1. INTRODUCTION

Since the beginning of the 20th century concrete, specifically reinforced concrete, has been the main building material for the majority of constructions. In most cases, the historical value of these buildings of the previous century has been acknowledged, and attempts for their preservation and rehabilitation have started to be performed.

Presenting advantages such as workability, flexibility in construction and durability, concrete has been considered as a suitable material for construction. However, by taking several buildings as examples, it can be shown that concrete can be sensitive to various deterioration modes. In addition to this, most of the deterioration modes are time-dependent, and this could strongly affect the durability and service life of concrete buildings.

In terms of assessment and rehabilitation of historic concrete structures, a first step could be the study of the material and its common decay mechanisms. This could be quite helpful to recognize any possible damage on structures and to identify the reasons of its appearance.

The knowledge about concrete as a building material can be used during in-situ investigations. The purpose of the investigations is to assess the condition of structures and get information about their construction. In cases of historic structures, the investigation should be based on non-destructive tests (NDT) to avoid causing any damage. However, since the information obtained through NDT is most of the times not sufficient, as it can be characterized as qualitative, minor-destructive testing (MDT) may also need to be applied, providing that it will not cause significant and irreversible damage to the structures.

Modelling of structures can be very useful to study their condition and structural behaviour. The models should be an effective representation of the structure, in order to present the real situation as much as possible. The models can be used for analysis purposes. The results of the analyses can be used to assess the condition and capacity of the structures and contribute to any decisions about rehabilitation.

The objective of this study is to attempt a further understanding on historic reinforced concrete structures through the above described methods. The study is based on literature information, in-situ investigation on two historic reinforced concrete buildings and modelling.
2. HISTORICAL EVOLUTION OF CONCRETE AS A BUILDING MATERIAL

The term concrete derives from the latin word *concretus*, which means grown together, compounded, hardened, and can be considered as an artificial equivalent of natural stone. Concrete consists of an accumulation of particles held together by some cementitious substance and consolidated under more or less heavy pressure. The precise antiquity of concrete cannot be stated. However, it is interesting to mention that parts of concrete buildings have been discovered in Mexico and Peru, dating back to prehistoric times. A fresco in the Temple of Ammon at Thebes, depicting hieroglyphically the making and use of concrete in 1950 B.C., can show that the ancient Egyptians were comprehensively familiar with the material. (W. N. Twelvetrees, 2008)

There are records of concrete being used in Roman times, in Mediaeval Europe and in eighteenth and nineteenth century Britain and France. Since about 1900 concrete has become a principle building material in all the world.

All the historical concrete versions were extremely different in composition, properties, as well as strength and durability. Much Roman concrete presented similarities to modern one, as it could set under water. The concrete from the Middle Ages was rather weak and contained lime. In the mid-eighteenth century Roman hydraulic cements that could fully cure under water were rediscovered by Smeaton.

Today’s concrete is reinforced concrete. The reinforcement is applied by steel bars carefully placed in order to carry the tensile and shearing stresses in structural elements. The properties of reinforced concrete are far more complex than the ones of concrete, since they depend on not only the properties of both materials, steel and concrete, but also on the effective load distribution between steel bars and concrete. The combination of the high tensile strength of steel and the compressive strength of concrete, as well as the ability to cast shapes of extraordinary variety and complexity, has led to a remarkable change in architecture since the last decade of nineteenth century, when it was first reclaimed. (S. MacDonald, 2002)

The reinforcement was placed inside concrete for the first time in France, by J.L. Lambot for a boat construction in 1855, and later by J. Monier in 1861 and by F. Coignet in 1867 for the construction of simple structures and tubes. The American W.E. Ward built a reinforced concrete house near New York in 1873, known as “Ward’s Castle”, which exists up to now.

Other pioneers were T. Hyatt, F. Hennebique, G.A. Wayss, M. Koenen and C.M. Döring. (P. Zararis, 2002,2004)

A more recent development was prestressed concrete, in which the steel is tensioned before loads are applied to concrete. The advantage of prestressed concrete is the possibility to use more slender sections and also the reduction or elimination of concrete cracking, which inevitably occurs when reinforced concrete is subject to tension or bending.
During the twentieth century there was a steady rise in the strength of ordinary concrete. Before the First World War, the typical concrete strength was about 11-15 N/mm². By the 1930s, typical cubic strengths had risen to 15-20 N/mm², and since the 1950s they have risen again to 20-30 N/mm². Since the 1930s it has been possible, using special mixes, to make higher-grade concrete, which nowadays can reach a strength of up to 120 N/mm².

During the same period an understanding of the parameters that affect the durability of concrete and reinforced concrete was also being developed. The protection of the embedded reinforcement against corrosion arising from contact with atmospheric oxygen and moisture has been assigned as a matter of great importance for the durability of concrete. The durability is connected with the exposure conditions and can be improved by increasing the thickness of the concrete cover around the reinforcement. Moreover, the increment of the cement content and the reduction of the water/cement ratio can enhance the corrosion protection as well as the strength of concrete.

In case of a concrete historical structure, the knowledge of the concrete quality in terms of strength and other properties and also the type and position of reinforcement is an integral matter for the part of inspection, assessment and rehabilitation. (S. MacDonald, 2002)
3. DETERIORATION MECHANISMS OF CONCRETE

All building materials including concrete have a limited lifetime. However, the situations of premature failure are issues that cause concern. In such cases the interaction between the environmental conditions, material properties and structural factors must be considered for evaluating the failure of the structures. Furthermore, poor practices and inefficient workmanship and supervision are common reasons for failure. (R.O. Heckroodt, 2002)

The deterioration of concrete that leads to the failure of concrete structures can be attributed to different causes, the main of which is the corrosion of the reinforcing steel. The other causes are less common, but still critical, factors of material failure. The signs of deterioration on concrete, as well as the causes and the verifying tests for each cause can be summarized with the following table:

<table>
<thead>
<tr>
<th>Visual appearance of deterioration</th>
<th>Type of deterioration and causes</th>
<th>Confirmatory testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large areas of rust stains, cracking along pattern of reinforcement, spalling of cover concrete, delamination of cover concrete</td>
<td>Reinforcement corrosion: exposure to normal climatic conditions, with cyclic wetting and drying</td>
<td>Cover depth of rebar Carbonation and chloride testing Exploratory coring Electrochemical testing</td>
</tr>
<tr>
<td>Expansive map cracking, restrained cracking following reinforcement, white silica gel at cracks</td>
<td>Alkali-aggregate reaction: concrete made with reactive aggregates</td>
<td>Core analysis for gel and rimming of aggregates Petrographic analysis Aggregate testing</td>
</tr>
<tr>
<td>Deep parallel cracking, pattern reflects reinforcement positions</td>
<td>Drying shrinkage/creep: initial too rapid drying, long-term wetting and drying cycles</td>
<td>Concrete core analysis Loading and structural analysis Aggregate and binder analysis</td>
</tr>
<tr>
<td>Deterioration of surface, salt deposits on surface, cracking caused by internal expansive reactions</td>
<td>Chemical attack: exposure to aggressive waters</td>
<td>Chemical analysis of concrete Core examination for depth of attack and internal distress</td>
</tr>
<tr>
<td>Surface leaching of concrete, exposed aggregate, no salt deposits</td>
<td>Softwater attack: exposure to moving fresh waters (slightly acidic) in conduits</td>
<td>Chemical analysis of water Core examination for leaching Aggregate and binder analysis</td>
</tr>
<tr>
<td>Surface discoloration, concrete spalling, buckling, loss of strength, microcracking</td>
<td>Fire damage: exposure to open fires sufficient to cause damage</td>
<td>Core examination for colour variations, steel condition Petrographic analysis Specialist techniques</td>
</tr>
</tbody>
</table>
Major cracking and localised crushing, excessive deformations and deflections of structural members | **Structural damage:** structure subjected to overload | Loading and structural analysis Core testing for compressive strength and elastic modulus

| Table 3.1- Concrete deterioration diagnostics *(R.O. Heckroodt, 2002)* |

### 3.1. Corrosion of the reinforcement

The steel reinforcement of concrete may consist of one or more of a variety of types: mild steel, high yielding steel, weathering steel, ferritic, stainless or galvanized steel.

