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Protection of Reinforced Concrete in Marine Environment with application to 'Marina Internacional Torrevieja' (Alicante)

Trabajo Final de Máster

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Preface

This master thesis covers a very specific domain of the durability of constructions in marine environment. While most engineering educations cover the aspect of the design and construction of new structures, little information is provided over the maintenance and repair of existing structures. However, the application of these last terms enables to reach the optimal use of materials and to reduce costs, which is in my opinion a fundamental task of an engineer.

The choice of a subject related to hydraulic engineering corresponds to my interests. My education of construction engineer started in the technology campus Ghent of university KU Leuven, in Belgium. After the achievement of the bachelor degree, I participated in a project in the north of Peru. Together with other students and local people, we designed and constructed a micro hydro power plant to provide a secluded village of green energy. This internship was a first introduction to water works and raised my interests. As a specialisation in hydraulic engineering is not provided at our campus, I participated in the exchange programme of Erasmus. This enabled the possibility to study at Universitat Politècnica de València where hydraulic engineering plays a strong role. A master thesis subject related to marine environments was a must and has been elaborated with passion.

I am grateful to my promotor Professor Vicent Esteban Chapapría, who provided the opportunity to investigate a yachting harbour and guided me during the project. In addition, I would like to thank the manager of the harbour 'Marina Internacional', Trinidad Pujante Murcia, for the warm welcome and the permission for investigation.

Louis Nollet

13th of June 2017

Objective

This master thesis provides answers for the following research question:

"How can we tackle the corrosion problems in the yachting harbour 'Marina Internacional' and ensure a sustainable state of the pontoons for another 30 years of operation?"

First, a profound literature study should provide new insights into the interaction between the concrete material, the steel reinforcement and the chloride ions. Also influencing parameters may be interesting to manipulate the deterioration process. Subsequently, the related experiments that provide this scientific information should be covered. Having good knowledge of this, types of measurements and inspection should be studied to assess real-life constructions. The last part of the literature study should contain methods of repair or improvement, applicable to the situation of the yachting harbour.

These chapters should provide a solid knowledge to visit the harbour, confirm the cause of deterioration and to collect the required information and results. Afterwards, the results can be processed and used for determination of the renovation project. In the end, a repair method should be proposed and described into detail and followed by a cost estimation.

Abstract

First, a literature study about reinforced concrete structures in a marine environment was elaborated. The environment was found very aggressive where the ingress of chlorides defines the durability of the structure, especially the state of the reinforcement bars. The related exposure class is divided in three increasing graduations, respectively: ‘exposed to airborne salt’, ‘permanently submerged’ and ‘tidal, splash and spray zones’. For each term, different design boundaries are imposed by the European standards. Chlorides may be part of the content of concrete ingredients or may enter the structure after curation. The process proceeds via several transport mechanisms, especially diffusion. Several experiments are available to specify this chloride transport. As it is a time-consuming process, accelerated tests have been developed, such as the Rapid Chloride Migration Test. However, they are less accurate and are still a subject of critical reviews.

Inspection is an indispensable part of a good maintenance of the construction. A visible inspection does not require any equipment, supplies an overview of the signs of deterioration and may identify the cause. Based on the results, further tests may be recommended. The available tests are divided by a destructive and a non-destructive feature. These tests give estimations with various accuracies of the superficial and internal parameters of the structure. Aiming to extend the lifecycle of concrete structures in marine environment, various protection systems are developed. The principles consist of methods for prevention of deterioration and repair works. Particular attention was given to the repair mortars, which allow to substitute the deteriorated layer of concrete and provide the reinforcement bars of a new, chloride-free protection.

In a second part of the master thesis, an application of the literature study was conducted to the yachting harbour ‘Marina Internacional’ of Torrevieja. The concrete pontoons are heavily affected by the marine environment and show a lot of visible corrosion damage. First, a general investigation charted the graduation of corrosion for the separated parts of the pontoons. Then, non-destructive tests were conducted to collect detailed information about the current state of the slabs. The evaluation indicated an unsustainable state of the pontoons, thus, a repair method was proposed. It was chosen to remove the exterior concrete layer and to replace it by a repair mortar. Sika®MonoTop-614F was determined as the most ideal repair mortar for the substrate, the scope and the environment of this project. The substance is applied by pouring and requires the setup of a formwork. After curation, the slabs should be installed on increased columns to anticipate the rising sea level.

