved. In this state, iron atoms in the steel turn into positively charged ions (oxidation or anodic reaction), creating an excess of electrons in the metal (Equation 1). To keep the electrical balance, the steel conducts another reaction (reduction or cathodic reaction) that dissolves the oxygen (Equation 2). The oxidation-reduction mechanism creates a corrosion cell (see Figure 2).

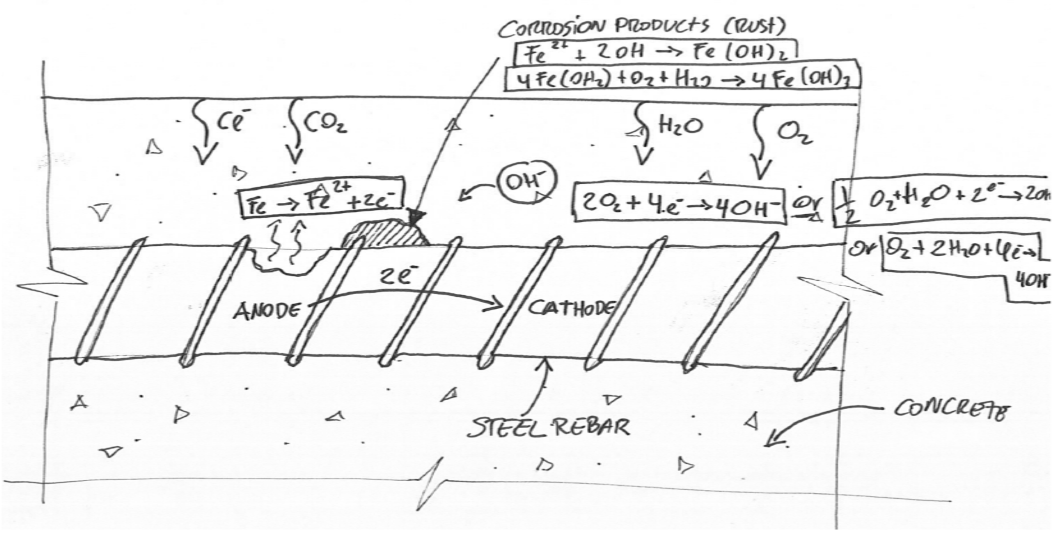

Figure 4. Typical corrosion cell in reinforced concrete

Concrete structures exposed to the atmosphere usually have an adequate supply of oxygen and moisture diffusing through the concrete cover to support the cathodic reactions. The hydroxyl-ions produced in the cathodic reactions migrate under the influence of an electrical field towards the anodic sites where they neutralise the dissolved ferrous ions (Fe^{2+}) to form a fairly soluble ferrous hydroxide, which are the corrosion products or rust (Equation 3 is the most common corrosion product). Depending on the surrounding environment (oxygen, moisture, temperature, pH, chlorides, etc.) these initial products can be further oxidised to voluminous forms of insoluble hydrated rust, for instance the one shown in Equation 4.

$$Fe \Rightarrow Fe^{2+} + 2e^{-} \qquad \text{Equation 1}$$

$$2O_2 + 4e^{-} \Rightarrow 4OH^{-} \qquad \text{Equation 2}$$

$$Fe^{2+} + 2OH \Rightarrow Fe(OH)_2 \qquad \text{Equation 3}$$

$$4Fe(OH)_2 + O_2 + H_2O \Rightarrow 4Fe(OH)_3 \qquad \text{Equation 4}$$

Two processes can break down the passivating environment in concrete. One is carbonation and the other is chloride attack.

Figure 5. Corrosion in concrete wall due to carbonation

1.1.3.1 Carbonation induced corrosion

Carbonation is a normally occurring process in concrete. Carbonation is due to the fact that in the presence of carbonate ions, formed by the CO_2 and H_2O in the environment plus the high alkalinity in the concrete, the calcium ions (Ca^{2+}) of calcium hydroxide (CH) in the concrete precipitate and form calcium carbonate ($CaCO_3$) (see Figure 4). In the process, the calcium ions are consumed, reducing their concentration, and the alkalinity of the concrete is reduced towards neutral pH values, around 7.

If the carbonation front reaches the concrete next to the steel reinforcement, due to insufficient concrete cover, cracks or poor quality of the concrete, the passivity layer starts a degradation process, and corrosion may occur (Lagerblad, 2005; Andrade, 2019) (see Figure 3).

The carbonated concrete layer, in OPC concretes, tends to be denser due to the water loss in the formation of $CaCO_3$. This denser carbonated layer protects the concrete behind and reduces carbonation. However, this is not the case in slag-cement concrete, where the carbonation process creates a more porous layer and water penetration and oxygen can easily penetrate to corrode the steel.

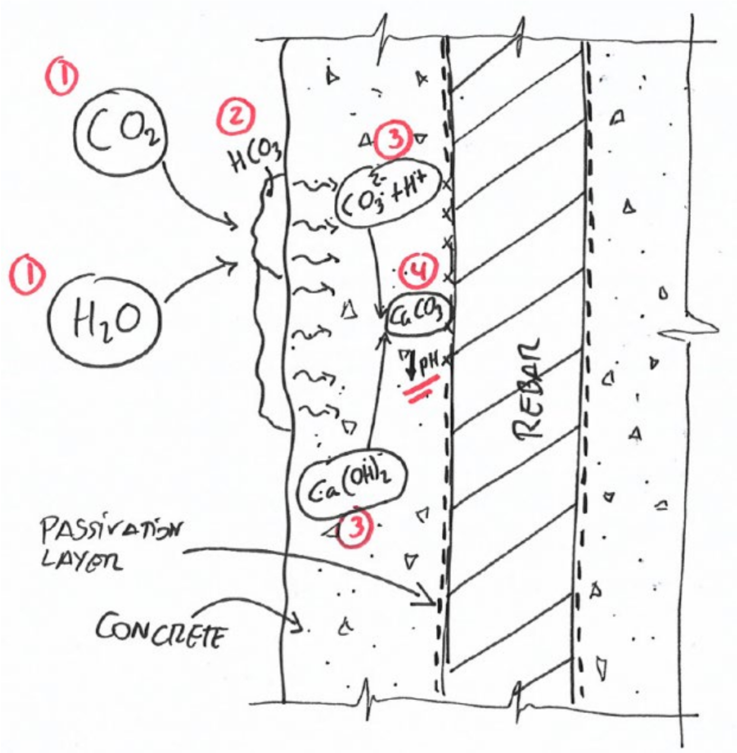

Figure 6. Carbonation process in concrete

1.1.3.2 Chloride induced corrosion

In the presence of chlorides the passivation layer is destroyed, independently from the pH value. When the first iron atoms are released into the solution by the chloride ions, they hydrolyse the water, lowering the local pH (Andrade, 2019). Contrary to the carbonation induced corrosion, the passivation layer is only destroyed locally. In this way, as the corrosion product is prevented from precipitation, and due to the existence of highly active and localized anodic sites, a severe pitting corrosion may occur without an earlier warning through visible signs at the surrounding concrete. The iron ions (Fe2+) react with the chloride ions adificying the area (Equation 5), with the presence of water it dissolves the corrosion products (Equation 6) and causes pits in the steel, that is why this corrosion type is also called pitting corrosion (Figure 5).

$$Fe^{2+} + 2Cl^{-} \Rightarrow FeCl^{2}$$ *Equation 5*

$$FeCl^{2} + 2H_{2}O \Rightarrow Fe(OH)_{2} + 2HCl$$ *Equation 6*

Chlorides can be present from two sources: added in the mix or ingressed from the environment. In historic concrete, chlorides may be present as they were incorporated in the mix through the use of contaminated aggregate, sea water or brackish water, or admixtures containing chlorides (Neville, 1995). The external source can be due to de-icing salts or contact with maritime environment.

This type of corrosion does not initially form spalls or cracks in the concrete surface, instead rust stains can be visible in the concrete surface. However, the inward corrosion can have large consequences since it can reduce the cross section of the steel without showing signs of damage.

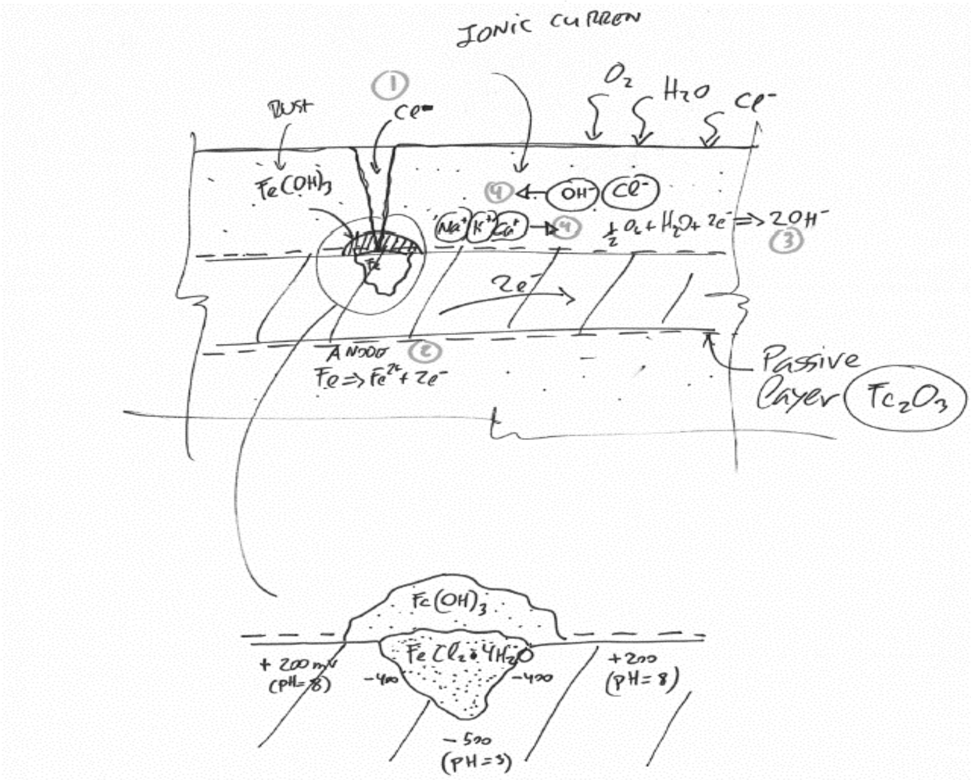

Figure 7. Chloride induced corrosion

1.2 Damage Types in (Historic) Concrete

Damage types are a consequence of one or more damage processes. A list of damage types in concrete and their explanation can be found in the website MDCS: https://mdcs.monumentenkennis.nl/damageatlas/7/material

Below is the list and classification of the damage types separated by damage to the concrete material and damage to the reinforcement.

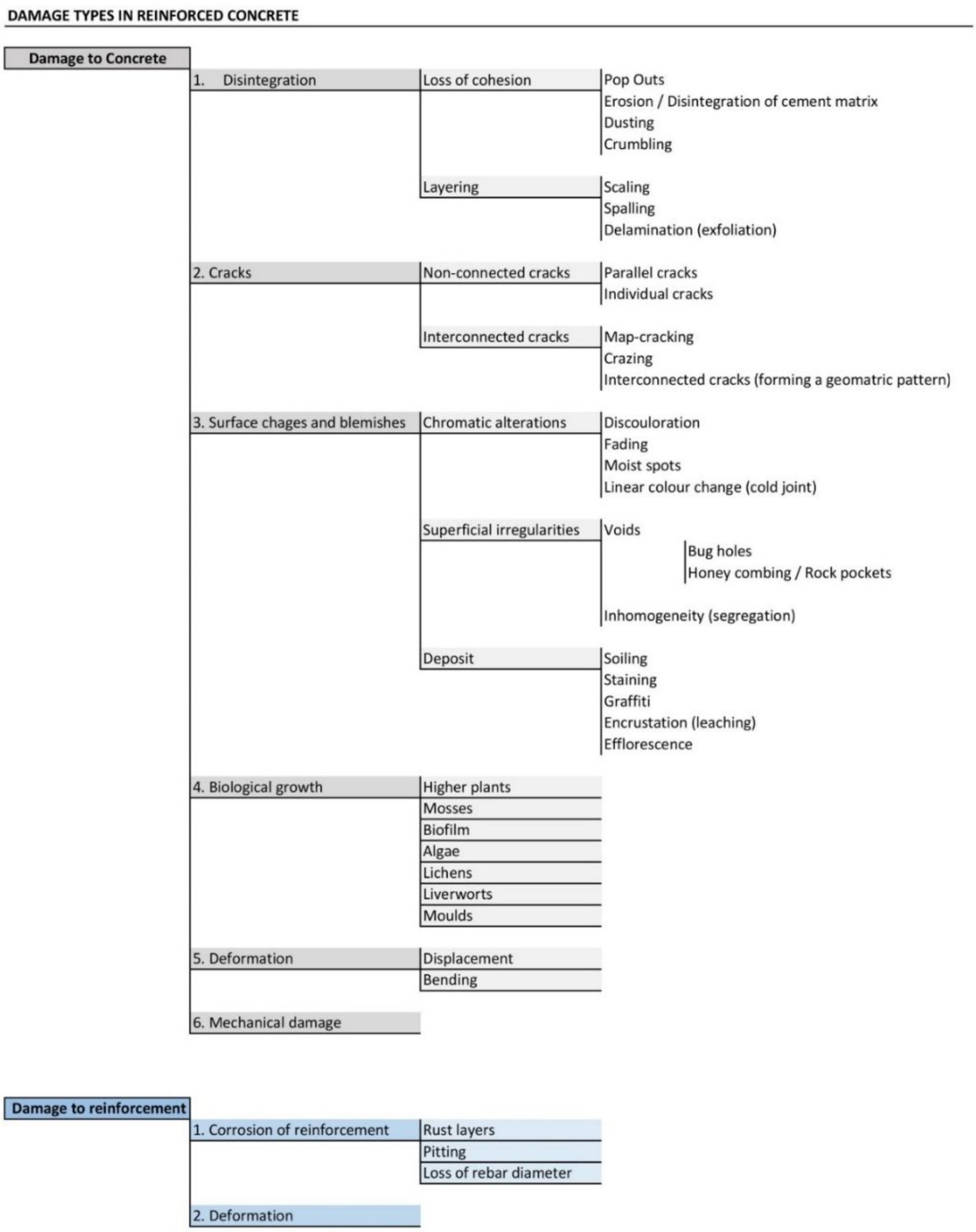

Figure 8 Tree summary of damage types in reinforced concrete

1.3 Remarks on the relation between damage type and damage process

The main goal of this chapter is to link the damage types to damage processes. Once a damage process starts, it is most likely to open way for other damage processes to begin. Identifying this “historic tree of damage” is important for selecting the type of assessment and conservation techniques.

This chapter is to be developed and completed during the project.

2 Survey, Assessment and Monitoring Techniques for Historic Concrete Buildings

The aging of historic concrete buildings and corrosion of reinforcing steel has resulted in an increasing need not only for suitable repair techniques, but also for methodologies and techniques for the assessment and monitoring of these structures.

Two procedures can be established for assessment of historic concrete buildings, a simplified assessment, and a detailed one.