The steel itself is rarely responsible for problems with concrete, since the problems more frequently encountered are caused by corrosion and positioning of the reinforcement. Steel in sound concrete, which has a pH of about 13, is in a passive state. A protective chemical skin forms over the surface of the reinforcement and inhibits corrosion. If this breaks corrosion will start. Early experience has shown that damage to the coating due to poor site handling could be the responsible for accelerated corrosion damage. *(F. Rendell et al, 2002)*

Apart from passivity, there are four more mechanisms that can prevent corrosion or at least control the corrosion current. These mechanisms are resistive control, cathodic control, anodic control and anodic protection. In case corrosion is neither absent as a result of passivity nor is controlled by polarization or resistivity, corrosion of some kind will take place.

The normal form of corrosion is the one of general corrosion, where the anodic and cathodic areas are too close to be distinguished and rust can grow to a volume several times that of the steel from which it was formed. The growing rust can eventually crack the cover and may also cause delamination.

Where there is a significant concentration of chloride contamination in wet concrete and little oxygen is available at corroding anodic areas, soluble iron compounds may escape and through cracks or pores in the concrete, leaving the reinforcement deeply pitted sometimes without there being any visible sign of damage on the concrete surface. In this case when the concrete is broken open, green or black partially oxidized corrosion products can be observed, spread through the concrete. This type of corrosion is often called black rust. Rust staining that has spread from reinforcement is often an indication that the cause of corrosion is chloride contamination.

As it has been mentioned before, passivity of steel can be lost if the alcalinity of the concrete is reduced by carbonation due to the carbon dioxide from the air. In this case steel is no longer protected. However, even if the concrete is strongly alcaline and uncarboned, passivity can also be lost if chlorides are present at a sufficient high concentration. *(P. Pullar-Strecker, 2002)*
3.2. Alkali-aggregate reaction

The existence of alkali-aggregate reaction as an intrinsic deleterious process between the constituents of concrete was demonstrated by Stanton in 1940, and was considered as a major scientific achievement. According to this demonstration it became clear that the characteristics of cement and aggregate used for concrete were strongly connected to deterioration conditions. (R.N. Swamy, 1992)

Alkali-aggregate reaction occurs in concrete when alkalis from the cement, or from an external source, react with certain aggregates to form products that are harmful in some way to concrete. There have been recognized four kinds of alkali-aggregate reaction: the alkali-silica reaction, the alkali-silicate reaction, the alkali-carbonate reaction and other alkali-aggregate reactions.

3.2.1. Alkali-silica reaction

The alkali-silica reaction in concrete happens when alkalis from the cement or an external source react with free silica present to certain aggregates to form alkali-silica gel. The alkali-silica gel has the property of absorbing water and expanding. This expansion can cause the aggregate particles and concrete to crack, and ultimately can damage concrete.

Three condition needs to occur at the same time so that the alkali-silica reaction can take place. These three conditions are: the presence of alkalis in the cement paste, the presence of a reactive aggregate in the concrete and a supply of water. The alkali-silica reaction is by far the most common and important of the alkali-aggregate reactions.

When concrete has been damaged by alkali-silica reaction, the characteristic feature it displays is a network of cracks which on the surface of concrete produce a crazed pattern referred to as “map-cracking”. Sometimes, but not always, white alkali-silica gel can be seen emanating from the cracks, but in dry weather this often dries to a white powder.

3.2.2. Alkali-silicate reaction

The alkali-silicate reaction is the same as the alkali-silica reaction except that in this case the reactive constituent in the aggregate is not free silica but silica present in the combined form of phyllosilicates. It is probable that the phyllosilicates are reactive only if fine-grained.

3.2.3. Alkali-carbonate reaction

The alkali-carbonate reaction in concrete occurs when alkalis from the cement or an external source react with certain dolomitic limestones containing clay. The alkalis react with the mineral dolomite, causing it to break down into brucite and calcite, in a reaction that is called “dedolomitization”. As a
result of the dedolomitization the rock is opened up, allowing water to enter, which causes the clay to swell and disrupt the aggregate, thereby cracking the concrete. The reaction should be called alkali-dolomite reaction as it does not occur with ordinary calcite limestones.

3.2.4. Other alkali-aggregate reactions

Because the alkalis sodium hydroxide and potassium hydroxide are so reactive, other reactions with aggregates can sometimes take place when concrete is wet. (G. West, 1996)

3.3. Creep of concrete

Creep of concrete is the time dependant increase in strain observed in the direction of the applied stress, which is usually determined by subtracting from the total measured strain in a loaded specimen, the sum of the initial elastic strain due to the applied stress, the shrinkage and any thermal strain in an identical free specimen. Creep can be further devided into basic creep and drying creep. Basic creep is the deformation observed when the specimens are scaled from the environment whereas the drying creep is dependent on the moisture distribution as in drying shrinkage.

According to reported work on basic creep of high strength concrete, creep reduces with the increase on the concrete compressive strength. (J. Anat & S. Setunge, 2002)

3.4. Chemical attack on concrete

Concrete is vulnerable to chemical attack by a wide range of chemical compounds in solution, and this applies to both Portland cement concrete and high alumina cement concrete. From a practical point of view, the chemicals that are aggressive to concrete can be devided into five main categories: acids (all), ammonium compounds (some, not all), magnesium compounds (some, not all), sulphates (all) and others, including alkali hydroxides.

The intensity of the attack depends on a number of factors, the principals of which are the chemical composition of the aggressive agent, the concentration, pH, porosity and permeability of the concrete, the type of cement used and the contact time. (P.H. Perkins, 1997)
3.5. Leaching of concrete

Leaching of concrete by percolating or flowing water has sometimes caused severe damage (dams, pipes, conduits), and is potentially important for the long-term storage of nuclear wastes. Pure water may be expected to remove alkali hydroxides, dissolve CH and decompose the hydrated silicate and aluminate phases.

The rate of the attack depends on the quantity and shape of concrete, the rate at which the water percolates through or flows over it, the temperature and the concentrations of solutes in the water. Attack is most likely to be serious with soft water. (H.F.W. Taylor, 1997)

3.6. Fire on concrete

Due to its low thermal conductivity, a layer of concrete is frequently used for fireproofing of steel structures. However, concrete itself may be damaged by fire.