In the end, an economic analyses of the repair works estimates the costs of the whole repair project. The total budget for execution is dominated by the purchase price of the repair mortar. After the project, the harbour should be sustainable for at least another 30 years of service, on condition of a good execution, an assessment of the indicated parts and a sound inspection on a regular basis.

Resumen

Primero se ha realizado un estudio sobre la literatura de las estructuras de hormigón armado en un ambiente marítimo. El ambiente que se encontró era muy agresivo y la durabilidad de la estructura especialmente la de la armadura está definida por la afección de cloruros. La clase de exposición relacionada está dividida en tres graduaciones crecientes respectivamente: ‘expuesto a las sales aéreas’, ‘permanentemente sumergida’ y ‘zonas de mareas, salpicaduras y oleaje’. Las normas europeas imponen fronteras de diseño para cada una de ellas. Los cloruros pueden formar parte del contenido de los materiales del hormigón o pueden entrar en la estructura después del curado. El proceso continúa a través de varios mecanismos de transporte, especialmente de la difusión. Varios experimentos están accesibles para especificar los detalles del transporte de cloruros. Al ser un proceso largo, se han desarrollado ensayos acelerados, tales como el ‘Rapid Chloride Migration Test’. Sin embargo son menos exactos y aún son objeto de revisiones críticas.

La inspección es una parte indispensable de un buen mantenimiento de la construcción. Una inspección visual no requiere de ninguna herramienta, pues proporciona una vista general de las señales de deterioración y se puede identificar la causa. En base a los resultados, se podrían recomendar ensayos adicionales. Los ensayos accesibles están divididos por su característica destructiva o no destructiva. Estos ensayos dan estimaciones con varias precisiones de los parámetros superficiales e internos de la estructura. Con objeto de ampliar el ciclo vital de las estructuras de hormigón varios sistemas de protección se han desarollado. Los principales consisten en métodos de prevención del deterioro y en trabajos de reparación. Se prestó especial atención a los morteros de reparación, que permiten sustituir la capa de hormigón deteriorado y ofrece a las barras de refuerzo una protección nueva y libre de cloruros.

En la segunda parte del trabajo fin de máster, una aplicación de la literatura estudiada se realizó en el puerto deportivo ‘Marina Internacional’ de Torrevieja. Los pantalanes hormigonados se ven muy afectadas por el ambiente marino y muestran muchos daños de corrosión. Primero, una investigación general trazó la graduación de corrosión de las partes separadas de los pantalanes. Después, pruebas no destructivas se realizaron para recoger información detallada sobre el estado actual de las losas. La evaluación indicó un estado insostenible, por tanto, se propuso un método de reparación. Se eligió remover la capa exterior de hormigón y remplazarlo por un mortero de reparación. Se determinó a Sika®MonoTop-614F como ideal para el substrato, el alcance y el ambiente del proyecto. La sustancia se aplica vertiéndola y requiere de encofrado. Después del curado, las losas deben instalarse en columnas aumentadas para anticipar al aumento del nivel del mar.

Para finalizar, un análisis económico estima los costes del proyecto entero. El presupuesto total está dominado por el precio del mortero de reparación. Después el proyecto el puerto debe ser sostenible por un período mínimo de 30 años de explotación, con la

condición de una buena ejecución, de un análisis de las partes críticas y de una inspección regular.

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Literature study

1 Introduction

The use of concrete in marine structures is pervasive and covers distinctive functions. The largest and more obvious marine concrete structures are port structures, such as quay walls and jetties, and coastal defence structures, such as breakwaters or sea walls. But concrete is also used in less noticeable applications, such as tidal pools and boat ramps. In this sector, concrete can form the main structural components or serve another role such as a weight coating providing stability for submarine pipelines, or as a protective cladding to prevent corrosion.

Marine concrete is subject to some of the harshest conditions in the engineering environment, where service life and structural reliability are crucial factors. Not only the attack from seawater and its components, but also the continuing wave loadings and the abrasive action of bed and suspended loads are present. A wide range of loading scenarios may cause potential deterioration processes, which can then lead to a large number of different effects. This master thesis focusses on the ingress of chlorides, originating from the salt in the sea water. The chlorides do not affect the concrete material, but accelerate the corrosion process of the reinforcement bars. Rust occupies more volume than the original steel and causes spalling of cover concrete. Consequently, the reinforcement loses its protection and the resistance against harmful effects is reduced.