A simplified assessment should be performed in all interventions by professionals (architects and/or engineers) specialized in historic concrete structures. It would involve visual inspections, desktop studies, preliminary calculations, etc. For this phase the use of damage mapping tools, e.g. MDCS[1], and simple semi- and non-destructive evaluation techniques (NDT) are highly recommended.

Depending on its outputs and the scope of the intervention, a detailed assessment may be required (See Figure 7 for flowchart from simplified assessment to a detailed one). The detailed assessment should be carried out by laboratories and companies specialised in destructive and non-destructive evaluation.

The aim in this section is to list current survey and monitoring techniques used for (historic) concrete buildings along with a brief explanation along advantages/disadvantages and limitations.

[1] MDCS – the Monument Diagnosis and Conservation System – is an interactive support tool for the inventory and evaluation of damage to monumental buildings. During visual inspections MCDS helps to identify the types of materials and the types of damage. Based on the damage types found, hypotheses on possible causes are suggested. On the basis of the final diagnosis, conservation can be planned. https://mdcs.monumentenkennis.nl/

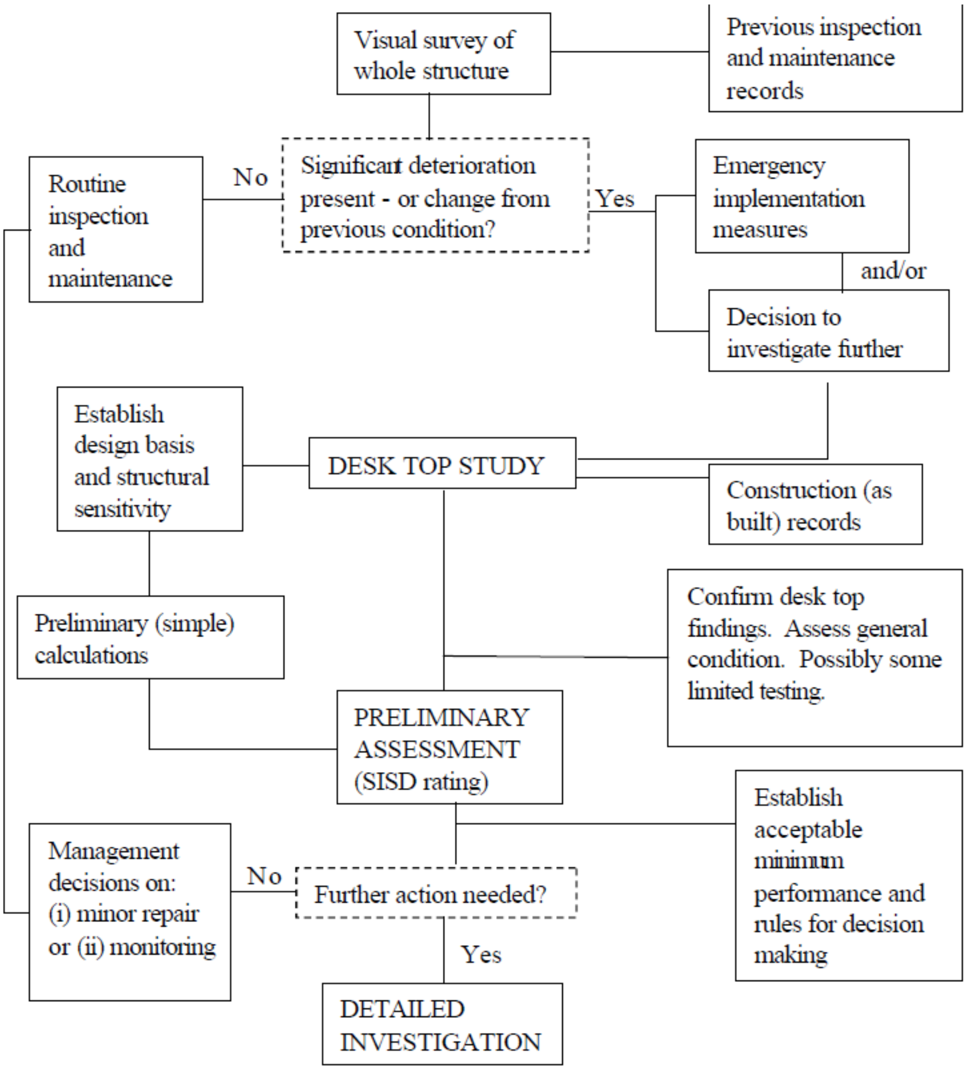

Figure 9. Flowchart from simplified assessment to a detailed one (from (Fagerlund, 2001)*)*

2.1 Survey of geometry

The geometry of the building can provide additional information on eventual structural damage, such as deformations, tilting etc. which can be then further studied with other techniques (see 2.3). Some methods used are:

- *Manual geometric survey*: it can be done by simple tools (tape measure, strings, total stations, etc.) and hand annotations.
- *3D scanning*: nowadays more affordable than in the past. It combines a geometric survey with current deformations and displacements of the structure.
- *Photogrammetry*: rectified and scaled photographs are used to obtain geometric information of the building. Its use is decreasing by the use of 3D scanners.

- *Image processing*: orto-photographs and 3D representations using simple photographs and image software such as *Pix 4D* and the free software *Autodesk 123D Catch*.

2.2 Damage mapping

The techniques for damage mapping are diverse, the most common practice may involve:

- *Visual inspections*: a first visual inspection is recommended to record (hand annotations or other methods) main damages and locations. This can be used to draft first hypothesis and visualize damage patterns.

- *Damage atlas and diagnostic support systems*: software mapping helps to record and manage large quantities of damage types (Figure 8). In the market there are different software for damage mapping. Some of them used by the authors are: *MDCS Atlas*[2] originally developed by TNO and further developed in collaboration with TU Delft, and *T-Pass*[3] developed by the American company Vertical Access.

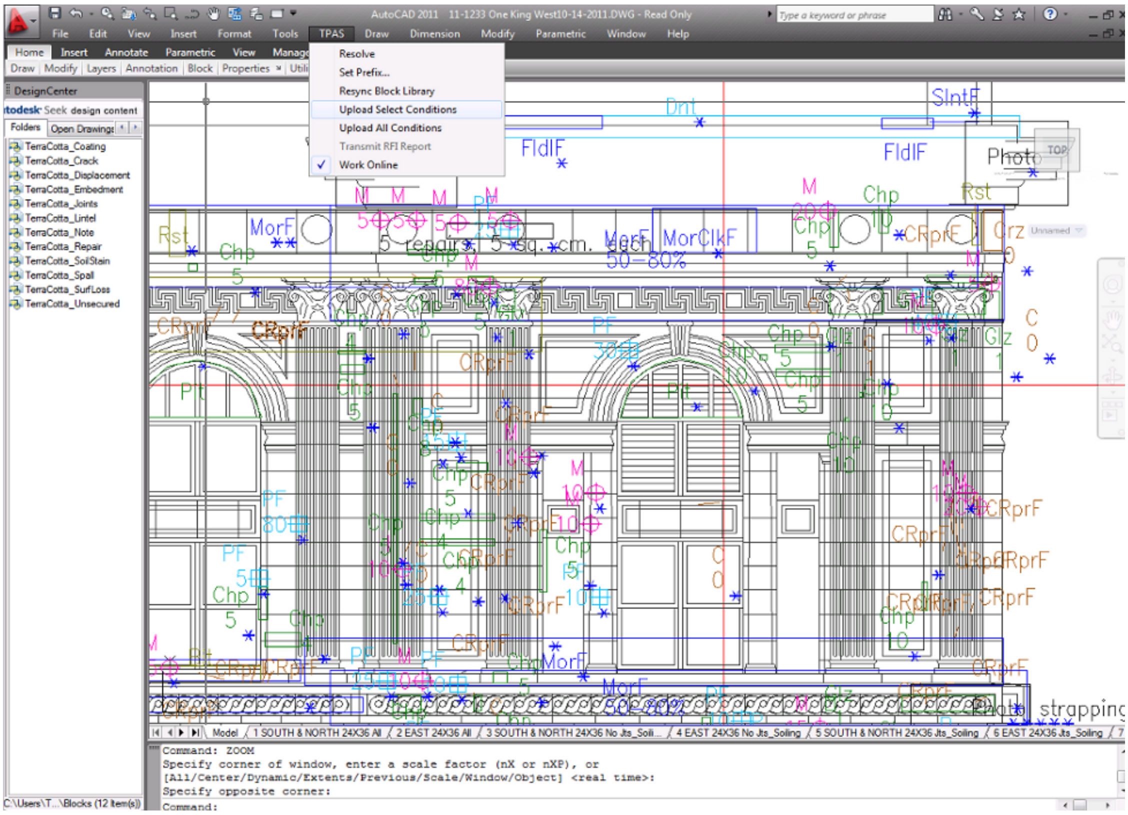

Figure 10. Example of a damage mapping software, T-Pass developed by Vertical Access LLC

[2] https://mdcs.monumentenkennis.nl/
[3] https://vertical-access.com/

- *Optical satellite imaging*: it is a technique used mainly for large historic sites or buildings, to map changes in the surroundings and how can affect specific monuments or sites (Themistocleous *et al.*, 2005). The use in buildings are still in the experimental phases but it can be an interesting tool for assessment in the future.

2.3 Measurement of displacements, deformations, tilts and settlements

Used to monitor new movements in an existing structure. Commonly used methods and techniques are:

- *Tilt meters*: they precisely measure small vertical displacements of walls and columns.

- *Settlement measurements*: typically, markers are installed in the building's concerning areas and they are read periodically by total stations or digital sensors.

- *Plumb line*: an easy way to measure vertical deformations in walls and columns. It is not as precise as tilt meter but the results may be sufficient for certain cases.

- *Inclinometer*: it is used to measure vertical displacements. It can be useful in structures with limited access.

- *Fibre optic sensors*: fibre optic sensors are versatile, they can measure movements in the structure or even moisture content (Aguirre-Guerrero and Mejia, 2018).

2.4 Crack monitoring

Concrete is a brittle material, cracks in tensile zones are inherent to the reinforced concrete and not necessarily a structural concern. Nonetheless, some cracks may be related to a structural misbehaviour or distress. Common techniques and methods to monitor cracks are the following:

- *Linear variable differential transformers* (LVDT): LVDT is an electromechanical sensor used to convert mechanical motion or vibrations, specifically rectilinear motion, into a variable electrical current, voltage or electric signals, and the reverse. LVDTs can measure crack movements precisely. Used mainly in laboratory tests where high accuracy is required.

- *Tell-tale* (crack gages) (Figure 9): crack gages are inexpensive and in some occasions can be a proper way to monitor a structure in combination with other methods.

Figure 11. Tell late or crack gage installed in crack

- *Crack meters*: they are electronic versions of crack gages. Crack meters are connected to digital read-outs to allow continuous monitoring and remote data transfer (Figure 10). Crack meters can also be connected to a more holistic structural health monitoring (SHM) to record other parameters such as humidity, temperature, wind, etc.

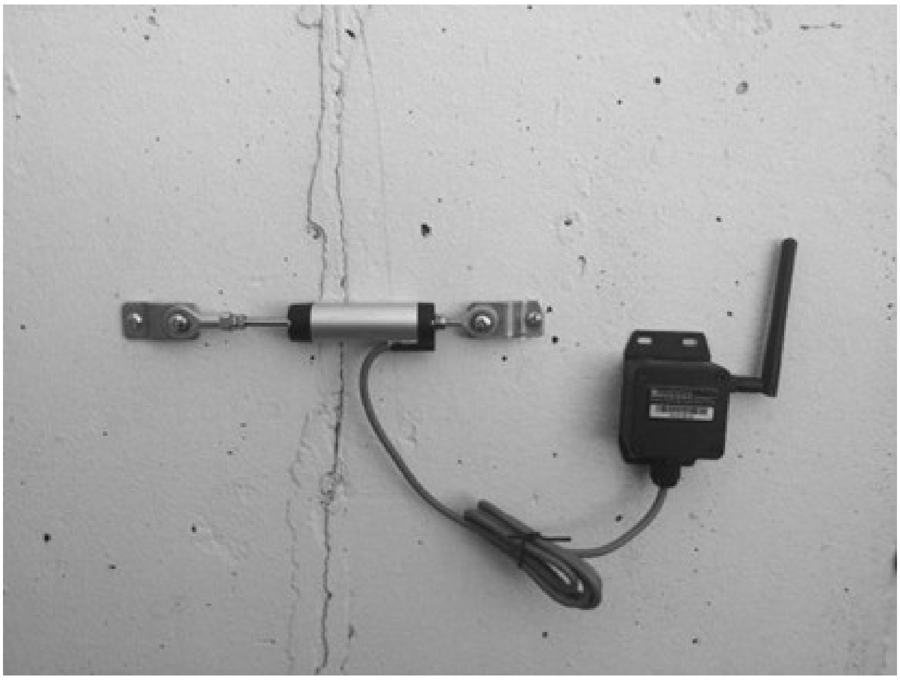

Figure 12. Crack meter installed in crack. Source: Resencys LLC

- *DEMEC strain gage*: it is a mechanical strain gauge developed as an accurate way of taking strain measurements at different points on a structure using a single instrument. It can also be used to monitor cracks in concrete.

2.5 Investigation of Internal structure

There are two main groups of NDT to assess concrete internal structure: the first is based on propagation of acoustic waves, the second on propagation of electromagnetic waves.

It must be taken into account that in the case of historic concrete, as the material might be different from modern concrete, special attention should be paid to the interpretation of the results.

2.5.1 Methods and techniques based on acoustic waves

Acoustic emission (AE) techniques are based on the propagation of acoustic waves in solids while measuring the time taken between a sending and receiving point. It can accurately predict internal cracks if properly calibrated (Ohtsu, Isoda and Tomoda, 2007; Jun Zhou, 2011). Typically they are more accurate when there is access to both sides of the element (Miguel Alcala, 2017).

- *Elastic wave tomography*: tomography techniques are based on elastic waves and acoustic emission (AE) to visualize internal defects in concrete in three-dimensions. The elastic wave velocity is theoretically associated with the elastic modulus of the material: it is quite a recent technology, used not only for the assessment of internal defects in the concrete such as voids, delamination, and cracks, but also for the determination of the modulus of elasticity of the material (Hashimoto et al., 2017).

- *Ultrasonic pulse velocity* (UPV): ultrasonic pulse velocity methods involve propagating ultrasonic waves in solids while measuring the time taken for the waves to propagate between a sending and receiving point. The features of ultra-sonic wave propagation can be used to characterize a material's composition, structure, elastic properties, density and geometry using previously established correlations, known patterns and mathematical relationships. This non-invasive technique is also used to detect and describe flaws in material (cracks, delamination, voids, etc.) as well as their severity of damage by observing the scattering of ultrasonic waves (Helal, Mendis and Sofi, 2015).
  By using the pulse velocity method it is also possible to estimate the strength of concrete test specimens and in-place concrete (Malhotra and Carino, 2003). The code for the standardized ultrasonic pulse velocity tests in concrete are defined in EN 12504-4:2005. It is more efficient if the element under investigation can be accessed on both sides, so that the transducer and the receiver sensor are aligned across the element (direct transmission).

- *Impact-echo*: the idea of using impact to generate a stress pulse is an old idea that has the great advantage of eliminating the need for a bulky transmitting transducer. The stress pulse generated by impact at a point, however, does not have the directionality of a pulse from a large-diameter transducer. The energy propagates into a test object in all directions, and reflections may arrive from many directions (Malhotra and Carino, 2003), making this system difficult to calibrate and interpret.