Up to about 300 °C, the concrete undergoes normal thermal expansion. Above that temperature, shrinkage occurs due to water loss; however, the aggregate continues expanding, which causes internal stresses. Up to about 500 °C, the major structural changes are carbonation and coarsening of pores. At 573 °C, quartz undergoes rapid expansion due to Phase transition, and at 900 °C calcite starts shrinking due to decomposition. At 450-550 °C the cement hydrate decomposes, yielding calcium oxide. Calcium carbonate decomposes at about 600 °C. Rehydration of the calcium oxide on cooling of the structure causes expansion, which can cause damage to material which withstood fire without falling apart. Concrete in buildings that experienced a fire and were left standing for several years shows extensive degree of carbonation.

Concrete exposed to up to 100 °C is normally considered as healthy. The parts of a concrete structure that is exposed to temperatures above approximately 300 °C (dependent of water/cement ratio) will most likely get a pink color. Over approximately 600 °C the concrete will turn light grey, and over approximately 1000 °C it turns yellow-brown. One rule of thumb is to consider all pink colored concrete as damaged that should be removed.

Fire will expose the concrete to gases and liquids that can be harmful to the concrete, among other salts and acids that occur when gasses produced by fire come into contact with water. (wikipedia, 2009)
3.7. Effect of repeated loading on concrete

Concrete used for structures is subjected to numerous numbers of repeated loads during their service life. The performance of concrete to repeated load is commonly determined by fatigue tests, and most of them have been carried out mainly for determining the limitation of fatigue rupture of concrete, because structures become seriously dangerous as mechanical property of concrete closes on its fatigue limitation.

Since members of a structure are designed to keep the stresses occurred in concrete beneath lower level than those of failure limitation, it is therefore quite seldom for concrete in actual structures to be subjected to such a higher level of repeated load that may result to fatigue rupture. However, repeated loads cause to concrete a gradual damage, even though the level of the stress is low, and it would lead to reduction of durability of concrete. This could be attributed to the fact that the microstructure of concrete might be changed and thereby permeability of concrete to carbon dioxide gas would be presumed to increase.

According to research on the effect of repeated load on pore structure and carbonation of concrete, it has been concluded that concrete subjected to repeated loads is changed in terms of microstructure by fatigue and thereby becomes more sensitive to carbonation, even if the level of the repeated loads is low. Furthermore, the usefulness of a scale for evaluating the degree of fatigue on concrete can be shown through comparing the change of pores and carbonation depth by fatigue. (K. Tanaka et al, 1999)

3.8. Concrete structures subjected to overloading

Concrete damage caused by structural overloading is usually quite obvious and easy to detect. Frequently, the event causing overloading has been noted and is a matter of record. The stresses created by overloads result in distinctive patterns of cracking that indicate the source and cause of excessive loading and the points of load application. Normally, structural overloads are one time events and, once defined, the resulting damage can be repaired with the expectation that the cause of the damage will not reoccur to create damage in the repaired concrete.

A structural engineer should be expected to perform the necessary structural analysis to fully define and evaluate the cause and resulting damage of most structural overloads and to assist in determining the extent of repair required. This analysis should include the determination of the loads the structure was designed to carry as well as the extent the overload exceeded the design capacity. The damage on the concrete must be inspected thoroughly to determine the entire effect of the overload on the structure. All displacements must be discovered and any possible secondary damage must be located.
It must be ensured that the damage did not first weaken the concrete and make it incapable of carrying the design loads.

The repair of damage caused by overloading can be performed with conventional replacement concrete. The need for repair and/or replacement of damaged reinforcing steel should be anticipated and included in the repair procedures. (G. Smoak, 2002)

**3.9. Deterioration caused by cyclic freezing and thawing**

The deterioration of concrete due to freezing and thawing cycles is very common in cold climates. This type of damage occurs when concrete undergoes cyclic freezing and thawing and the concrete pores, during freezing, are nearly saturated with water (more than 90 per cent of saturation). During freezing, water experiences about 15 per cent volumetric expansion. If the pores and capillaries in concrete are nearly saturated, the expansion creates tensile forces that cause the cement mortar matrix to fracture. This process occurs from the outer surface inwards in an almost layering fashion.

The rate of this deterioration procedure depends on the number of freezing and thawing cycles, the degree of saturation during freezing, the concrete porosity and the exposure conditions. Typical examples of locations subjected to freezing and thawing damage are tops of walls exposed to snow or water spray, horizontal slabs exposed to water and vertical walls at the water line. If such concrete has a southern exposure, it will experience daily cycles of freezing during the night and thawing during the morning. In contrast with this, concrete in a northern position may only experience one freezing and thawing cycle each winter, which could hardly be regarded as a worrying condition.

Another type of damage caused by freezing and thawing cycles is known as D-cracking. In this case the expansion occurs in low quality, absorptive, coarse aggregate instead of the cement mortar matrix. D-cracking can usually be observed at the exposed corners of walls or slabs formed by joints. A series of roughly parallel cracks exuding calcite usually cuts across the corners of such damage.

The mitigation of freeze-thaw deterioration can be succeeded by reducing or eliminating the cycles of freezing and thawing, or reducing the absorption of water into concrete. Usually, it is not practical to protect or insulate concrete from cycles of freezing and thawing temperatures. Instead of that, concrete sealing compounds can be applied to exposed concrete surfaces in order to prevent water absorption. The sealing compounds are not effective in protecting inundating concrete, but they can provide protection for concrete exposed to rain, windblown spray or snowmelt water.

Repair of concrete damaged by freeze-thaw cycles normally consists of concrete replacement, when the damage is 6 inches or deeper. When the damage is between 1.5 and 6 inches, the replacement can be executed with epoxy-bonded concrete or polymer concrete. The replacing concretes must contain air entraining admixtures (AEA). AEA produce small bubbles into the concrete matrix which
provide space for the water to expand during freezing. With the proper type and concentration of AEA, there should be very little damage by cycling freezing and thawing, apart from the very severe climates. Repair of spalls or shallow deterioration less than 1.5 inches deep is not suggested. (G. Smoak, 2002)
4. VISUAL INSPECTION ON STRUCTURES

4.1. Introduction

Visual inspection is a very important part of the investigation process, which most of the time provides valuable information that are quite helpful to estimate the condition of a structure. Visual features may be related to various aspects such as workmanship, structural serviceability and material deterioration. Their distinction and identification is quite significant matter. These signs can be, for instance, cracks, pop-outs, spalling, disintegration, colour change, weathering, staining, surface blemishes and lack of uniformity. Extensive information can be gathered from visual inspection to give a preliminary indication of the condition of the structure and provide guidance to the formation of a subsequent testing programme. The visual inspection however should not be restricted only to the structure being investigated. It should also include neighbouring structures, the surrounding environment and the climatic condition, which can probably be a very difficult part of the whole structural investigation. The importance and benefits of a visual survey should not be underestimated, as the omission of what appears to be insignificant evidence can often lead to a wrong conclusion being made. (IAEA, 2002)

4.2. Tools and equipment of visual inspection

An engineer carrying out a visual survey should be well equipped with tools to facilitate the inspection. These involve accessories such as measuring tapes or rulers, markers, thermometers, anemometers and others. Binoculars, telescopes, borescopes, endoscopes or fibre scopes may be useful where access is difficult. A crack width microscope or a crack width gauge is useful, while a magnifying glass or portable microscope is suitable for close up examination. A good camera with the necessary zoom and micro lenses and other accessories, such as polarized filters, facilitates pictorial documentation of defects, and a portable colour chart is helpful in identifying variation in the colour of the concrete. A complete set of relevant drawings showing plan views, elevations and typical structural details allows recording of observations and mapping of different types of damage to be made. (IAEA, 2002)

4.3. General procedure of visual inspection

Before any visual inspection can be made, the engineer must study intensely all relevant structural drawings, plans and elevations to become familiar with the structure. In addition to this, a historical survey must be executed, to collect any available important documents. These normally include technical specification, reports of previous tests or inspection procedures, construction records, details
of materials used, methods and dates of construction. The information provided by the historical investigation is essentially the first step of structural investigation, particularly in the case of historical buildings.

The visual survey should be carried out systematically and cover the defects present, the current and past use of the structure, the condition of adjacent structures and the environmental condition. All defects must be identified and classified, and, where possible, their causes must be identified. The distribution and extent of defects need to be clearly recognized. It is quite important to distinguish whether the defects are random or appear in a specific pattern and whether the defect is confined to certain locations of members or is present all over the structure. Visual comparison of similar members is particularly valuable before testing to determine the extent of the problems in such cases. A study of similar structures or other structures in the local area constructed with similar materials can also be helpful in providing ‘case study’ evidence, in particular if those other structures vary in age from the one under investigation. There is a need to identify associated or accompanying defects, especially which particular defect predominates. (IAEA, 2002)

Segregation or excessive bleeding at shutter joints may reflect problems with the concrete mix, as might plastic shrinkage cracking, whereas honeycombing may be an indication of a low standard of construction workmanship. Lack of structural adequacy may show itself by excessive deflection or flexural cracking and this may frequently be the reason for an in situ assessment of a structure. Long term creep defects, thermal movements or structural movements may cause distortion of doorframes, cracking of windows, or cracking of a structure or its finishes.

Material deterioration is often indicated by surface cracking and spalling of the concrete and examination of crack patterns may provide a preliminary indication of the cause. Systematic crack mapping is a valuable diagnostic exercise when determining the causes and progression of deterioration. Observation of concrete surface texture and colour variations may be a useful guide to uniformity. Colour change is a widely recognized indicator of the extent of fire damage.

Visual inspection is not confined to the surface but may also include examination of bearings, expansion joints, drainage channels and similar features of a structure. Any misuse of the structure can be identified when compared to the original designed purpose of the structure.

An assessment may also need to be made of the particular environmental conditions to which each part of the structure has been exposed. In particular the wetting and drying frequency and temperature variation that an element is subjected to should be recorded because these factors influence various mechanisms of deterioration in concrete. Settlement of surrounding soil or geotechnical failures need to be recorded. Account must also be taken of climatic and other external environmental factors at the location, since factors such as freezing and thawing conditions may be of considerable importance when assessing the causes of deterioration.
A careful and detailed record of all observations should be made as the inspection proceeds. Drawings can be marked, coloured or shaded to indicate the local severity of each feature. All defects must be noted by different characteristic symbols, so that they can be distinguished and easily recognised.

Defects that commonly need recording include:

- cracking which can vary widely in nature and style depending on the causative mechanism
- surface pitting and spalling
- surface staining
- differential movements or displacements
- variation in algal or vegetative growths
- surface voids
- honeycombing
- bleed marks
- constructional and lift joints
- exudation of efflorescence

(IAEA, 2002)

4.4. Applications of visual inspection

For existing structures, the presence of some features requiring further investigation is generally indicated by visual inspection, and it must be considered as the most important component of routine maintenance. It will also provide the basis for judgements relating to access and safety requirements when selecting test methods and test locations.

As before mentioned, a visual inspection provides an initial estimation of the condition of the concrete which leads to the conclusion on a subsequent testing programme. Through such inspections, proper documentation of defects and features in the concrete structure can be accomplished. Furthermore, visual inspection can reveal considerable information regarding the structure such as the construction methods, weathering, chemical attack, mechanical damage, physical deterioration, abuse, construction deficiencies or faults. (IAEA, 2002)
5. TESTING OF CONCRETE IN STRUCTURES

5.1. Introduction

An important range of test methods is available for use during inspection, or on samples extracted from the structure for laboratory investigation. For existing structures, the need for testing may arise from a variety of causes which can include:

a) change of usage or extension of a structure

b) acceptability of a structure for purchase or insurance

c) assessment of structural integrity or safety following material deterioration, or structural damage such as caused by fire, blast or overload

d) serviceability or adequacy of members known or suspended to contain material which does not meet specifications, or with design faults

e) monitoring of strength development in relation to framework stripping, prestressing or load application (J. H. Bungey, 1982)

The tests for concrete structures can also be classified as non-destructive (NDT), which are not harmful, semi-destructive or minor destructive (MDT), which usually cause minor, but reversible, damage to the structure, and destructive (D), which are the most invasive. In historic concrete structures, there is usually a need for better understanding or assessing the properties of historic materials for restoration and rehabilitation purposes, especially when not enough information, such as documentation, is available. In such cases, only non-destructive tests should be regarded as suitable, in order not to harm the ancient or historic material and not to cause any alteration to parts of the structure. However, NDT can give qualitative results in most cases, and their calibration can be rather difficult. As a result of this, MDT can also be used to form a basis for the calibration of NDT.

5.2. Available test methods

According to the necessity of information, there is a variety of tests that are available. The principle options can be presented in the following table:
<table>
<thead>
<tr>
<th>Information required</th>
<th>Methods available</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member behaviour and strength</td>
<td>Load tests with deflections and strain measurements</td>
</tr>
<tr>
<td>Concrete strength</td>
<td>Cores</td>
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<td></td>
<td>Rebound hammer</td>
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<td></td>
<td>Pull-out and internal fracture</td>
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<td></td>
<td>Break-off and pull-off</td>
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<tr>
<td></td>
<td>Penetration resistance</td>
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<td></td>
<td>Ultrasonic pulse velocity</td>
</tr>
<tr>
<td>Cracking</td>
<td>Ultrasonic pulse velocity</td>
</tr>
<tr>
<td></td>
<td>Acoustic emission and holography</td>
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<tr>
<td>Honeycombing and compaction</td>
<td>Ultrasonic pulse velocity</td>
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<tr>
<td></td>
<td>γ-radiography</td>
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<tr>
<td></td>
<td>Cores</td>
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<td></td>
<td>Pulse echo techniques</td>
</tr>
<tr>
<td>Density</td>
<td>γ-radiometry</td>
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<tr>
<td>Permeability</td>
<td>Absorption, flow tests and capillary rise</td>
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<tr>
<td>Moisture content</td>
<td>Nuclear methods</td>
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<tr>
<td></td>
<td>Electrical resistivity</td>
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<td></td>
<td>Microwave absorption</td>
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<tr>
<td>Cement content</td>
<td>Chemical analysis</td>
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<td></td>
<td>Nuclear methods</td>
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<tr>
<td>Mix properties and constituents</td>
<td>Chemical analysis</td>
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<tr>
<td></td>
<td>Cores</td>
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<tr>
<td></td>
<td>Micrometric methods</td>
</tr>
<tr>
<td>Reinforcement detection</td>
<td>Magnetic methods</td>
</tr>
<tr>
<td></td>
<td>X- and γ-radiography</td>
</tr>
</tbody>
</table>
Concrete deterioration

Chemical analysis
Thermoluminescence
Ultrasonic pulse velocity
Micrometric methods

Abrasion resistance and soundness
Rebound hammer
Wear tests
Physical methods
Infrared thermography

Table 5.1. - In-situ tests on concrete (J. H. Bungey, 1982)

The methods that have been previously reported at Table 5.1. can generally be performed as non-destructive, apart from chemical and petrographic methods which require cutting or drilling small samples from the concrete. Many of these methods require expensive equipment, with extensive safety precautions in some cases. Cost consideration, as well as the aspect of damage, time and reliability can strongly influence the choice of the method to be executed. (J. H. Bungey, 1982)

Table 5.2. indicates the damage resulting from strength tests, along with the main restrictions in each case. Table 5.3. gives information about the relative cost, the speed, the damage and the reliability of calibration for each test.