2.5.2 Electromagnetic waves

- *Infrared thermography*: infrared thermography is a technique and method used to detect infrared energy emitted from an object, convert into temperature, and display an image of temperature distribution. It can be effective in concrete structures to detect superficial defects such as voids, delamination and incipient spalls (Rocha and Póvoas, 2017) (Figure 11). The concrete elements can be artificially heated artificially for best results.

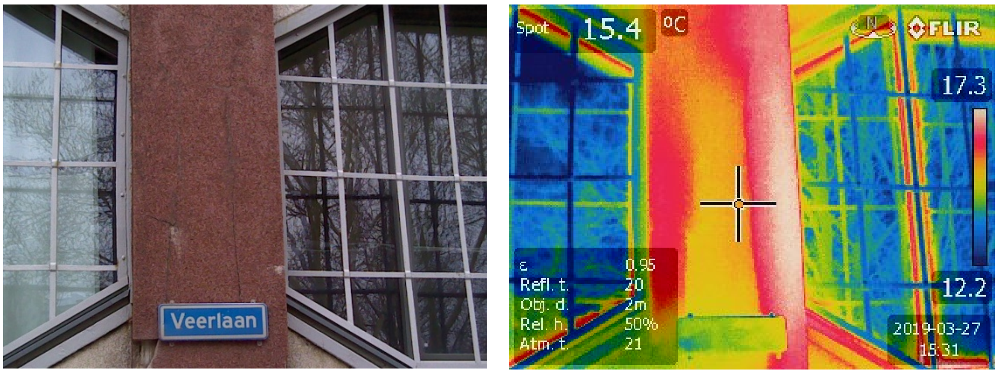

Figure 13. Photograph of a concrete column with vertical cracks and spalling (left). Infrared image of the same picture showing different temperatures in the damaged areas of the column. Fenix Building, Rotterdam (2019)

- *GPR (impulse radar)*: GPR signals are primarily influenced by the dielectric of the material they are probing, and can be useful in locating reinforcement, delamination and voids (Lauer, 2003) (Figure 12). Experimentally, GPR has also been used to measure corrosion in the reinforcement (He *et al.*, 2016; Mamun *et al.*, 2019). GPR is not effective when the humidity content of the concrete is high. The presence of water in concrete leads to modifications of its electromagnetic properties and thus to specific effects upon waveform characteristics (Laurens *et al.*, 2002).

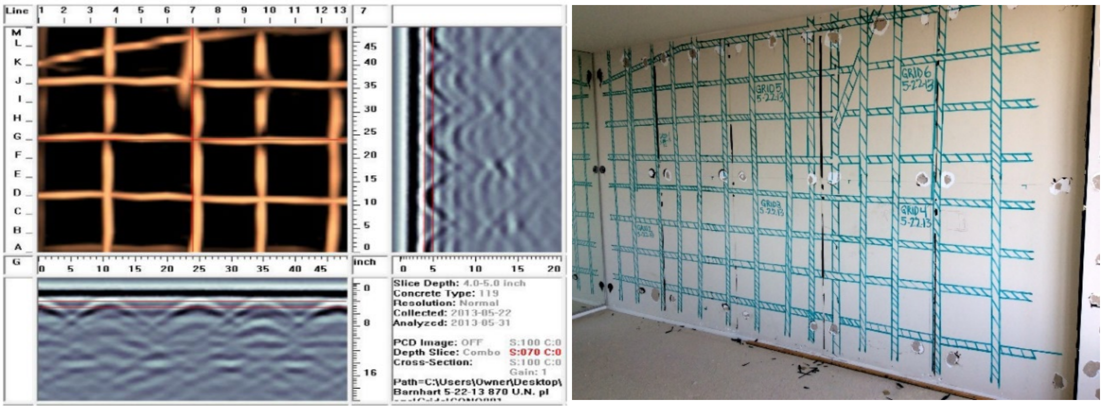

Figure 14. GPR test on existing concrete wall. GPR readings (left) and marked layout of the reinforcement in the concrete (right). UN Plaza building, New York (2013)

2.5.3 Other methods:

- *Endoscopy*: an endoscope, or borescope, is an optical device consisting of a rigid or flexible tube with an eyepiece on one end, and a lens on the other end. A remote object is illuminated by a light source and an internal image formed by the objective lens is relayed to the eyepiece, which magnifies the internal image and presents it to the viewer's eye (Miguel Alcala, 2017).

- *Petrographic and chemical analysis of concrete*: petrography can be used to obtain information about the quality of the concrete, i.e. the original mix design comprising cement type and content, water- cement ratio, degree of compaction and the type and distribution of aggregates. The risk of potential silica-alkali reaction can be assessed based on the type of aggregates. The depth of carbonation, the extent and progress of ASR, and other causes of cement paste deterioration, such as sulphate attack, can also be identified. Specimens for petrography are generally obtained from cores from which thin sections of approximately 25-30μm thickness are prepared and examined using a polarising microscope (Brueckner and Lambert, 2013).

2.6 Characterization of mechanical properties of concrete material

- *Ultrasonic pulse velocity* (See description in section Investigation of Internal structure)

- *Rebound hammer*: it is a NDT used to measure the compressive strength of the concrete. The obtained results are not always accurate (Holčapek, Litoš and Zatloukal, 2014) and some rebound hammers may not work properly for concretes with a compressive strength lower than 20 MPa. For more accurate results the rebound hammer should be combined with ultrasonic pulse velocity measurements; this combination of techniques is known as SONREB and is recommended by RILEM guidelines (RILEM TC, 1993).

- *Sample extraction for mechanical testing*: core extraction for determination of compressive strength and modulus of elasticity is often used. Thin sections of the core can also be prepared for petrographic analysis. The European code regulating the extraction and testing is EN 12504-1:2009.

- *Pull-out test*: it measures the force required to extract a standard steel embedded insert from the concrete surface. Using established correlations, the force required to remove the inserts provides an estimate of concrete strength properties (Helal, Mendis and Sofi, 2015)

- *Pull-off test*: the pull-off test is an in-situ strength assessment of concrete which measures the tensile force required to pull a disc bonded to the concrete surface with an epoxy or polyester resin. The pull-off force indicates the tensile and compressive strength of concrete using established empirical correlation charts (Helal, Mendis and Sofi, 2015).

- *Penetration resistance methods*: penetration resistance methods are semi-destructive procedures that explore the strength properties of concrete using previously established correlations. These methods involve driving probes into concrete samples or elements using a uniform force. Measuring the probe's depth of penetration provides an indication of concrete compressive strength by referring to correlations. The most commonly used penetration resistance method is the Windsor probe system. The system can be used on-site and consists of a powder-actuated gun, which drives hardened allow-steel probes into concrete element while measuring penetration depth (Helal, Mendis and Sofi, 2015).

2.7 Characterization of transport properties of concrete material

The durability of (reinforced) concrete structures is determined to a great extent by the quality of the outer part of concrete, the concrete cover of the reinforcement.

There is no generally accepted method to characterize the pore structure of concrete and to relate it to its durability. However, several experimental investigations have indicated that concrete permeability both with respect to air and to water provides an indication for the resistance of concrete against the ingress of aggressive media in the gaseous or in the liquid state and it is thus a measure of the potential durability of a particular concrete (Ebensperger and Torrent, 2012). Water absorption by capillarity can assess the ingress of water borne substances by capillarity.

Nowadays there are devices that can measure these three parameters (water absorption, air permeability, and water permeability) such as the Autoclam Sorptivity System. These parameters are also used to assess surface treatments after application and to monitor them as part of the maintenance plan.

- Air permeability measurement: air permeability of the concrete can be related to the carbonation rate and the ingress of chloride ions. Generally permeability is measured in laboratory on samples collected on site. The Torrent permeability tester (TPT) has the advantage that can be used in-situ. Although, the test is affected by temperature and moisture content (Jacobs and Hunkeler, 2006); (Ebensperger and Torrent, 2012).

- *Water permeability* measurements: the penetration of water into concrete under a pressure head (permeability) is an important durability performance parameter for concrete exposed to water pressure. The coefficient of water permeability is also a very sensitive indicator of durability for other exposure conditions, as the factors controlling permeability also control other penetration modes (Concrete Institute of Australia. Durability Committee, 2015). Water permeability is important for freeze-thaw deterioration, salt scaling and chloride penetration.

- *Water absorption by capillarity at atmospheric pressure*: the capillary water absorption (total amount and rate of absorption) is related to open porosity, pore size and connectivity of the pores in the concrete. The capillary absorption is strongly related to the intake of water borne salts and other aggressive liquids by capillary suction in building materials. On the other side, it does not seem to be a significant correlation between water absorption and carbonation, and between water absorption and chloride migration (De Schutter and Audenaert, 2004).

- *Moisture content measurement*: moisture can be measured by taking a sample (0.1 g to 5 kg) and measuring its change in weight after drying (Hall and Hoff, 2012). Although there is not an accepted standard to measure moisture content inside the concrete element (the ASTM D4263 test only measures the humidity content in the surface) this is the most accurate method. An alternative, although indicative method, is by measuring electrical resistivity of the concrete (Waters, 1974; Hall and Hoff, 2012). Salt ions in the concrete can affect the values obtained, thus the composition of the concrete must be known beforehand for a reliable estimation (Jacobs and Hunkeler, 2006).

2.8 Environmental and operational conditions

- *Temperature and relative humidity (RH)* (air and surface): by measuring the RH of the environment and the temperature on the concrete surface, the risk of water condensation can be determined. Environments with high RH values can increase the moisture content (MC) of the concrete cover; which in turn increases the corrosion rate.

- *Loading tests*: an option to validate the structural capacity of a structural element is an in-situ load test (American Concrete Institute, 2014). In historic structures, in-situ load testing must be performed carefully to not permanently damage the element. A continuous monitoring of the strain and load is recommended to never reach a failure mode.

- *Accelerometers*: accelerometers can measure the acceleration (small movements) of the structure due to external factors (temperature, wind, vibrations, etc.) with the goal to measure the dynamic response of the building and determine its frequency. The accelerometers are attached to representative parts of the building and stay in place for a determined period of time. Nowadays there are different wireless systems that can easily be installed alone or as part of also as part of the structural health monitoring (SHM) systems (Zhu *et al.*, 2018).

- *Visual vibrometry*: it is an experimental NDT. Visual vibrometry uses ultra-high-speed cameras to visualize and analyse the vibration response of an object or element under environmental loads. When developed for structural elements, it could predict material properties like stiffness, and damping (Davis *et al.*, 2015).

2.9 Evaluation of Corrosion

Deterioration caused by reinforcement corrosion is normally divided into two main time periods**, the initiation period** and **the propagation period** (Figure 13). The initiation period is defined as the time until the reinforcement becomes depassivated, either by the presence of chloride salts or by carbonation. As soon as the concrete at the depth of the reinforcement is carbonated or contains a critical amount of free chlorides, the reinforcement becomes depassivated and corrosion may occur. This limit state defines the beginning of the propagation period. During the propagation period the reinforcement is corroding, which may lead to deterioration of the concrete as well. The formation of expansive corrosion products provokes cracks along the reinforcement, and subsequently, spalling of the concrete cover may occur, as corroded steel can increase between 2 to 6 times the initial volume. Finally, the loss of cross section of the reinforcement may lead to reduction of the load bearing capacity (Markeset and Myrdal, 2008).

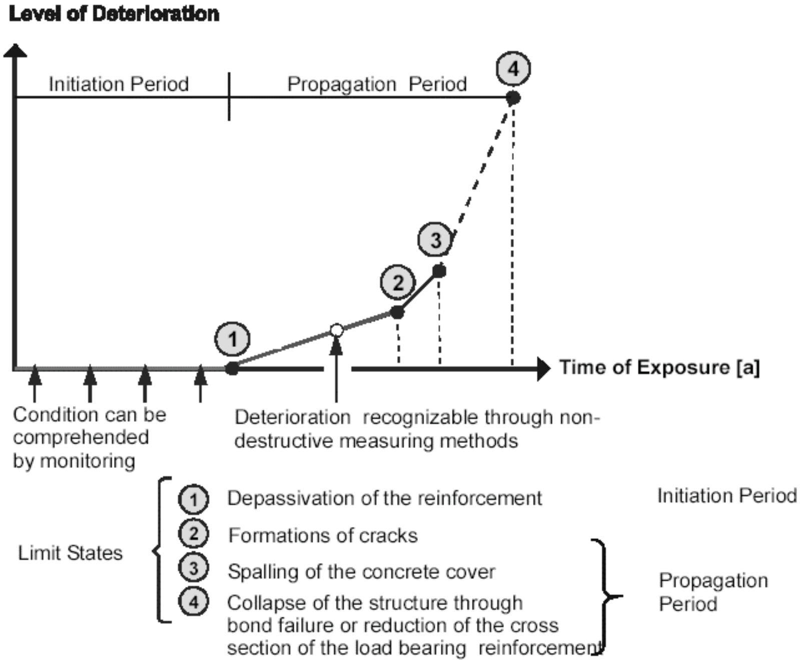

Figure 15. Typical deterioration levels for a steel reinforced concrete structure suffering from corrosion ("Model Code for Service Life Design", Model Code, 2006, Bulletin 34.)

As explained at the beginning of the chapter, the assessment methodology should be framed in two levels: a simplified method and a detailed method. A simplified method would encompass the visual inspection and the desk work; whereas the detailed method would encompass the in-situ testing (Refer to Figure 14).

The European project CONTECVET details a way of how to assess the residual service life of a concrete structure affected by corrosion (Eduardo Torroja Institute and Geocisa, 2001).

If the results of the simplified assessment recommends a detailed assessment, the engineer should establish in each case the extent and detail to be applied in each particular structure. The following parameters are recommended to be verified:

- Reinforcement detailing, including concrete cover thickness, rebar's placement and rebar's cross section.
- Mechanical strength of the concrete.
- Depth of the carbonation front and chloride content
- Corrosion rate and complementary electrochemical parameters: such as resistivity and half-cell potential.
- Yield strength and tensile strength (if possible as this is obtained by a destructive test).

Aside of the above parameters, others may be required such as: w/c ratio, permeability, cement composition, etc. These may be particularly relevant for historic concrete.

	Purposes	*Information needed*
Preliminary visual inspection	Identification of deterioration mechanism.	- Chlorides – Carbonation - Stress Corrosion Cracking
	Mapping of damages.	- Location - Aggressive front - Crack map, spalling, delamination - Section loss.
	Grouping in homogenous lots	- Type of structural element - Environmental aggresssivity - Level of damage.
	Selection sites for testing.	- Lots groups - Deterioration mechanism
Desk work	Collection of background data	- Calculations, structural models - History of events. - Age of the structure
	Exposure classification	- Climatic data. - Environmental actions: rainfall, chloride content, moisture.
	Grouping in lots	- Type of structural element - Environmental aggresssivity - Level of damage.
In situ testing	Testing for assessing	- Carbonation and chloride content - Concrete microstructure - Mechanical strength - Steel yield stress - Corrosion rate - Resistivity - Susceptibility to SCC
	Measurements	- Geometry and dimension of element - Loads on structure. - Rebar detailing - Cover thickness - Section loss

Figure 16. Procedure for corrosion assessment (Eduardo Torroja Institute and Geocisa, 2001)

Electro-chemical parameters (corrosion rate, resistivity and half-cell potential) are the most complex parameters to process since the values can differ greatly depending on different factors. A brief explanation of each technique is listed below.

2.9.1 Corrosion Rate

The corrosion rate gives the quantity of metal that goes into oxides in time (µm/year). The amount of oxides is linked to the cracking of concrete and the loss of steel section and concrete bond.

The most commonly used technique to measure corrosion rate is the polarisation resistance, R_p, which is based on very small polarisations induced in the steel.

Using Faraday's law of electrochemical equivalence, the corrosion rate in terms of amount of steel dissolving and forming hydroxide/oxide may be calculated from the electric current. Assuming that the mass density of iron is 7.87 kg/dm^3, Faraday's formula can be expressed as:

$$V_{corr} = 11.6 \cdot i_{corr}$$ *Equation 7*

Where:
V_{corr} = corrosion rate (μm/year)
i_{corr} = corrosion current density (μA/cm^2)

There are several devices for corrosion rate measurements on the market. It should be noted that different devices may give different values on the same spot (Markeset and Myrdal, 2008). This is due to different principles of measurement and the degree of steel confinement obtained by the instrument during measurement, the corrosion current density can be about five times that measured with a polarisation resistance device (Broomfield, 2007). Even though the relative values may be different for the same element, they can be interpreted similarly because the fluctuations in the readings are likely to happen in the same locations.

In the case a corrosion rate device is not available, the table C.3 in Annex C of the CONTECVET report gives approximate values to estimate the corrosion rate with the values obtained with electrical resistivity and half-cell potential (Eduardo Torroja Institute and Geocisa, 2001).

2.9.2 Assessment of corrosion

2.9.2.1 Electrical resistivity of the concrete

The electrical resistivity of concrete gives information on the concrete water content and its quality. It is well established that there is an inverse relationship between concrete resistivity and corrosion rate (Sadowski, 2013). Therefore, electrical resistivity is a useful complementary technique for locating areas of corrosion and corrosion risk.

In addition, resistivity can provide information about the chloride content and chloride threshold of concrete elements (Morris, Vico and Vázquez, 2004).

Concrete resistivity is commonly measured directly on the surface of the structure by the Wenner technique. It is a NDT that can be used historic and protected buildings.

On the downside, measurement of concrete resistivity is greatly affected by environmental factors such as temperature and humidity, and by nearby metal or embedded steel (Banea, 2015). It is strongly recommended to take resistivity measurements when concrete is in Saturated Surface Dry (SSD) condition (Azarsa and Gupta, 2017).

2.9.2.2 Half Cell Potential

The main objective of potential measurements on a structure is to locate areas in which reinforcement is likely to be depassivated; thus, the steel is able to corrode if appropriate oxygen and moisture conditions occur. The potential is measured by making electrical connection with the reinforcement, therefore breaking the concrete to reach the steel, and placing an electrode on the concrete surface (Eduardo Torroja Institute and Geocisa, 2001).The most common reference electrode used is Cu/$CuSO_4$ reference electrode. This technique is being widely used since it is the only corrosion monitoring technique standardized by ASTM (ASTM C876-15, 2015).

The reading of the half-cell potential can vary depending on the different parameters, such as temperature, relative humidity, carbonation front, concrete cover and moisture content. Several standards have attempted to established orders of magnitude to determine the corroded areas (RILEM TC 154-EMC, 2003; ASTM C876-15, 2015). However, absolute values do not seem to provide a direct relationship with the potential corroded areas. That is the reason why the values must be plotted in mapping diagrams or graphs to be interpreted.

It is however useful to mention the thresholds suggested by (ASTM C876-15, 2015):

- Values lower than -350 mV: 90% probability of corrosion
- Values higher than -200 mV: 90% probability of no corrosion
- values between -200 and -350 mV: uncertain

An important note on these values is that they are not valid *to evaluate reinforcing steel in concrete that has carbonated to the level of the embedded steel*. As in old concrete buildings (> 60 years), the concrete cover is most likely carbonated, these values are not valid in these cases. Besides, absolute values can hardly be taken into account to indicate corrosion hazard, as the relationship between concrete conditions (refer to factors below) and potential values is not well-defined enough. Therefore, calibration has to be made in each structure.

Factors affecting the potential measurement and should be taken into account are:

- *Moisture*. Moisture content in concrete has **a large effect** on the measured potential **leading to more negative values**.
- *Concrete cover*. The potentials become **more positive** with **increasing concrete cover**. **Thin concrete covers** can lead to **more negative potentials** which would seem to indicate high levels of corrosion. **It is advisable to make concrete cover measurements** along with the half-cell measurements.
- *Electrical resistivity of the concrete*. Low electrical resistivity leads to more negative potentials. High electrical resistivity leads to more positive potentials (Azarsa and Gupta, 2017).
- *Temperature*. It influences electrical resistivity of the concrete. Higher temperatures will cause resistivity go lower. A low temperature will cause the resistivity to be higher.
- *Oxygen content.* With decreasing oxygen concentration and increasing pH-value at a steel surface its potential becomes more negative. The values will be highly negative and the half-cell potential reading may lead to misinterpretation.

The mere potential values may therefore be misinterpreted as passive areas when only considering the absolute value of the potential. Locating areas of actively corroding steel is best achieved by considering the spatial variation of potential and not the absolute values of the potentials (RILEM TC 154-EMC, 2003). The potential field measurement alone does not offer quantitative conclusions about the rate of corrosion

As an alternative method to half-cell potentials, AE monitoring could be used. A study points out that AE monitoring could provide earlier warning of the corrosion than the half-cell potentials (Ohtsu, Isoda and Tomoda, 2007). However, there is not enough valid information yet that this method is viable in in-situ conditions.

Other alternative method to half-cell potential is measuring the concrete resistivity (Sadowski, 2013; Azarsa and Gupta, 2017). Though, there is not still an accepted way to accurately predict corrosion with only this method.

2.10 Structural Assessment

This chapter has been developed in collaboration with the University of Cyprus (Antroula Georgiou and Ioannis Ioannou). For seismic assessment please refer to Appendix A.

As per CEB Bulletin 243 (1998), [structural] **assessment** is *"The process of gathering and evaluating information about the form and current condition of a structure or its components, its service environment and general circumstances, whereby its adequacy for future service may be established against specified performance requirements, loadings, durability or other criteria*".

Codes for new construction are appropriate for the design of new-buildings with provisions and features for adequate performance, regular configuration, structural continuity, careful detailing and appropriate material quality. In order to evaluate the performance of existing buildings that were designed without the aforementioned appropriate features (mostly with unfavourable configuration and poor detailing), appropriate codes for existing buildings may be utilised.

The first step to know is what type of new requirement is wanted in the building in order to do a proper structural assessment. For instance, there may be cases where the structure of the building is in appropriate condition and there are local damages due to decay that need to be fixed. In other cases, the building is intended to be upgraded for seismic retrofitting (see Appendix A. Seismic Assessment for more information for further information about this topic). Or the building may need to carry new loads due to a new use of the building (Pardo Redondo, Friedman and De Miguel Alcalá, 2017). Therefore, the structural assessment can be classified in three categories regarding the type of intervention:

- Structural assessment (SA) for local damages: repair of local structural element.

- SA for load capacity: structural analysis to obtain what is the approximate capacity of the building structure to determine

- SA for natural conditions retrofitting (seismic, wind, flood, fire, etc.).

Even though listed historic buildings are often exempt to comply with the same standards as new buildings, this might not be appropriate in the case where public safety is of utmost importance and may thus be prioritized over and above the objectives of historic preservation. Therefore, the best fit solution should be somewhere between legislation for historic and existing buildings and new codes. Attempts are being made, such as the International Existing Building Code (International Code Council., 2017), but still it is a complex task to standardized the assessment and intervention in the wide variety of existing buildings.

In the case of historic buildings, some evaluation and retrofit techniques might not be acceptable, i.e. (a) condition assessment or material testing that would disturb historic elements, (b) potential damage that might otherwise be found acceptable for normal structures, (c) retrofit measures that involve removal of architectural components to gain access to the structure and (d) retrofit measures that permanently alter the external appearance or configuration of the building (ASCE/SEI 41-17, 2017).

3 Conservation Techniques and Materials

The aim of a concrete repair is to maintain the structural capacity, to maintain its function and, if the concrete is visible, to harmonize with the aesthetics for a specific period of time.

The repair should be compatible, which means that it should not cause any damage to the existing fabric and be, at the same time, as durable as possible (van Hees, Lubelli and Nijland, 2014).Compatibility includes aesthetic and technical compatibility.

In this section different conservation techniques and materials used on historic concrete are listed and shortly described; potential advantages and limitations of each technique are discussed.

In the last decades a number of **standards** have been published regarding the repairs in concrete elements. Main examples are:

- American standard ACI 562-16 – Code requirements for assessment, repair and rehabilitation of existing concrete structures and commentary.
- European standard EN 1504:2017 - Products and systems for the protection and repair of concrete structures (Parts 1 – 10).
- British Standard Institution. 2005-2017. BS EN 1504 Products and Systems for the Protection and Repair of Concrete Structures- Definitions- Requirements- Quality Control and Evaluation of Conformity- Parts 1 to 10. London, UK: British Standard Institution.
- REHABCON Manual (2004) – Strategy for maintenance and rehabilitation in concrete structures (Entreprise of the European Commission, 2004).
- CONREPNET project. Thematic network on performance-based remediation of reinforced concrete structures.
- Different RILEM recommendations.
- Australian standard SA HB 84:2018 – Guide to concrete repair and protection.
- New Zealand standard - Historic concrete structures in New Zealand: Overview, Maintenance and Management.

3.1 Concrete repair materials

The European standard (EN1504-10, 2003) subdivides concrete and mortar repair products in two categories: structural and non-structural. Depending on whether applied to restore the structural integrity and durability of the elements or to restore aesthetics and geometry.

Repair materials are generally classified as cementitious materials, polymer modified cementitious materials, and polymer concrete (Jun Zhou, 2011). The difference lies in the type and percentage of the binder used. The properties of these repair materials are strongly affected by the type of binder. For instance, cementitious materials based on Portland cement (OPC) have generally higher shrinkage and lower bond strength than polymer materials.

Aside of the mentioned traditional repair materials, engineered cementitious composite (ECC) materials are becoming a subject of experimentation for repairs thanks to their good mechanical properties and durability (Li, 2004). However, ECC materials tend to have a lower bonding strength with the existing concrete (Lukovic, Ye and Van Breugel, 2012).

The choice of the most suitable repair material for historic concrete, must consider the compatibility with the original material. The repair material should have similar mechanical strength and elasticity than the parent concrete.

The repair materials can be used for different techniques such as patching, concrete recasting, additional concrete cover, and grouting.

3.1.1 Cementitious materials

Cementitious materials are similar to conventional cement mortar and concrete. The main differences are generally a lower water-to-cement ratio and higher percentage of aggregates, meant to reduce the shrinkage of the repair (Jun Zhou, 2011). Often, depending on the application, admixtures are used in the composition, e.g. to improve workability, reduce shrinkage, accelerate development of strength and/or adjust other properties. The American Concrete Repair Guide (ACI 546R-96, 2001) lists 13 different types of cementitious materials depending on the components and admixtures.

The European standard (EN 1504-3, 2001) establishes requirements for structural and non-structural repair products. For instance, for structural repairs, the minimum compressive strength is 25 MPa. However, it should be considered whether this strength is always needed and/or appropriate in the case of repair of historic concrete.

3.1.2 Polymer modified cementitious materials and polymer based material

The polymer modified cementitious materials (PMCM) and polymer based material (PBM) are made by partially or totally replacing the Portland cement, or other cement type, by a polymer, typically an organic polymer (latex or epoxy).

The content of polymer in PMCM is between 10 to 20% by weight of cement. In PC the main binder is polymer, cement may be present as aggregate or filler (ACI 546R-96, 2001).

Polymer repair materials typically have higher compressive and tensile strength, better bonding strength and less shrinkage than cementitious materials. However, polymers are less stable to fire, high temperatures, and UV light than normal concrete (Lukovic, Ye and Van Breugel, 2012).

The replacement of part of the cement by a polymer can improve the bonding strength, mechanical properties, permeability and durability of the concrete; nevertheless, this does not necessarily improve the compatibility of the repair with historic concrete buildings.

3.1.3 Engineered Cementitious Composites (ECC)

Engineered Cementitious Composites (ECC) are a special class of High Performance Fibre Reinforced Cementitious Composites (HPFRCC).

ECC are cement based composites with the addition of fibres that provides tensile capacity to the concrete. ECC exhibits ductile behaviour under uniaxial tensile load, in contrast to the quasi brittle nature of ordinary concrete and fibre reinforced concrete (REF). In plain ordinary concrete, there is no load carrying capacity after the first crack, and cracking immediately leads to failure. In conventional fibre-reinforced cementitious composites (not ECC), matrix cracking is followed by a reduction in load carrying capacity.

In contrast, in ECC, the fibres themselves are able to transfer additional load after the formation of the first crack. On further loading, multiple micro-cracks with crack widths less than 100 µm form along the member, leading to a significant increase in tensile strain capacity. The tensile stress-strain curve hence exhibits a post-cracking hardening branch similar to that of ductile materials such as aluminium.

The most common ECC mixture (also known as M45 in the literature) is PVA fibre-based and possesses compressive strength values similar to moderate to high strength concrete (30-90 MPa), although the compressive strain capacity is nearly 50% higher.

Table 1: Mix proportions of PVA-ECC (kg/m^3)

Cement	Sand	Class F Fly ash	Water	Superplasticizer	PVA Fiber
583	467	700	298	19	26

Table 1. Mix proportions of ECC 45 M45 (Wang and Li, 2006)

Despite the good results, further research is still needed to use ECC as a repair material (Li, 2004; Jian Zhou, 2011; Lukovic, Ye and Van Breugel, 2012)

3.1.4 Comparison between materials

Cementitious materials, excluding ECC, are generally considered the most compatible option when intervening on historic concrete. They can have similar properties (e.g. modulus of elasticity, compressive strength, thermal expansion coefficient and resistivity) to the parent concrete, and can therefore achieve the desired compatibility (Jun Zhou, 2011). In addition, cementitious materials are generally more economical, aesthetically compatible, and stable to UV rays and high temperatures than polymer-based materials.

On the other hand, compared to polymer (modified or based) materials, cementitious materials have the following drawbacks: lower bonding and tensile strength, larger shrinkage (which can cause debonding and cracking), longer curing time, and limited use in aggressive or acid environments, unless the source of damage is removed.

Table 2: Comparative properties of cemenetitious materials and polymer based materals

Repair material / Properties/behavior	**Cementitious materials (CM)**	**Polymer-modified CM and polymer concrete**
Mechanical properties		
Shrinkage		
Comparable properties to substrate		
Curing period		
Cost		
Availability		
Placing and finishing		
Aggressive environment		
Aesthetic aspect		
High temperature		
Durability	?	

Table 2. Comparative properties of cementitious materials and polymer based materials (Lukovic, Ye and Van Breugel, 2012)

3.2 Concrete repair techniques

3.2.1 Patch Repair

Patching entails removing of loose concrete that has cracked, spalled or delaminated, plus any contaminated concrete; preparation of the exposed surfaces; and replacement with a repair mortar (Figure 15 & Figure 16). Patching is the most common method for repairing local damages in concrete, however, still its mechanism and complexity are not fully understood.