<table>
<thead>
<tr>
<th>Test method</th>
<th>Probable damage</th>
<th>Major restrictions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Collapse load test</td>
<td>Member destroyed</td>
<td>Member must be isolated, and preferably removed from rest of structure before the test</td>
</tr>
<tr>
<td>Overload test</td>
<td>Possible loss of member</td>
<td>Member must be isolated, or allowance made for load distribution to adjacent parts of the structure</td>
</tr>
<tr>
<td>Cores</td>
<td>Holes to be made good</td>
<td>Limitation of cores size and proportion Safety precautions for critical</td>
</tr>
</tbody>
</table>
### Penetration resistance (Windsor probe)
- Cone approx. 50 mm dia. To be made good
- Minimum edge distance
- Minimum member thickness

<table>
<thead>
<tr>
<th>Pull-out</th>
<th>Bolt hole remains</th>
<th>Preplanned</th>
</tr>
</thead>
</table>

| Internal fracture | Bolt to be cropped or cone approx. 75 mm dia. To be made good | Drilling difficulties |

| Ultrasonics | None | Two smooth surfaces necessary |
| Rebound hammer | None (for mature concrete) | Smooth surface necessary |

#### Table 5.2.- Strength tests: damage and restrictions *(J. H. Bungey, 1982)*

<table>
<thead>
<tr>
<th>Test method</th>
<th>Cost</th>
<th>Test speed</th>
<th>Damage to concrete</th>
<th>Representativeness</th>
<th>Reliability of strength calibrations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Collapse load test</td>
<td>High</td>
<td>Slow</td>
<td>Total</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td>Overload test</td>
<td>High</td>
<td>Slow</td>
<td>Variable</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td>Cores</td>
<td>High</td>
<td>Slow</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Good</td>
</tr>
<tr>
<td>Penetration resistance</td>
<td>Moderate</td>
<td>Fast</td>
<td>Minor</td>
<td>Near surface only</td>
<td>Moderate</td>
</tr>
<tr>
<td>Pull-out/ Internal fracture</td>
<td>Moderate</td>
<td>Fast</td>
<td>Minor</td>
<td>Near surface only</td>
<td>Moderate</td>
</tr>
<tr>
<td>Ultrasonics</td>
<td>Low</td>
<td>Fast</td>
<td>None</td>
<td>Good</td>
<td>Moderate</td>
</tr>
<tr>
<td>Rebound hammer</td>
<td>Very low</td>
<td>Fats</td>
<td>Unlikely</td>
<td>Surface only</td>
<td>Poor</td>
</tr>
</tbody>
</table>

#### Table 5.3.- Strength tests: relative merits *(J. H. Bungey, 1982)*
5.3. Non-destructive testing of concrete

5.3.1. Introduction

Testing concrete structures is often necessary in order to decide whether the structure can perform according to its designed purpose. There is a variety of tests for concrete that includes methods entirely non-destructive, where there is no damage to concrete, methods where the concrete surface is slightly damaged, and partially destructive techniques, such as core, pull-out and pull-off tests, where the surface needs to be repaired after testing. The diversity of properties that can be assessed using non-destructive tests and partially destructive tests is quite large and contains essential parameters, such as density, elastic modulus and strength, surface hardness and surface absorption, as well as reinforcement location, size and distance from the surface. In some cases it is also possible to check the quality of workmanship and structural integrity by the ability to detect voids, cracking and delamination.

Non-destructive testing is applicable for both old and new structures. For new structures, the principal applications are likely to be for quality control or the resolution of doubts about the quality of materials or construction. For existing structures, it is usually connected to an assessment of structural integrity or adequacy. In either case, if, for instance, only removing cores for compression testing is executed, the cost of coring and testing may only allow a relatively small number of tests to be carried out on a large structure which may be deceptive. In these situations, non-destructive testing can performed as an initial procedure, followed by coring. (IAEA, 2002)

According to applications, there are three general categories of NDT methods used for inspection of concrete structures and civil engineering constructions.

The first category includes the tests which estimate the in-situ strength indirectly, such as surface hardness, and directly, such as penetration resistance and pull-out techniques.

The second category includes the tests which measure the material properties of concrete, such as moisture, density, compressive wave velocity, modulus of elasticity, thickness, and temperature. Ultrasonic, nuclear and electrical methods are in this category.

The third category includes the tests, which are used to detect and locate the defect areas within concrete structures such as honeycombing, fractures, flaws and delaminations. Impact-echo, ground penetrating radar, pulse-echo, infrared thermography and acoustic emission methods are in this category. (IAEA, 2005)

Table 5.4. presents the main NDT techniques and applications for concrete structures. Some of them have been recognised by the standard committees of the professional associations, such as American Concrete Institute (ACI) and American Society for Testing and Materials (ASTM). (IAEA, 2005)
5.3.2. Overall review of available NDT methods for concrete structures

Below are presented the most common applications that are used in the concrete industry.

5.3.2.1. Mechanical methods

Some of the mechanical devices, that are currently available, can be used to measure strength or hardness of materials. Other types of mechanical devices are used to detect defects or properties of materials using stress wave propagation techniques. Mechanical methods are divided into six major techniques:

(a) Surface hardness equipment

The aim of the surface hardness methods is to correlate surface hardness and compressive strength of concrete. Available equipment is capable of estimating the concrete hardness by impacting the surface and measuring the indentation or rebound value. The Schmidt hammer testing system and the Equitip hardness tester are the two most widely used concrete hardness testing equipment. The performance of both devices is based on the rebound of their impact body as it comes in contact with
the concrete surface. The new Schmidt hammer is equipped with digital displays, which enhance data collection (fig. 5.2.)

The Schmidt rebound hammer also known as Swiss hammer was developed in 1948. Impact hammer or simply rebound hammer measures the surface hardness of concrete by releasing a spring loaded plunger which impacts the concrete and measures the rebound distance. This rebound hammer tests only the concrete underneath on relatively small contact area. The results are therefore, sensitive to local variations in the concrete such as the presence of a large hard aggregate, soft paste or voids. (S.R.M. Khan et al, 2004)

For both the Schmidt hammer and the Equitip hardness tester, the hardness values must be corrected if the impact direction is different from the vertical direction. Conversion tables are used for this purpose. Other factors that may influence the results for concrete are: the moisture content, surface conditions, and aggregate size and type. Although the use of surface hardness testing equipment is simple and straightforward, the accuracy of the results is highly dependent on the correct positioning of the devices on the specimen surface.