In the evaluation of 247 repair cases (35% buildings, 30% bridges, and 35% civil and other structures) in the CONREPNET programme, 20% failed within 5 years, 60% within 10 years and 90% within 25 years (Matthews, 2007). The most common failure modes were cracking, debonding and corrosion. The same report mentions that the main factor of failure was incorrect design of the repairs in terms of materials, preparation of substrate and curing; other relevant factors were poor workmanship, and wrong diagnosis.

In general, for a successful patch repair, the removal of the concrete must extend behind the reinforcement to re-passivate the steel. Also the selected repair material must fulfil two requirements: **compatibility** and sufficient **bond strength** with the parent concrete in order to transfer the required stresses and restore the element (Lukovic, Ye and Van Breugel, 2012).

A more technical definition of compatibility of a patch repair can be defined as the balance of physical, chemical, and electrochemical properties and dimensions between the repair material and existing substrate phase of a repair system, which ensures that the repair can withstand all anticipated stresses due to volume changes and chemical and electrochemical effects, without distress and deterioration over a designated period of time (Emmons, Vaysburd and McDonald, 1993)

Property	Relationship of repair mortar (R) to concrete substrate (S)
Strength in compression, tension and flexure	R ≥ C
Modulus in compression, tension and flexure	R ≈ C
Poisson's ratio	Dependant on modulus and type of repair
Coefficient of thermal expansion	R ≈ C
Adhesion in tension and shear	R ≥ C
Curing and long-term shrinkage	R ≤ C
Strain capacity	R ≥ C
Creep	Dependent on whether creep causes desirable or undesirable effects
Fatigue performance	R ≥ C
Chemical reactivity	Should not promote alkali-aggregate reaction, sulfate attack, or corrosion of reinforcement in the S
Electrochemical stability	Dependant on permeability of R.and chloride ion content of S

Table 3: General requirements of patch repair materials for structural compatibility (Lukovic, Ye and Van Breugel, 2012)

Bond strength relies on two parameters: adhesion force and moisture transport. Both parameters are dependent of a number of factors, to mention a few: the surface preparation (the resulting roughness of the surface after removing the damaged concrete), the temperature, the moisture content of the substrate, the mortar and the environment, the curing process, and the viscosity, contact angle with water, and superficial tension of the repair mortar (Lukovic, Ye and Van Breugel, 2012).

One of the reasons of premature failure of patch repairs lays in the fact that there is not a good understanding of the bonding mechanism and bonding properties at the interface between mortar repair mortar and parent concrete (Lukovic, Ye and Van Breugel, 2012).

In some cases, patching repairs may induce corrosion damage when electrochemical incompatibility exists between new mortar and existing concrete; this causes damage to adjacent, previously sound areas (Zhang and Mailvaganam, 2006). This mechanism is also known as the ring anode effect, in which the initial anode of the damaged area is converted into a cathode. The repaired area is in a more protected environment and the surrounding areas of the patch (up to 20 cm) will become more anodic (Zhang and Mailvaganam, 2006). This problem is particularly found in repairs of chloride-contaminated concrete (Soutsos M , Dyer, 2010). To avoid this problem repair mortars must have similar chemical composition and resistivity than parent concrete.

It is important to assess and monitor the quality of patch repair. To this scope, different techniques can be used. For example acoustic emission (AE) techniques, and pull-off tests when the bond-stress needs to achieve a structural composite action (Kesner, 2017).

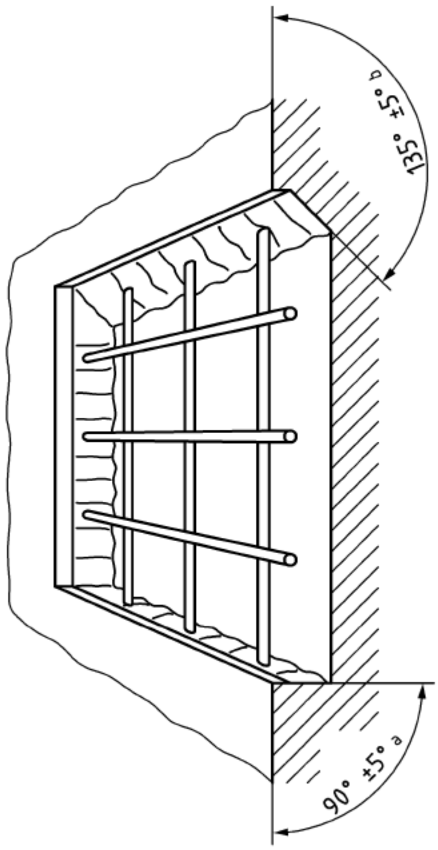

Figure 17. Recommended patch repair according to EN 1504-10. In the image "a" stands for minimum angle and "b" for maximum angle (EN1504-10, 2003)

3.2.2 Concrete recasting

A typical method to restore a structure with ongoing reinforcement corrosion is to remove the old carbonated or chloride infected concrete cover, and replace it by a new concrete cover. If properly executed, it can stop the corrosion until the new cover is carbonated, or chloride infected to an unacceptable degree (Tekniska högskolan i Lund. Division of Building Materials., 2004). It can be said that this follows the same principles of the patch repair but it is used for larger areas.

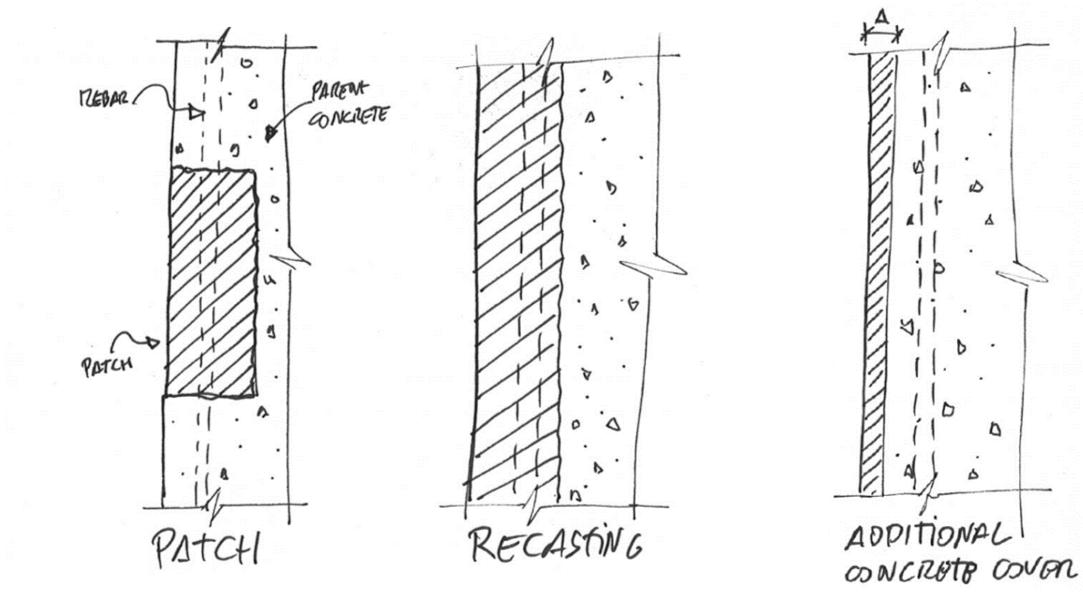

Figure 18. (From left to right) Patch repair, recasting, and additional concrete cover

3.2.3 Additional concrete cover

Another possibility is to apply an additional concrete cover or plaster, above the old concrete without removing this (Figure 17 & Figure 16). The new plaster will diminish the rate of penetration of carbon dioxide, moisture, oxygen and chloride into the old concrete, and therefore reducing the corrosion rate. On the other side, if the new cover does not allow an adequate vapour transport or it has defects, such as cracks and discontinuities, this intervention might increase the moisture content of the concrete, which can have negative consequences as accelerating the rate of corrosion (Figure 18). If the corrosion has not initiated, then the new cover might retard the corrosion (Tekniska högskolan i Lund. Division of Building Materials., 2004)

Figure 19. Radio Kootwijk Station building (1921). The exposed concrete walls and structure was fully plastered to prevent further corrosion

3.2.4 Grouting and Injections (in cracks, voids or interstices)

According to (American Concrete Institute, 2013) grout is a mixture of cementitious materials and water, or other binding medium, with fine aggregate. Grout is proportioned to produce a pourable consistency without segregation of the constituents. It may contain fly ash, slag, and liquid admixtures.

Cracks are inherent to reinforced concrete. They can, however, have a negative effect on the durability and structural capacity of the element. The cause of the cracking can be diverse and must be identified, as treatment methods will vary depending on different actors such as whether the cracks are active or not.

Grouting and grout injection can be classified as the same technique. In basic terms, it consists on drilling holes into the element to access the cracks, voids or leached channels, to pump, with controlled pressure, the fluid grout in order to infill the void space. If the element already have open cracks sufficiently wide, or accessible voids, drills may not be required.

The injections can be used for tighten (water or air tightness), protect (from ingress of aggressive material to the concrete or reinforcement) and/or strengthen; this latter is always used along with a strengthening repair method (REHABCON Annex G, 2000).

According to the EN 1504-5, there are three types of grout or injection material depending on their material composition: hydraulic binders, polymer binders, and gels. The products can also be classified in three categories according to the intended use: force transmitting filling (F), ductile filling (D), and swelling fitted filling (S) (EN 1504-5, 2005).

Figure 20. Fenix II building (1923) in Rotterdam. A new cement plaster was installed in the 1980s and seems to have accelerated the corrosion process

Cement based grouts are the more compatible with historic concrete since it can have similar mechanical, chemical, thermal and conductivity properties. In addition, cement-based grouts are the only ones to protect steel of further corrosion (Tekniska högskolan i Lund. Division of Building Materials., 2004)

Injections are not advisable when the steel reinforcement is corroded and the cracks are caused by the corrosion; it may increase the corrosion rate of the steel due to increase of water content. Injection is neither effective for treating cracks due to ASR.

In general, injection would be required if the environment is aggressive and cracks are wider than 0.2 mm (Tekniska högskolan i Lund. Division of Building Materials., 2004). Micro-cracks can be infilled using polyurethane and polyepoxide compositions (Marukha, 2013), However, two parameters are to be considered when injections are used in historic concrete, one is the compatibility with the parent concrete, and the second is the lack reversibility of this method.

The grouting and injection technique and material are to a certain extent founded on empirical relations and improvisation (REHABCON Annex G, 2000). Therefore after application testing is advisable to ensure the voids and cracks are fully filled.

A recent line of investigation, within the self-healing concretes, is the use of bacteria to infill existing cracks and pores (Larosche *et al.*, 2009; Joshi *et al.*, 2017; Wang *et al.*, 2017). Specific bacteria is introduced into the cracks and/or pores, when in contact with water, the bacteria creates calcium carbonate by metabolic conversion. The calcium product originated is capable to fill cracks and pores and make the concrete element more resistant to the ingress of water (Figure 19).

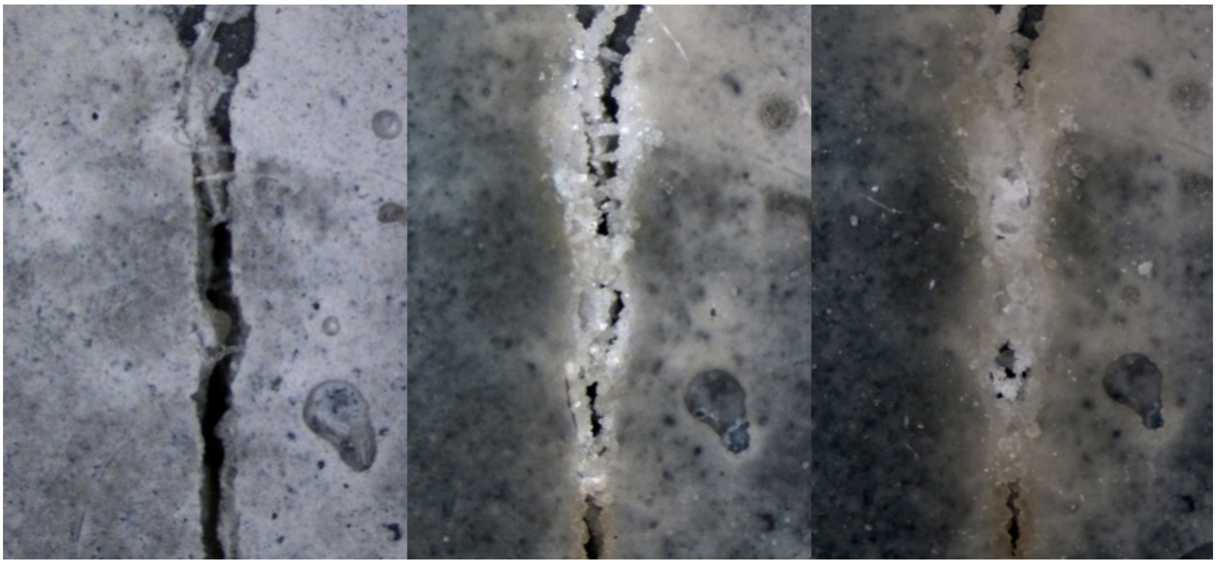

Figure 21. Crack filling with bacteria (TU Delft)

3.3 Surface treatments

Surface treatments include hydrophobic impregnation (water repellents), consolidants, and coatings (anti-graffiti, anti-carbonation, chloride protection, migrating corrosion inhibitors) (EN 1504-2, 2000). The main purpose of the surface treatments is preventing or slowing down the deterioration (e.g. acid attack, weathering, reinforcement corrosion, ASR, etc.).

Surface treatments can also extend the length of the corrosion initiation period by limiting transport of water, chloride, sulphate, acids or some other aggressive compounds. However, for historic concrete structures, the degradation has generally started. Therefore, the efficiency of these treatments are often limited to reduce the rate of degradation.

Occasionally, the requirement set are difficult to achieve with one single product, for this reason surface treatment systems with several products are typically used. The combination of different treatments must be carefully studied for an efficient repair and prevent blister and lack of bonding (Kumagai *et al.*, 2015).

Cleaning by dry (abrasive), wet or chemical methods the concrete surface is often used to restore aesthetical features. However, caution must be considered when using them. Abrasive methods can remove protecting concrete cover and erode the original surface finish. Water-based methods may not be effective, does not seem effective for certain stains, and using chemicals can be too aggressive (Marie-Victoire and Texier, 1999)..

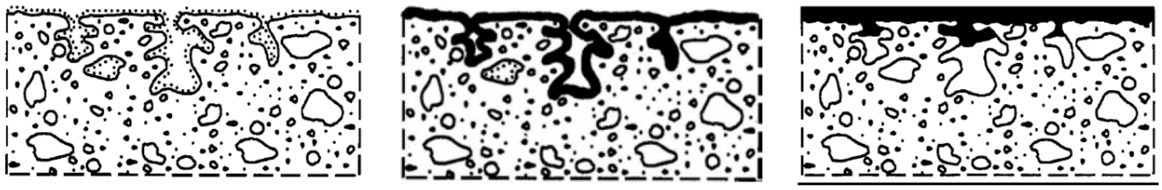

Figure 22. (From left to right) Hydrophobic impregnation, Impregnation, and Coating (UNE EN 1504-2)

3.3.1 Hydrophobic impregnation

Hydrophobic impregnation produces a water-repellent surface; it creates a discontinuous thin film (usually 10 – 100 µm) that partially fills the capillaries.