![figure 5.1.-Application of Schmidt hammer](image-url)
(b) Penetration resistance and pull-out systems

Both systems are semi-destructive, but since in some literature they are classified as non-destructive techniques they will be briefly discussed.

The penetration technique essentially uses a powder-actuated gun or driver which fires a hardened alloy probe into the concrete. The Windsor probe testing system is the most widely known penetration resistance device available for both laboratory and in situ measurements. A steel rod is placed in the concrete at the time of construction. When it is pulled, a dynamometer measures and registers the force (fig. 5.3.). The pull-out test is mainly used during the early construction phase in order to estimate the in-situ strength of concrete.

The main disadvantage of this technique is that the tests cannot be repeated and the steel rods have to be pre-planted. The final results are strongly affected by the maximum size and shape of the coarse aggregates. (Malhotra, 1984)

Both penetration and pull-out methods are techniques for measuring compressive strength at early stages of concrete curing.
(c) Impact-echo technique

Impact-echo is a technique developed for thickness measurement and delamination location in concrete. The system is based on a high resolution seismic reflection survey on concrete structures using an impact source, a broad band unidirectional receiver and a waveform analyzer.

The mechanical impact generates stress pulses in the structure (fig. 5.5.). The stress pulses undergo multiple reflections between the top and the bottom concrete layer. The surface displacements are recorded and the frequency of the successive arrivals of the reflected pulses is determined. Wave reflections are used for detection of discontinuities and voids in concrete structures. Discontinuities, defects and reinforcements could be identified in the resulting frequency spectra, as the wave reflects from their surfaces. Thus, knowing the thickness of a given layer, together with the derived frequencies, compression and shear wave velocities can be calculated. If, on the other hand, the
thickness is unknown, the time-distance graph of the primary surface stress wave is used to calculate the thickness.

Recent studies show that impact-echo technique can be used for concrete early strength gain estimation and evaluation of micro-cracking and chemical attacks in concrete structures.

![Field impact-echo system by Andec (Canada)](image)

(d) Impulse-response technique

The impulse-response method follows a principle similar to that of the impact-echo. A stress pulse is generated by a mechanical impact on the surface of the object. The force-time function of the hammer is recorded. A transducer records the vibration response of the surface as the reflected waves arrive. The processing of the recorded waveforms of the force and the arriving reflected waveforms reveals information about the condition of the structure. The impulse response of the structure is calculated by dividing the Fourier transform of the reflected wave by the Fourier transform of the force-time function of the impact.

The technique is used to evaluate the dynamic stiffness of the concrete piles. Discontinuities, voids and the base material affect the impulse-response evaluation. The main drawback of the system is the size and shape limitation of the structure undergoing testing.

(e) Spectral analysis of surface waves (SASW) technique

The SASW method uses the Raleigh wave (R-wave) to determine the stiffness profile and layer thickness of thin concrete layers. The SASW system includes an impact device, two receiving transducers, and a two-channel waveform analyzer. The characteristics of the impact device and the relative positioning of the transducers are determined by the stiffness and thickness of the layers.
The R-wave produced by impact contains a range of frequencies, or components of different wavelengths. This range depends on the contact time of impact; the shorter the contact time, the broader the range of frequencies or wavelengths. The velocity of the individual frequency components is called phase velocity. For the component frequency of the impacts, a plot of phase velocity versus wavelength is obtained. This curve is used to calculate the stiffness profile of the test object. The experimental results are compared with theoretical curves until the results match.

The main drawbacks of SASW are the limitation on the maximum layer thickness of the two media, and the matching of theoretical and experimental data. (IAEA, 2005)

5.3.2.2. Ultrasonic methods

Ultrasonic equipment is constructed to operate based on two different principles: resonant frequency and pulse velocity method. Almost all of the field and laboratory instruments for concrete and rock evaluation make use of pulse velocity systems. This method operates by measuring the average time taken for the ultrasonic wave to travel between the source and the receiver. The distance between the two points is divided by the travel time, which gives the average velocity of compressive wave propagation in the material. This method is used for measuring uniformity and in some cases the compressive strength.

In addition to quality evaluation, ultrasonic waves can be used for determining fractures and voids within structures. This is known as the pulse-echo technique, which makes use of reflected waveforms from the interfaces to locate defects or measure thickness from only one direction.

Ultrasonic equipment (pulse velocity) is capable of locating discontinuities, of quality evaluation and of thickness measurements. Pundit and V-Meter are the most commonly used ultrasonic instrument utilized for both field and laboratory testing. In recent years, Andec Mfg Ltd. has developed a more sophisticated high-power digital system called Soniscope. Soniscope’s 4000 volts pulsar and digital signal processing package allows the user to pass through thick concrete monoliths and produce tomographic images based on ultrasonic wave velocity variations.

For ultrasonic systems, the main drawbacks are problems caused by wave scattering, poor coupling of sensors to medium and unwanted noise, which are mainly due to the heterogeneous nature of concrete elements.

The principle of the resonant frequency method is based on the relation between the natural frequency of vibration of an elastic medium and its dynamic elastic properties. For a vibrating beam of known dimensions, the natural frequency of vibration is related to the elastic properties and the density of the medium. Therefore, the dynamic elastic modulus of the material can be calculated by measuring the natural frequency of vibration of the samples.

In addition to the dynamic elastic properties of the concrete, moisture content and the strength of the samples could also be evaluated by using the mathematical relationships between the damping
constant and the physical properties of the specimen. This is mainly a laboratory testing procedure and representative samples from the structures are tested for quality control purposes. However, field tests have been recorded on concrete columns. (IAEA, 2005)

![Digital ultrasonic system by Andec (Canada)](image)

**5.3.2.3. Electrical methods**

The change in the electrical properties of the concrete such as electrical resistance, dielectric constant and polarization resistance can be monitored in order to evaluate thickness, moisture content, density and temperature variations.

Electrical resistivity has been used to measure concrete thickness by applying the dialectic difference between the concrete and the base material. A change in the slope of the resistivity versus depth curve is used to estimate the depth of a concrete slab. The dielectric constant of concrete increases with any increase in the moisture content. Capacitance instruments are used to measure the in-situ moisture content of the concrete. Linear polarization is used to calculate the corrosion rate of steel reinforcement bars in concrete slabs and pavements. Problems may occur due to variability of moisture content and temperature in the concrete.

The Half-Cell potential instruments are used to determine corrosion in reinforcement bars based on the anomalies in the electrical field generated by the instrument on the surface of the concrete structure. The main drawback of the electrical methods is the assumption that the resistivity of each layer is constant and varies slightly with depth, which is far from reality. (IAEA, 2005)
5.3.2.4. Magnetic methods

Magnetic devices are available for detecting ferromagnetic materials. For concrete structures these devices are used for detecting the position of the reinforcing bars, pre-stressing tendons, and metal ducts within the concrete, and can also be used for identification of the bars’ corrosion.

Three main techniques using different magnetic principles are available for NDT applications:

(a) Magnetic Induction,
(b) Flux Leakage Theory, and
(c) Nuclear Magnetic Resonance (NMR).

The Magnetic Induction systems operate based on the fact that steel rods or any other ferromagnetic objects within the specimen affect and in fact distort the primary field generated by the instrument. The instrument consists of a U-shaped magnetic core with two mounted coils. An alternating current is passed through one coil and the induced current is measured in the other coil. The presence and distance of steel reinforcement bars and ferromagnetic mineralized zones can be located by their effect on the induced current.

Other magnetic devices use the Flux Leakage methodology. These instruments are sensitive to changes in magnetic lines of force (flux) flowing through the materials. When a ferromagnetic material is magnetized, magnetic flux flows through and completes a path between the poles. However, if the pathway is disturbed by a crack or discontinuity, its magnetic permeability will be disturbed and this results in a flux leakage. The intensity of flux leakage can be used to characterize the various discontinuities and their shapes.
Nuclear Magnetic Resonance (NMR) systems use the interaction between nuclear magnetic dipole moments and a magnetic field. This interaction can be used to measure the moisture content of the material by detecting the signals from the hydrogen nuclei present in water molecules. The main drawback of these systems is that they cannot be used in heavily reinforced concrete elements or in tunnels with steel culvert supports, since the effect of a secondary field cannot be eliminated as a result of the intense presence of ferromagnetic alloys and detection and positioning of the targets is very difficult. (IAEA, 2005)

5.3.2.5. Electromagnetic methods

Short-pulse or Ground Penetration (or Probing) Radar (GPR) is an electromagnetic equivalent of ultrasonic and stress-wave reflection techniques. An antenna transmits the EM pulses into the object. The energy travels through different materials with different velocities. This variation of velocity has a direct relationship to the material quality, which is controlled by the material's dielectric properties. A change in the material's dielectric constant, which occurs at interfaces such as concrete and air, or between two different rock types, causes a change in wave shape.

The reflected signals are received by the receiving antenna, separate from the transmitting antenna, or in the same casing. Signals are positive when they are traveling from a lower to a higher dielectric material (e.g. air to concrete) and are negative when they are traveling from higher to lower dialectical material (e.g. concrete to air). Electromagnetic waves are also known as microwaves or centimeter waves. An EM wave, which has a range of wavelengths between 0.3 and 30 cm corresponds to frequencies in the range of 1 GHz to 100 MHz. The commercially available GPR systems operate with the frequencies ranging from 20 MHz to 2 GHz.
The radar system operates by a transmitting antenna and a receiving antenna that collects the reflected waveforms. The control unit controls the functions of the GPR system such as scanning speed, signal filters, amplifications and time measurements. Ground Penetrating Radar could be used for detection of delaminations, cracks, voids, and reinforcing steel bars within the concrete with reasonable accuracy (fig.5.9.). The GPR has been demonstrated to be an effective tool in measuring the thickness and geometry of pillar structures, in addition to locating faults and shear zones in underground coal, salt and gold mines.

The resolution of the survey can be accurately set by using various antenna-signal frequency combinations. The GPR could be used for inspection of underground concrete linings, shafts and dams. The system could also be used to map the cavities and fracture patterns behind the linings, particularly in the case of shafts and subway tunnels. (IAEA, 2005)

![Ground Penetrating Radars for detection of delaminations, cracks, voids, and reinforcing steel bars in concrete structures](image)

**5.3.2.6. Infrared thermographic methods**

Infrared thermography, or infrared scanning, is a technique which operates based on the capability of various materials to absorb heat. Solar radiation is the main source of heat for surface structures. As the solar ray flows through a structure, air voids and fracture openings absorb a larger percentage of heat than the surrounding material. This can be monitored and registered by an infrared camera. The same principle holds for steel reinforcement but at a different intensity. Using infrared thermography, it is possible to locate voids, delaminations, fractures and steel reinforcing bars.

However, it is not possible to locate their exact position. Test results are highly affected by the surrounding conditions such as time of day or seasonal changes. Moisture content of the concrete or...
rocks also affects the readings. As for the underground openings and shaft linings, an artificial primary source of heat is needed. In the tunnels, a convectional heating of the air and lining surfaces can be achieved with diesel or gas machinery exhaust. The release of toxic fumes as a result of engine exhaust is the main drawback and it is not practical to use an infrared thermography system in a closed environment. (IAEA, 2005)

5.3.2.7. Acoustic emission methods

Acoustic emission or stress emission is a general term used for any transient waveform released from a solid, which is under stress. In concrete structures, the main source of wave emission would be crack development or the slip between the concrete and the reinforcing bars. On the concrete surface a number of transducers receive and later register these low amplitude stress waves. The variation of arrival time of waveforms registered by each transducer is used to locate the source. Later interpretations can be performed using the intensity of wave emission. An increase in wave emission in a structure could be analyzed as an unsafe condition; however, this is not considered as a satisfactory deduction. (IAEA, 2005)

5.3.2.8. Radioactive methods

Radioactive methods are mainly used in the concrete industry. Radioactive systems are classified into two main subgroups:

(a) Radiography Technique

Using a radioactive source, this technique provides a photographic image of the concrete which makes it possible to locate the reinforcement bars, voids, fractures, and honeycombing. High costs associated with the source, dangerous high voltage equipment, and safety factors make this technique undesirable for field use.

(b) Radiometry Technique

Because gamma rays are capable of passing through concrete, various types of radioisotopes are used and pre-planted within the structures at the time of construction. Using calibrated charts the thickness, moisture content, and density of concrete can be measured. This can be done as a result of change in the emerging intensity of gamma rays, which are collected with the aid of a scintillation or Geiger counters. Similarly, high operational costs and safety factors have limited the use of radiometry in the field of NDT. (IAEA, 2005)
5.3.2.9. **Penetrability methods**

Many of the degradation mechanisms in concrete involve the penetration of aggressive materials, such as sulfates, carbon dioxide, and chloride ions. In most cases, water is also required to sustain the degradation mechanisms. As a result, concrete that has a surface zone that is highly resistant to the ingress of water will generally be durable. To assess the potential durability of in-situ concrete, it is necessary to focus on methods that assess the ability of the surface zone to restrict the passage of external agents that may lead to direct deterioration of the concrete or to depassivation and corrosion of embedded reinforcement.

Based on three principals mechanisms external agents can penetrate into concrete: (a) absorption, ingress of liquids due to capillary forces; (b) permeation, flow of liquid under action of pressure head; and (c) diffusion, movement of ionic or molecular substances from regions of high concentration to regions of lower concentration of the substances. The penetrability tests and equipment are developed to assess the durability potential of concrete surface. (IAEA, 2005)

5.4. **Minor-destructive testing of concrete**

5.4.1. **In-situ investigations**

5.4.1.1. **Concrete coring**

The examination and compression tests of cores cut from hardened concrete is a well established method, enabling visual inspection of the interior regions of a member to be coupled with strength estimation. Other physical properties which can be measured include density, water absorption, indirect tensile strength and movement characteristics, whilst cores are frequently used as samples for chemical analysis following strength testing.

5.4.1.2. **Core location and size**

Core location is governed by the basic purpose of testing, bearing in mind the likely strength distributions within the member, related to the expected stress distributions. Where serviceability assessment is the principal aim, tests should normally be performed at points where likely minimum strength and maximum stress coincide. Where the core is to be used for compression testing, a minimum diameter of 100 mm is required by both British and American standards. The accuracy decreases as the ratio of size of aggregate to core diameter increases. More specifically, the diameter of 150 mm and the one of 100 mm is regarded as the preferred size for a maximum aggregate size of
40 mm and 25 mm respectively. If the size of aggregate is less than 20 mm, the minimum core diameter can be 75 mm.