Hydrophobic agents increase the contact angle between the liquid and pore wall to a value over 90 degrees and prevent consequently the ingress of liquids that occurs by capillary suction. Ingress of water under pressure is still possible, as well as water vapour transport.

The most typical products on the market for hydrophobation of concrete are based on silane or siloxane; after the reaction in the concrete pore system, the end product is silicone. When the silicone fixes to the pore wall it confers water repellent properties to it (Janz, Byfors and Johansson, 2004).

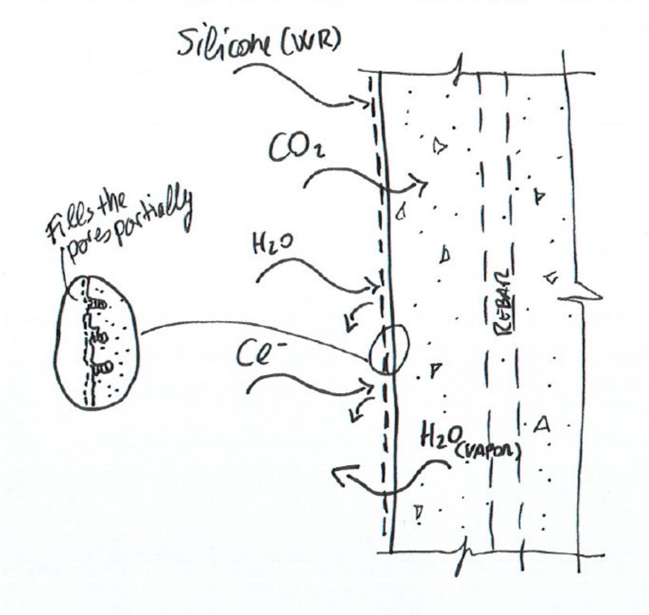

Figure 23. Schematic carbonation process of the concrete

From an environment perspective, the use of volatile organic compounds (VOCs) as solvent in impregnation and coatings has a substantial environmental impact. There is an increasing health and safety, and environmental concerns about emissions from all types of VOCs. Health and safety concerns are centred on issues such as flammability, storage, transportation, and also for the health of individuals where these products are manufactured and used (Matthews *et al.*, 2014).

A more eco-friendly alternative to the use of VOCs is the use of water-based systems (emulsions) in which the main solvent is water. Nonetheless, the information about the performance of the water-based hydrophobic agents is quite limited compared with those of the solvent-based hydrophobic agents (Xue *et al.*, 2017).

Hydrophobic impregnation is effective in preventing chloride penetration and water absorption of new concrete where corrosion has not been initiated. Where corrosion has initiated, as it is typical in historic concrete, it can only lower the corrosion rate.

While stopping or reducing the ingress of water by capillarity, water repellent treatment have generally a moderate to minor effect on water vapour transport. This is especially important for concrete repairs as the drying process is not stopped, although slowed down, and hygric equilibrium with the environment can be achieved.

The durability of these systems can have a life span of decades (Janz, Byfors and Johansson, 2004). Also, in some cases it is favourable to combine impregnations and coatings with hydrophobic impregnation, this latter may improve the bonding strength, keep the concrete cover dry and minimize the risk of water/moisture accumulation beneath the film forming layer.

3.3.2 Consolidation

Consolidation will partially fill the pores in the treated area by impregnation. Because of pore filling, consolidation may provide some resistance to ingress of liquid in concrete, but it does not change the contact angle of water with the pore walls. Liquid water transport is still possible and water vapour transport is generally minimally affected.

Consolidation systems used nowadays are often based on polymers as acrylics and epoxies with low viscosity. In the past also inorganic substances used as consolidants such as water-glass (sodium and potassium silicate) and fluoride compounds and others have been used as hardeners (Janz, Byfors and Johansson, 2004). Consolidants are not typically used alone as protective systems but as primers in combination with coatings. Only a consolidant to protect damaged concrete is often not effective enough (Janz, Byfors and Johansson, 2004).

Preparation of the surface to achieve a rough surface and moisture content (less than 5% EN-1504-10:2017) of the based concrete is key for a successful application (Baltazar *et al.*, 2015). Penetration depths should be at least 2 mm.

3.3.3 Coatings

Coatings form a continuous layer on the concrete surface, thus shielding the concrete surface completely. For effectiveness, this product is to be applied in several layers, and the durability of the protection will depend on the composition and thickness. Coatings are typical between 0.1 to 5 mm thick. The most common coatings are based on epoxy, polyurethane, vinyl, acryl, chlorinated rubber, styrene butadiene, cement and bitumen. Coatings stop the ingress of liquids, but they also stop or reduce significantly the water vapour transport. Moisture present in the concrete would therefore be trapped under the coatings.

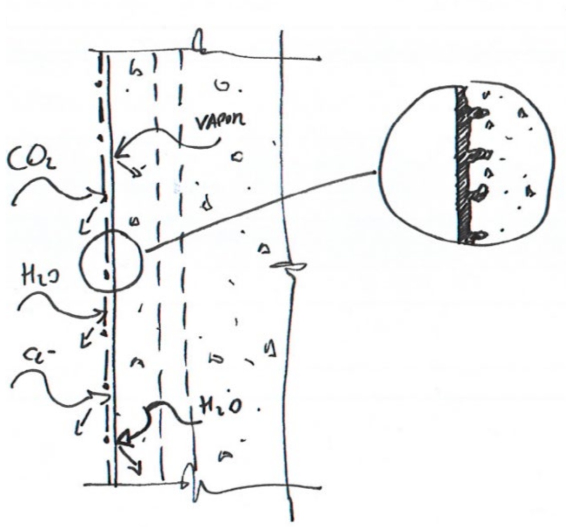

Figure 24. Coating in concrete surface

3.4 Electrochemical Techniques

Electrochemical techniques to protect the steel against corrosion are based on thermodynamics laws of metals in different mediums.

All electrochemical techniques methods have principles and practical details in common. The main difference are the amount of current flowing through the concrete and the duration of the treatments. The driven mechanism for this technique is that by means of an external conductor, called the anode, a direct current is flowing through the concrete to the reinforcement which thereby is made to act as the cathode in an electrochemical cell. The final result of the current flow is to mitigate or stop the corrosion by re-passivation of the rebar due to polarisation of the reinforcement to a more negative potential or by removing the aggressive ions (chloride) from the pores of the concrete or by reinstalling the alkalinity of the pore solution (Entreprise of the European Commission, 2004).

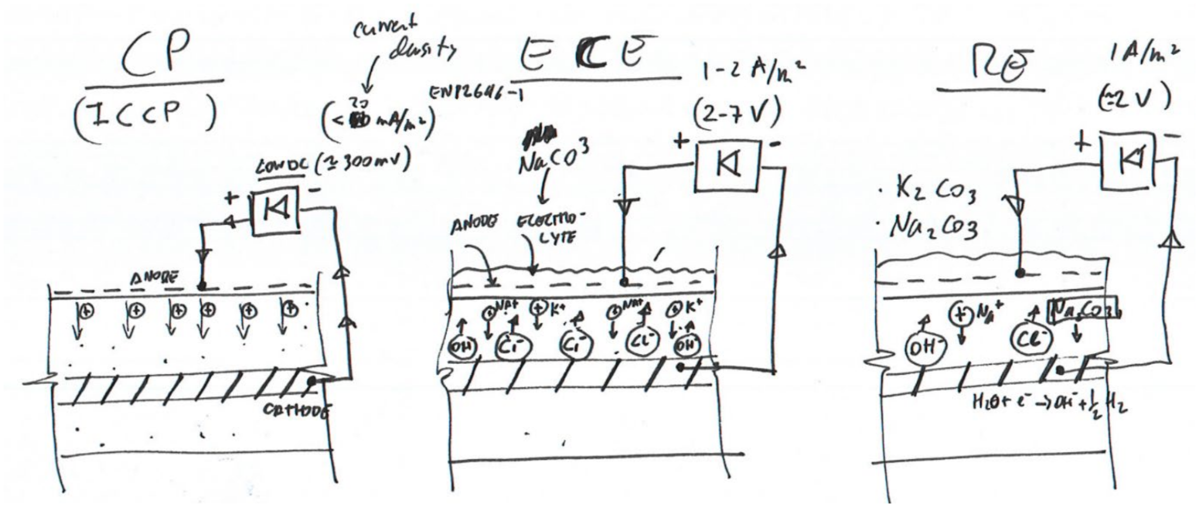

Figure 25. (From left to right) ICCP, ECE and RE schemes

3.4.1 Cathodic protection (CP)

A good illustration to understand how thermodynamics in metals works is the Pourbaix diagrams. Named after Marcel Pourbaix, the scientist who proposed this approach, the thermodynamically stable regions in an electro-chemical system are represented in graphical form (Figure 24). Each metal has three different corrosion stages: passivity, corrosion and immunity. The corrosion stage will depend on the pH of the medium and the potential of the metal. The conclusions by Pourbaix are the basis for cathodic protection.

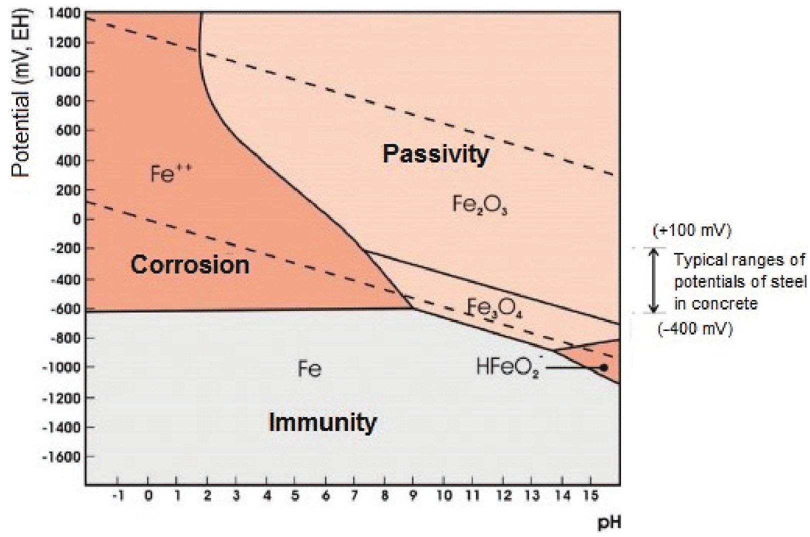

Figure 26. Simplified Pourbaix diagram for iron in water showing the most stable products at a given pH and potential

Cathodic protection is based on shifting the potential of the steel to more negative values, to make the metal work as cathode and so reducing the corrosion current to negligible values.

Cathodic protection has been demonstrated to be successful when appropriately applied in providing cost effective long-term corrosion control for steel in concrete.

There are three main methods: sacrificial anode cathodic protection (SACP), remote sacrificial anode cathodic protection (RSACP), and impressed current cathodic protection (ICCP). All the CP techniques may be suitable for historic reinforced concrete.

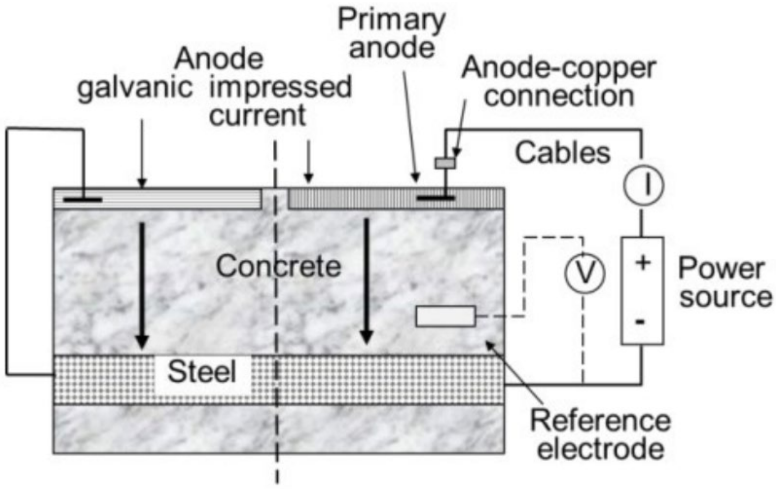

Figure 27. Main components of galvanic protection (left), and impressed current cathodic protection (ICCP) systems, from (Polder and Peelen, 2018)

In case of SACP, a very active metal (anode) is connected to the rebar and placed in the same electrolyte. The anode polarizes the metal to cathodic values (see Figure 24). In the case of ICCP an inert anode is connected to the reinforcement and direct current (DC) is applied to shift the potential to cathodic values (Kuznetsov, 2012). The ICCP is by far the most effective solution in the long term, however the visual impact on the concrete surface may be the greatest. An additional concrete cover, plaster or conductive coating or overlay need to be installed in the concrete surface. In the market there are different ICCP solutions from activated titanium meshes embedded in relative thick mortars, to conductive pastes (with thickness about 2 mm), or titanium-activated cylinders drilled into the concrete (Carmona, Garcés and Climent, 2015).

RSACP was developed in the early 2000s and it combines both of the aforementioned methods. RSACP uses a sacrificial anode but by adding impressed current the sacrificial anode does not require to be close to the element or structure or in other words, the sacrificial anode does not require to be in the same electrolyte (Wood and Farrell, 2019).

Cathodic protection has other beneficial effects, such as that may regenerate the alkalinity at/nearby the surface of the reinforcement. In the concrete, positive ions from the current move in the same direction as the current (from anode to cathode), and negative ions in the opposite directions. Cathodic protection reverses this flow of ions pushing the negative ions away from the steel. This also has a beneficial effect in chloride contaminated concrete, as the chloride ions (Cl^-) will be moved, or in the reduction of the chloride ingress into the concrete. (Kuznetsov, 2012).

Some **negative side-effects** of CP are to be taken into account.

- Embrittlement of steel: The most important potentially deleterious side-effect is the embrittlement of steel by atomic hydrogen generated as a cathodic reaction. This is of outermost importance in high-strength steel used in pre-stressed construction. In alkaline environments (pH>12), with reinforcement cathodically protected, hydrogen evolution can occur only at potential more negative than about -950 mV. Therefore, to avoid the risk of hydrogen embrittlement of the steel, it is recommended to fix a lower limit potential of -900 mV.

  Low yield strength steels ($\sigma < 700$ MPa) utilized for reinforced concrete constructions are almost not susceptible to hydrogen embrittlement, but high yield strength steels and tempered steel, utilized in pre-stressed constructions, the embrittlement must be taken into account (Kuznetsov, 2012).

- ASR: CP may rarely cause alkali-silica reaction in the concrete around the reinforcement due to an increase of alkalinity. The risk exists when relatively high current densities (>10 mA/m^2) are applied and when the conditions necessary for ASR are present (Kuznetsov, 2012).

- Acid attack: Another negative side-effect, is that the acid, produced by the anodic reaction, may reduce the alkalinity in the concrete at the anode/concrete interface. Generally, this acid is minimal and should not present problem for the concrete. However, at very negative potentials (i.e. when high current densities are applied), it might become a problem: the bond between concrete and anode materials might be affected and loss of adhesion between rebar and concrete may occur. The risk is higher for plain bars (Kuznetsov, 2012).

- The use of anodic protection may be not acceptable in many conservation projects because a significant amount of surface material may need to be removed to facilitate embedding the wiring in the concrete (Wood and Farrell, 2019).

When considering the risks of negatives side-effects of CP in the specific case of historic concrete, the following consideration apply.

Historic concrete structures often used plain bars for the reinforcement with lower yield strength, and the aggregates in the concrete mix may contain reactive silica. Pre-stressed concrete is not common in historic structures but it can be found in building structures since the 1940s. Therefore the risk of ASR and debonding should be taken into account when applying CP in historic buildings. Embrittlement and acid attack should also be considered depending on the type of structure.

3.4.2 Electrochemical chloride extraction (ECE)

ECE is a simple process whereby chloride ions are removed from chloride contaminated concrete through ion migration. An anode embedded in an electrolyte media (water retaining substance such as paper fibre wetted with calcium hydroxide or tap water) is attached to the surface of the concrete. The anode is preferably an activated titanium wire mesh. The anode and the rebar (cathode) are connected to the two terminals of a direct current (DC) power supply where the anode is positively charged and the rebar is negative. The chloride ions (Cl^-) will leave the concrete and concentrate around the external anode. Thus the chloride content of the concrete is reduced, particularly around the negatively charged reinforcing steel, where the concrete becomes free of chlorides. Another positive consequence of ECE is that the electrolytic production of hydroxyl ions at the surface of the reinforcing steel results in a high pH being generated around the steel (see realkalization).

ECE uses high current densities (1 to 2 A/m^2) moving relatively large amounts of chloride within a short time (6 to 10 weeks). After this period, the installation can be removed leaving the reinforcing steel in a chloride-free and high-alkaline concrete, resulting a strong re-passivation of the embedded steel (Kuznetsov, 2012).

With this method the original concrete surface is left unchanged after the treatment; ECE may be therefore considered a suitable solution also for buildings with architectural values.

It might happen that the concrete has such a high content of chlorides in the original mixture (e.g. old concrete in which chlorides were used as accelerators, typically in percentage > 0.3% of the binder weight), the chloride content cannot be completely removed in a practical time span. In such cases, other methods should be used, for instance CP to put the steel into a sort of "immunity" state (Figure 24).

The **negative side-effects** of ECE are similar to the ones mentioned for CP. Since ECE creates a fast increase in pH, the risk of ASR needs to be taken into account and thus a petrographic and chemical analysis of the concrete should be performed in advance, in order to judge on the feasibility of the ECE application (Kuznetsov, 2012).

3.4.3 Electrochemical realkalisation (RE)

Realkalisation is a method to stop corrosion of the reinforcement induced by carbonation. Thanks to electrochemical realkalisation, the pH of the concrete around the steel can be increased, and the passivating properties of the concrete pore solution are partially restored. The technique involves passing a current through the concrete to the reinforcement by means of an externally applied anode mesh. This mesh is temporarily attached to the concrete and embedded in an electrolyte (typical sprayed cellulose with 1 molar solution of sodium carbonate). During the treatment, the electrolyte is transported into the carbonated concrete. The dominant ion transport mechanisms may vary, with electro-osmosis and electromigration of ions are the two main contributors. Simultaneously, electrolysis at the steel surface produces a very alkaline environment (Kuznetsov, 2012).

The development of realkalization is schematically shown in Figure 26. The figure shows the movement of ions and during the application of the method

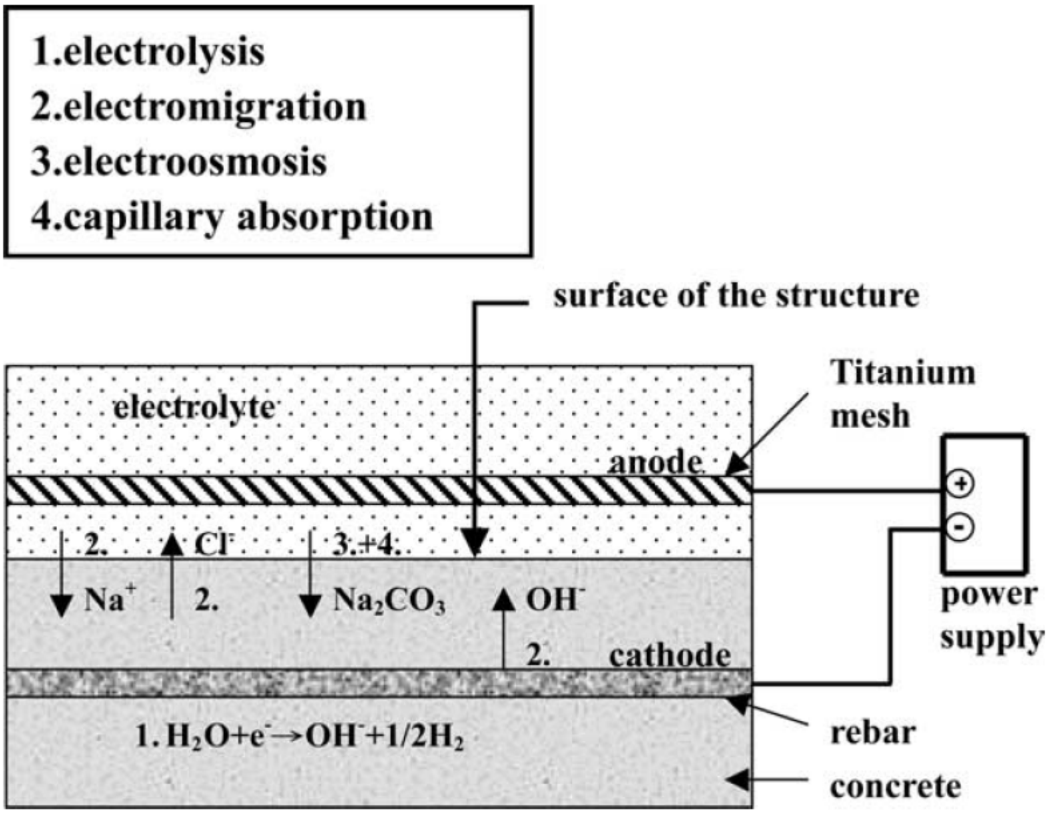

Figure 28. Illustration for the electrochemical realkalisation process (Yeih and Chang, 2005)

Realkalisation usually takes 1 to 2 weeks. It applies a current from 0.8 to 2 A/m^2. Its advantage is that carbonated concrete can be left in place; there is no need to remove and replace it. After the treatment the anode and electrolyte are removed and the concrete surface is left unchanged. Therefore, it may be suitable for historic concrete.

It is to be noted that re-passivation of the steel is not possible, but it is possible to reduce the corrosion rate and prolong the life-span of the structure (Aguirre-Guerrero and Mejia, 2018). In practice, RE is not a common practice in the field of historic buildings and limited information is known about the long-term effects in old concrete.

Also in the case of realkalisation, some **negative side-effects** are present.

- Reduction in mechanical properties: when realkalisation is applied for periods longer than 13 days, it may affect the compressive strength and elastic modulus of the concrete (Yeih and Chang, 2005). During RE, sodium and potassium ions move towards the cathode attacking the CSH gel.

- ASR: for the same reasons and at the conditions already discussed for CP and ECE, a risk of ASR reaction might be present.

- o Increase of corrosion rate: RE drastically changes the composition of the pore solution in the concrete. In **passivated steel**, applying RE may increase the current corrosion rate after the treatment (Aguirre-Guerrero and Mejia, 2018).

- o Debonding: theoretically, due to the increase of alkali concentrations and hydroxyl ions in the pore water at the steel/concrete interface, the risk of loss of bond strength between concrete and reinforcement could be a concern. However, Yeih and Chang demonstrates that *[loss of] bond strength due to the realkalisation although exists but is not a major concern [...]* (Yeih and Chang, 2005). The total charge passing in realkalisation (100 – 200 Ah/m^2) is much lower than chloride removal, which is much lower than the values to reduce bond (Kuznetsov, 2012).

3.5 Strengthening and ductility increase

Strengthening is defined in this case as the method to increase the structural capacity of a structural member or structure. The element to be strengthened does not necessarily presents any damage; however, its structural capacity is not sufficient for the new imposed loads.

The method chosen will depend on the type of forces the existing element must withstand. The most common materials used are concrete, steel (steel frame or rebar), and carbon-fibre plates or sheets.

3.5.1 Strengthening with additional reinforced concrete

This method increases the section of the concrete element by adding a new layer of reinforced concrete. The new concrete layer relies on the bonding strength of the new layer to the existing concrete to develop the desired composite action (Rehabcon, 2000). To this scope, the surface of the existing concrete must be prepared, most often grinded or scarfed, to achieve a proper bonding. Often mechanical connections are needed to improve the bonding strength if the stresses to be transferred are high or the surface cannot be roughened.

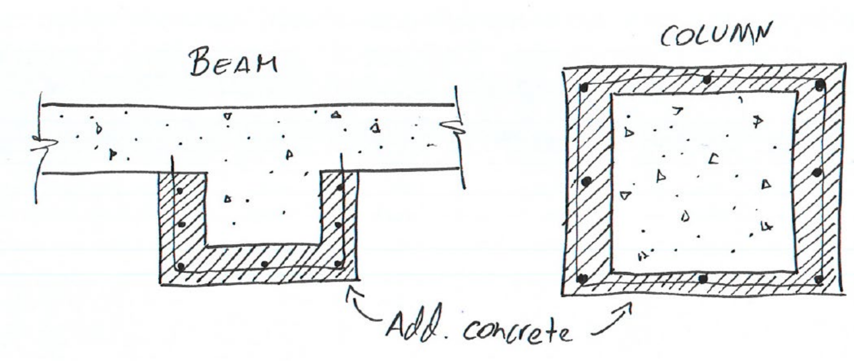

Figure 29. Additional reinforced concrete layer in beam (left) and column (right)

Regular Portland cement concrete can be used for the new layer if there is enough access for pouring and compacting (Figure 28). On other situations, where the access is limited, modified concretes with superplasticizers and additives for self-compaction are used (del Rio Bueno, 2008).

It should be taken into account that this method is not reversible and the new layer will have a considerable thickness (at least 3 to 4 cm), which will affect the aesthetics of the element in the case it has an architectural or aesthetic value.

Figure 30. New reinforced concrete overlay in existing concrete footing. Source: Structural Technologies

3.5.2 Strengthening with added steel

The added steel provides new support and/or bracing to the existing concrete elements.

The new elements can reduce the spans of the horizontal elements by installing intermediate supports, and therefore increasing their capacity, or they can re-support partially or entirely the existing element.

This method is respectful to the original concrete, as it is clearly distinguishable and easily reversible. The new steel elements are not necessarily bonded to the existing concrete, and therefore this method can be considered reversible. The downside is that it carries a considerable visual impact.

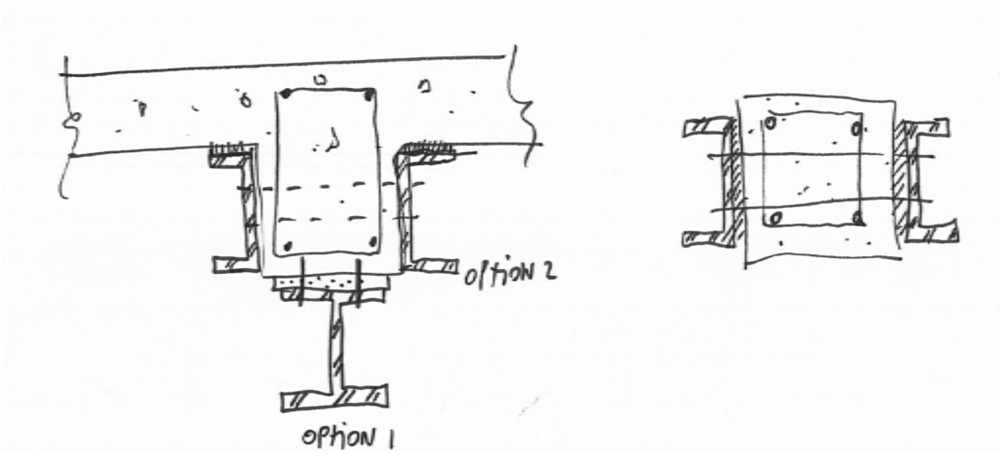

Figure 31. Added steel to resupport (option 1) or reinforce (option 2) a concrete beam (left). Reinforcing of a column with steel channels (right)

3.5.3 Strengthening by using externally bonded plates (steel or carbon fibres)

Strengthening by bonding by using externally bonded plates to the existing elements aim to achieve a composite action of both elements, so that the structural capacity of the original member increases. The bonding between the two elements is key in this system and it must be carefully designed. For the reinforcement both steel plates and carbon-fibre plates or sheets are used.

Figure 32. Wall reinformcement with steel strips (UN Plaza, New York)

The bonding between steel plates and concrete can be done by mechanical connections, typically concrete bolts, or in some occasion by epoxy resins (Daly, 2004). Carbon-fibre elements, due to its fibrous structure and its anisotropic nature, are solely bonded by epoxy resins; using bolts will create undesired effects, such as stress concentrations and tear the new element

From a preservation point of view, some considerations are to be made. Carbon-fibre elements might be not be compatible, from the structural point of view, with historic concrete because of their very high yield strength (up to 10 times that of steel). In the case of historic concrete, and generally for low strength concrete (< 20MPa), reinforcement with steel plates is more suitable. However, steel can be affected by corrosion, while carbon fibres plates are not.

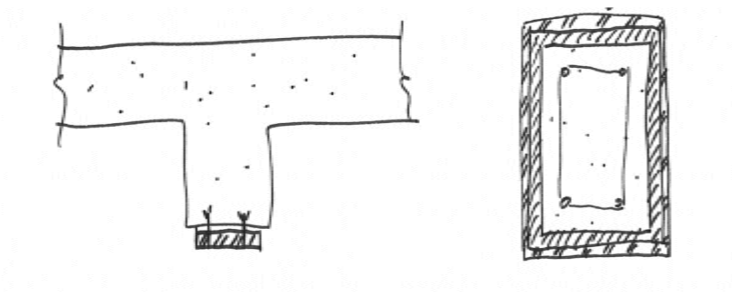

Figure 33. Beam (left) and column (right) reinforcement with added plates

3.5.4 New post-tensioned reinforcement

New post-tension elements, typically steel cables or bars, can be installed in RC members to increase tensile and flexural capacity. The post-tension elements can be installed at the surface (Figure 33) or embedded into the existing elements by notches or trenches that will be patched after installation. The post-tension elements are typically anchored to the concrete elements.