According to the Concrete Society Technical Report No.11, the cores are suggested to be as short as possible (l/d=1.0-1.2) for reasons of drilling costs, damage, variability along length and geometric effects on testing. Hence, procedures for relating core strength to cylinder or cube strength usually involve correction to an equivalent standard cylinder with l/d=2.0. In addition to this, it can be argued that uncertainties of correction factors are minimized if the core length/diameter ratio is close to 2.0.

The number of cores required depends on the scope of testing and the volume of concrete involved.

5.4.1.3. Drilling for mechanical tests

The normal drilling machine is a rotary cutting tool with diamond bits (fig.5.10). The equipment is portable, but rather heavy and must be firmly supported and braced against the concrete to prevent relative movement, which will result to a distorted or broken core. Water supply is also necessary to lubricate the cutter. Drilling should be performed by a skilled operator in order to assure the uniformity of pressure, which is a very important parameter. A cylindrical sample is obtained, which may contain embedded reinforcement. The reinforcement is usually removed by breaking off by insertion of a cold chisel down the side of the core, when the depth of the drilled core is sufficient. The core can be removed using the drill or tongs. The hole can be treated by ramming a dry, low shrinkage concrete into it, or by wedging a cast cylinder of suitable size into it with cement grout or epoxy resin. Each core must be examined at this stage, since if there is insufficient length for testing, voids or excessive reinforcement, extra cores must be drilled from adjacent locations. Each core must be clearly labelled for identification, with the drilling surface shown.
5.4.1.4. Drilling for petrographic analysis

Petrographic analysis is a type of laboratory analysis to identify concrete microstructure, as well as various defects on concrete. The identification of concrete microstructure is quite significant, as it is strongly connected with concrete strength, performance and long-term durability. (D. Jana, 2005)

In order to perform a petrographic analysis, several cores need to be extracted from the surface of the structure. Hence, the size of cores for petrographic analysis is not that significant. In fact, they are much smaller than the ones needed for mechanical testing. Most cores for petrographic examination are usually drilled 75-100 mm in diameter so that the maximum surface area will be the diameter times 150-200 mm. (D. A. St. John, 1990)

5.4.2. Laboratory tests

5.4.2.1. Compression testing

The standard procedure in the United Kingdom is to test the cores in a saturated condition, although in some countries dry testing is used if the in-situ concrete is in a dry state. If the core is to be saturated, testing should be performed in not less than two days after capping and immersion in water. The mean diameter must be measured to the nearest 2 mm by caliper, with measurements on two axes at ¼ and mid-points along the length of the core, and the core length also measured to the nearest 5 mm.
Compression testing will be carried out at a rate of 15 N/mm²/min in a suitable testing machine and the mode of failure noted. If there is cracking of the caps, or separation of cap and core, the result should be considered as being of doubtful accuracy. Ideally cracking should be similar all round the circumference of the core, but a diagonal shear crack is considered satisfactory, except in shore cores or where reinforcement or honeycombing is present. (J. H. Bungey, 1982)
6. DESCRIPTION OF THE INDUSTRIAL AREA OF PORTO MARGHERA

The industrial zone of Porto Marghera was created in the 1920s and resulted on bringing the railway to Venice. In the 1930s the zone was broadened and brought also automobiles. In the 1960s, a Second Industrial Zone spread southward from Marghera over what until 1953 had been tidal-land. By 1970, the industries at Marghera –chemicals, petroleum, plastics, etc.- provided nearly 40.000 jobs. (F.C. Lane, 1973)

Marghera has been one of the principal planning matters under discussion for Venice as indicated in the Revised City Master Plan of 1999 and in preliminary documents of the Strategic Plan (Pugliese, 2003; Barbiani, 2002). The site, along the south-western coastal zone of the lagoon, is composed of about 5000 hectares, divided into three zones. The first one includes surface lands that have been artificially created with waste soil extracted during the maintenance of industrial and urban canals in the period from 1920 to 1960, and then partially occupied by factories of basic industries. The second is connected with an industrial zone where the major part is occupied by commercial port infrastructures of chemical and related industries. The third zone covers a band of near lagoon and port canal water surface. Therefore, the perimeter of the site includes mainland and surrounding waters located between the wonderful historical center of Venice and the post-World War II urban sprawl. The site is further divided into thirteen macro-areas, out of which nine are industrial and four still natural, but with very high soil and water pollution. (D. Patassini et al, 2005)

The companies that were developed in Porto Marghera during the year of 1978 can be presented at Table 6.1. Table 6.2. contains some other data regarding the industrial zone.

<table>
<thead>
<tr>
<th>sector</th>
<th>number of establishment</th>
<th>number of employees</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Aliment</td>
<td>9</td>
<td>403</td>
</tr>
<tr>
<td>2. Water - Electric energy - Gas</td>
<td>14</td>
<td>1048</td>
</tr>
<tr>
<td>3. Ceramics - Fireproof materials - Glass - Building and construction materials</td>
<td>16</td>
<td>2495</td>
</tr>
<tr>
<td>4. Chemicals</td>
<td>23</td>
<td>13686</td>
</tr>
<tr>
<td>5. Mechanical</td>
<td>57</td>
<td>4445</td>
</tr>
<tr>
<td>6. Metallurgic - Iron and Steel</td>
<td>15</td>
<td>6287</td>
</tr>
<tr>
<td>7. Petroleum producing</td>
<td>23</td>
<td>1360</td>
</tr>
<tr>
<td>8. Transport</td>
<td>43</td>
<td>215</td>
</tr>
<tr>
<td>9. Various</td>
<td>36</td>
<td>448</td>
</tr>
<tr>
<td>------------</td>
<td>----</td>
<td>-----</td>
</tr>
<tr>
<td>Total</td>
<td>236</td>
<td>30387</td>
</tr>
</tbody>
</table>

Table 6.1- Companies of Porto Marghera (1978)

<table>
<thead>
<tr>
<th>Year</th>
<th>Surface Area Occupied (m²)</th>
<th>Number of Establishment</th>
<th>Number of Workers</th>
<th>Maritime Traffic (tons)</th>
<th>Rail Traffic (tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1928</td>
<td>3988000</td>
<td>81</td>
<td>5270</td>
<td>513048</td>
<td>521356</td>
</tr>
<tr>
<td>1938</td>
<td>4800000</td>
<td>94</td>
<td>16500</td>
<td>2291000</td>
<td>936467</td>
</tr>
<tr>
<td>1948</td>
<td>5280000</td>
<td>118</td>
<td>21200</td>
<td>1459179</td>
<td>942440</td>
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<tr>
<td>1958</td>
<td>5550000</td>
<td>195</td>
<td>29000</td>
<td>6002044</td>
<td>1425000</td>
</tr>
<tr>
<td>1968</td>
<td>13170000</td>
<td>239</td>
<td>33000</td>
<td>15866799</td>
<td>1413320</td>
</tr>
<tr>
<td>1978</td>
<td>13170000</td>
<td>236</td>
<td>30387</td>
<td>21628707</td>
<td>1137738</td>
</tr>
</tbody>
</table>

Table 6.2- Data on Porto Marghera