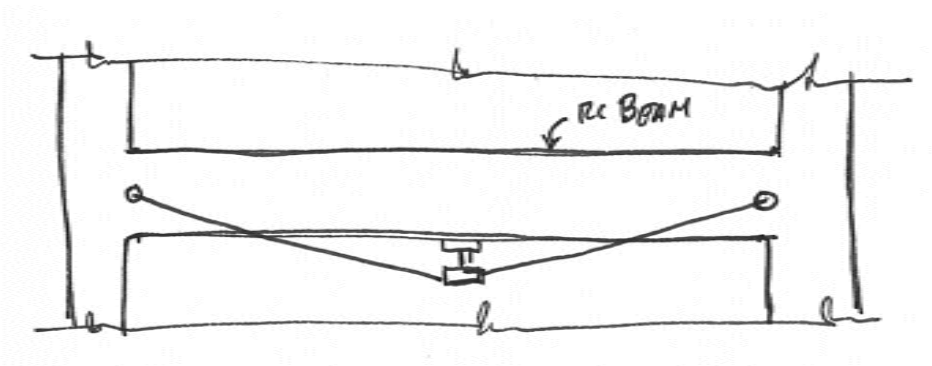

Figure 34. Side view of a concrete beam reinforced with pretension cables supporting an intermediate support

This is a somehow reversible technique, if installed at the surface, and its visual impact can be minimal.

Figure 35. External post-tensioning reinforcement of existing concrete beams. Source: Pullman. A structural technologies company: https://www.pullman-services.com/Services/structural-strengthening/external-post-tensioning-2/

3.5.5 Ductility increase

Ductility and deformability are interrelated concepts signifying the ability of a structure to sustain large deformations without collapse (Raghucharan and Prasad, 2015). In other words, a ductile material is the one that can undergo large strains while resisting loads.

Ductility and strength of a building structure are essential to safely withstand earthquake loads. For instance low-ductile structure will collapse easier under seismic load than a more ductile structure.

Historic concrete structures were, in most cases, not designed to withstand seismic events (Zeinoddini and Dabiri, 2013). Increasing the ductility of a (historic) building structures improves their resistance to earthquakes (See Appendix A. Seismic Assessment for more information).

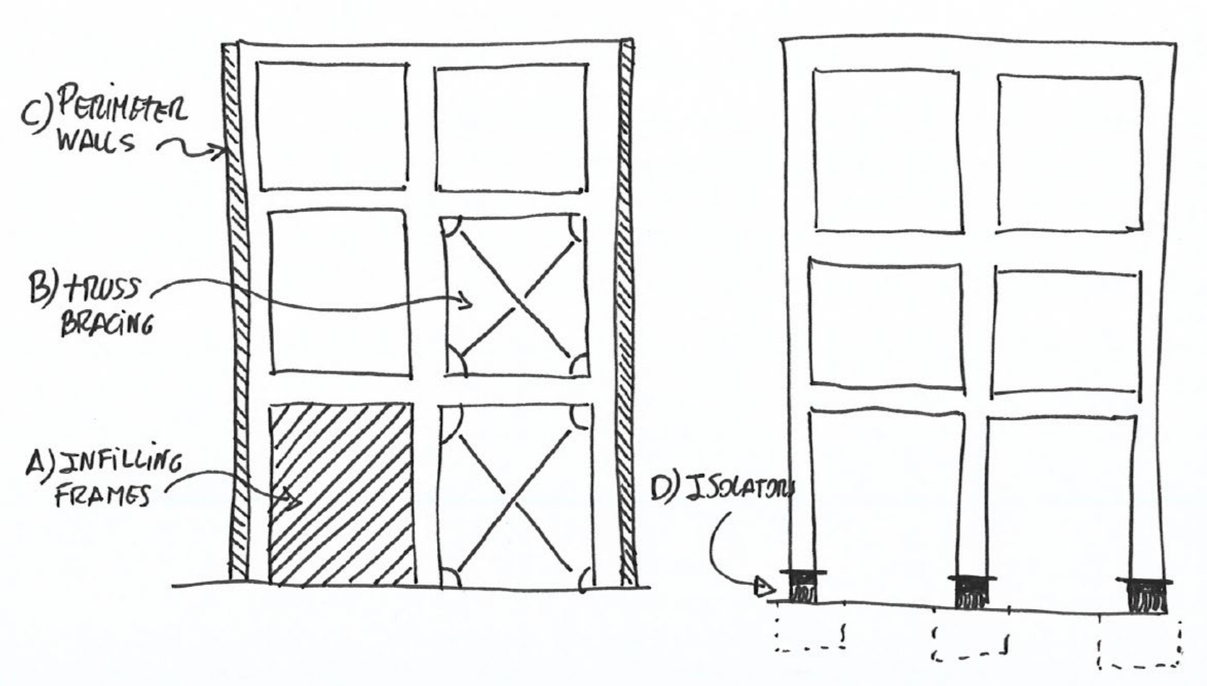

Figure 36. Different methods to increase ductility in a concrete building structure

It may require strengthening existing elements (see techniques listed above) and/or adding new structural elements. The most common used additional structural elements are:

- Infilling frames (with cast in situ concrete, precast concrete panels, steel panels, concrete blocks, or brick infill).
- Truss bracing in frames (tension and/or compression braces, steel concrete post tensioning cables).
- Perimeter walls (cast-in-situ concrete, precast concrete panels).
- Modification of the element by selective material removal from the existing element (Federal Emergency Management Agency (FEMA), 1997).
- Use of seismic isolators in the base of the building (Clemente and Martelli, 2019).
- Reinforcement of column-beam joints.

Ductility increase requires a global investigation of the building structure and other non-structural elements (e.g. facades, interior walls, etc.) and accurate structure analysis performed (ACI Committee 562, 2016).

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Appendix A. Seismic Assessment

In fact, in Eurocode 8 – Part 3: Assessment and retrofitting of buildings (EN 1998:3, 2005), it is stated that "*Although assessment and retrofitting of existing structures for non-seismic actions is not yet covered by the relevant material dependent Eurocodes, this part of Eurocode 8 was specifically developed due to the many older structures constructed without seismic provisions and the damage caused by earthquakes*".

In recognition of the high risk of damage in existing substandard buildings, standards for structural assessment and upgrading have been developed worldwide, namely: Part 3: *Strengthening and Repair of Buildings* of Eurocode 8 (EN 1998:3, 2005), ASCE standard on *Seismic evaluation of existing buildings* (2001) (ASCE/SEI 41-17, 2017), Guidelines for *Assessment of Existing Concrete Structures*, Japan (Japan Concrete Institute, 2014), Task Group 7.1 *Seismic Assessment and retrofit of existing structures* of ***fib*** Commission 7 *Seismic design* (fib Bulletin No. 24, 2003), the New Zealand *Seismic Assessment of Existing Buildings Guidelines* (NZSEE, 2017), 1997 NHERP *Guidelines for rehabilitation of existing buildings* (FEMA 273, 1997), (FEMA P154, 2015), (ASCE/SEI 31-03, 2004).

Considerations

As earlier mentioned, the seismic evaluation and retrofit of historic structures should be based on a performance-based design methodology and not on the seismic design procedures described in the building codes and standards of new structures. Evaluation of any existing building may follow three levels of escalating degree of complexity, as specified in (ASCE/SEI 41-17, 2017), (JBDPA, 2005): (1) Screening procedure, (2) Assessment based on identified deficiencies, and (3) Systematic evaluation procedure.

It is important to identify the structural and non-structural elements that participate in resisting the seismic forces. This information assists the structural analysis in recognizing potential seismic deficiencies such as load path discontinuities, weak links, irregularities, and inadequate strength and deformations.

In historic structures it is important to identify the location of historically significant features and fabric, so that care is taken in the design and investigation process to minimize the effect of intervention on these features.

Seismic assessment procedures (linear and non-linear)

As a first step for the assessment most of the National Authorities issue Rapid Assessment Documents that can identify possible global defects of the structure and possible collapse during future seismic events (NRC CNRC, 1993; JBDPA, 2005; FEMA P154, 2015; NZSEE, 2017). Some of the critical structural weaknesses that are usually observed in old type constructions are: (a) irregularities in plan (T-, L-, U- or E-shaped plan, etc.), (b) irregularities in elevation, (c) short columns, (d) site and soil issues and foundation (potential for landslide, liquefaction etc.) and pounding with adjacent buildings. Other than the seismic action that depends on the location of the structure and the geological information of the site and foundation details, adjacent building development and grading activities are also of crucial importance. Poor foundation can lead to settlement of floor slabs and foundations and visible differential movement. Adjacent buildings may influence structural integrity due to pounding, especially if the floor and roof levels of the adjacent buildings do not align in height. Additional factors that pertain to the long term effects of ageing and corrosion induced damage need be evaluated also, as they may prove to be the controlling parameters of the structural condition.

For the more detailed analysis of the structure, two parameters are required: (i) the possible seismic excitation and loading of the structure that is site-specific and is given in terms of either acceleration or displacement, based on the elastic spectra, and (ii) the model of the structure that can be either linear or non-linear. The possible analysis procedures given by most of the assessment codes are (a) linear static, (b) linear response spectrum (modal), (c) nonlinear static (pushover), (d) nonlinear dynamic (time-history), (e) q-factor. Some of these methods take into consideration the maximum modal response, ignoring the duration of the phenomena and combine modal maximum results irrelevant to the inelastic behaviour (Priestley, 2003). Since the damage is related to strains and displacements and not accelerations new approaches have been developed: (Moehle, 1992) and (Priestley, 2003), (NZSEE, 2017), (FEMA 273, 1997), (EN 1998:3, 2005). These perform displacement based assessment, as the earthquake does not represent a set of given lateral loads, but rather a demand of accumulation of energy or dynamic ground displacement.

Another parameter of variability that is not taken into consideration by the Codes for Assessment is the variability in stiffness characteristics of the structure that are constantly being modified during the excitation, as soon as cracking of the structure takes place. Having therefore an elastic spectral acceleration and displacement over the entire range of the response does not take into consideration damping phenomena that are generated from crack formation and energy dissipation (Priestley, 2003).

Strength & deformation capacity of non-seismically detailed components

The structural assessment of buildings requires good understanding of the various components of the structure, such as beams, columns and diaphragms, as well as their interconnection and materials' mechanical properties, such as strength, deformability and toughness. The existing components' strengths are determined in order to calculate their ability to deliver loads to other components, and their capacity to resist forces and deformations.

Reports from destructive earthquakes worldwide have shown some common typical details of structures that collapsed, such as: use of low reinforcement ratios, insufficient details for anchorage and confinement particularly at exterior beam column joints, captive column arrangements due to incomplete masonry infills, low material quality, smooth bars, insufficient lap-splice lengths, discontinuous reinforcement, design based on allowable stresses and not on capacity based hierarchy, compounded effects due to corrosion of steel (carbonates, chlorides) and cracking/deterioration of concrete from exposure to climatic changes (fib Bulletin No. 24, 2003). Additionally, material characteristics of older type construction show great variability due to in-situ concrete mixing, poor quality concrete raw materials, low content of cement in the mix design, leaching of cement due to ageing and carbonation, and poor compaction.

In older type construction, shear reinforcement was used only for supporting the longitudinal reinforcement against buckling, to resist a small fraction of the design shear force due to vertical loads and for torsion (fib Bulletin No. 24, 2003), although this is not always the case and varies depending on the country. Confinement especially in the case of the reversed cyclic loading of seismic events can be used as a means of providing ductility and lack of it can result in brittle types of failures at low ductility levels. The calculation of the strength and deformation capacity of old type members is still being investigated in many research centres worldwide (Park and Pauley, 1975), (Lehman, Calderone and Moehle, 1998), (Panagiotakos and Fardis, 2001), (Greek Organisation for Seismic Planning and Protection (OASP), 2013), (FEMA 273, 1997), (Priestley *et al.*, 1996), (Inel and Aschheim, 2004). Usually the various models adopt a lumped inelasticity cantilever with a tip load and a length of L_s (=shear span) with a plastic hinge length l_p.

Additionally, curvatures φ, drifts θ and displacements Δ of the members at yield and ultimate, may be used to calculate deformation capacities, ductility and local damage of the components. The above parameters are used to assess the hierarchy of damage on the local members of the structure and assist in guiding the repair and retrofit strategy. Those parameters are also used in the nonlinear analysis of the structure.

While the moment-curvature analysis would imply a ductile performance of the members, the actual amount of displacement is a matter of failure prevailing mechanisms between flexure, shear, lap-splice, buckling, beam-column joint failure mechanisms (fib Bulletin No. 24, 2003; Syntzirma and Pantazopoulou, 2007; Pardalopoulos, Thermou and Pantazopoulou, 2013). Especially in the case of shear mechanisms, shear resistance breaks down with damage accumulation and a variety of models have been proposed to calculate the degradation of shear resistance as a function of deformation ((Ma, Xiao and Li, 2000), (Priestley, Seible and Calvi, 1996), (Lehman, Calderone and Moehle, 1998), (Martin-Perez and Pantazopoulou, 1998), (FEMA 273, 1997), (Moehle, Elwood and Sezen H, 2002), (Kowalsky and Nigel Priestley, 2000),(EN 1998:3, 2005)). Lap-splice failure is also considered as a systematic type of failure in older type constructions, due to the splicing of bars within the plastic hinge formation zones without the use of stirrups, the use of smooth bars with low bond strengths and the use of short embedment or lap-splices. Models by Priestley et al. (1996) and (Lynn et al. 1998) may be used to calculate the strength of members with lapped reinforcement.

In the case of old type construction, monolithic beam-column joints were not designed for shear actions and most of the failure of exterior joints observed are related to joint failures, which in addition have smooth bars and no confinement by stirrups. These are of crucial importance as they serve the load paths of the structure, transferring (shear, moment and axial loads) loads from the beams to the columns, and of the overall frame action. Joint shear resistance may be calculated by empiric l models derived from experimental results (Priestley, Seible and Calvi, 1996), (California Department of Transportation, 2010)), (BS EN 1998:3, 2005), (Pantazopoulou and Bonacci, 1992, 1994), (FEMA 273, 1997).

Modelling techniques

Modelling of the structure must include all elements (primary and secondary if detrimental) and take into consideration accidental torsional effects using the mean values of the material properties. The seismic action used in linear analysis (static and modal) must be the one corresponding to the elastic response spectrum without any decrease by the q-factor (EN 1998:3, 2005). Analysis may be performed in 2-D frames or 3D models, if torsional response should be included. Design codes tend to become more sophisticated specifying 3-D modal analysis as routine design tool. The advantage of this analysis is the improved representation of the higher modes of vibration in regards to static analysis, while its drawbacks are driving the design to unrealistic estimates of member stiffness, doubtful validity of results from modal combinations, underestimation of drifts in lower stories and no consideration of the alteration of stiffness due to the axial force variation caused by the seismic axial load (Priestley, 2003).

In the case of force-based methods that use the fundamental period and shape mode to determine the distribution of forces through the structures, the use of elastic stiffness characteristics results to errors, which may even be worst in the case of inelastic response taken that with force based analysis some members remain elastic while others are ductile (Priestley, 2003). Some design codes do not even take into consideration the early cracking introduced by the seismic excitation using the gross section stiffness of the members. A number of codes use a general decrease factor of 0.5, while others use different adjustment factors for different member types (Greek Organisation for Seismic Planning and Protection (OASP), 2013; NZSEE, 2017).

In the case of the non-linear static (pushover) analysis, Eurocode 8 (EN 1998:3, 2005) requires the use of at least two vertical pattern distributions (uniform and modal) to be applied at the location of masses in the model. The model is loaded with vertical (G+0.3Q) and increasing lateral loads and the capacity curve is determined. The elastic spectrum is then used to derive the target displacement (Informative Annex B, (EN1998-1-2004, 2004)).