First, a small chapter over concrete parameters is provided as a basis. Subsequently, a chapter about the exposure of reinforced concrete structures in marine environment covers the influence of chlorides in the construction material. Then, experiments, simulating the aggressive environment, are described. The following chapter covers the inspection from the mixing process of concrete to the end of the lifecycle of the construction. Further, protection systems are presented. The second part of the master thesis consists of the investigation of the harbour and a proposal of repair works. In the end, an economic analysis was made of the whole project.

2 Concrete

2.1 Ingredients

“The ingredients of a bad concrete are simply cement, aggregates, water, plasticizer, … Surprisingly the ingredients of a good concrete are exactly the same, it is only the knowledge which make the difference.”

A. Neville

2.1.1 Cement

Cement is a hydraulic binder in powder form, which forms plastic paste when mixed with water. In function of time it hardens gradually and increases strength.

There are different kinds of cement: Portland, Slag, Pozzolanic, etc. The last two examples contain respectively high addition of slag and pozzolana. The hydraulic or pozzolanic additions are used to obtain certain properties such as increased rate of strength development (early strength), increased or reduced heat of hydration, higher resistance to chemical aggressive products factors (Kurdowski, 2014).

In the European standard EN 197–1, another principle was applied where cements are divided on types according to the quantity and kind of mineral additions, see figure 1.1. These rules of division facilitate the classification of cements for concrete production. The fundamental performance properties of cements, being the basis of classification are as follows (Kurdowski, 2014):

- The strength of the mortar after 28 days of hardening (class of cement);
- The rate of strength development: strength after 2 days of hardening, and rarely in the case of 32.5 N cement after 7 days;
- Setting time.

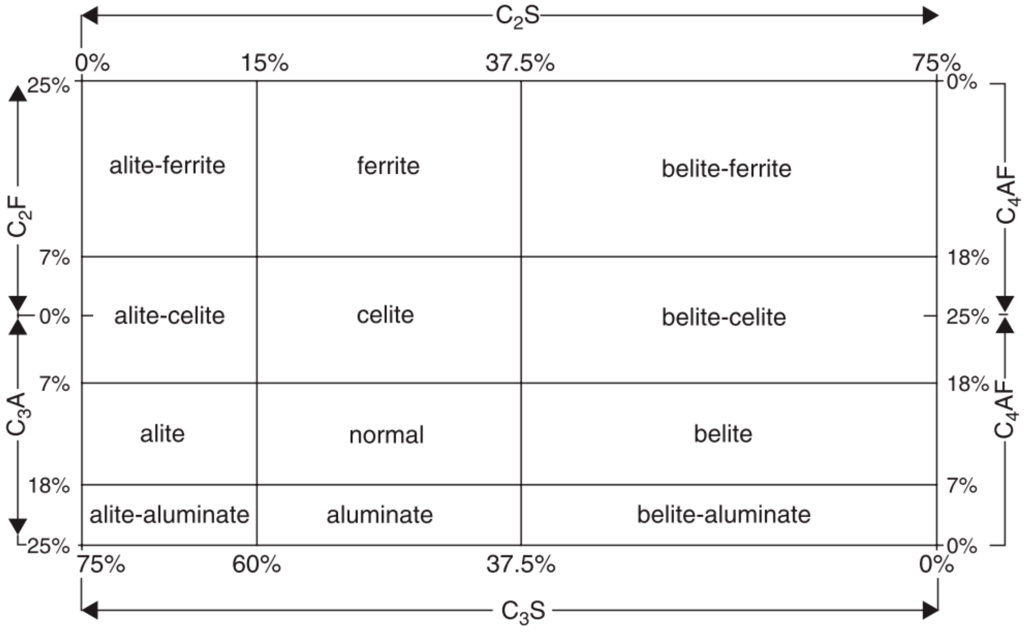


Figure 2.1: Classification of cements based on phase composition (Kurdowski, 2014)

2.1.1.1 Portland cement

Portland cement is a hydraulic binder which mainly exists of lime silicates obtained by a homogenized mixture of CaO, SiO_2 and in lower proportion Al_2O_3 and Fe_2O_3. According to European standards (EN 197-1), the primary components of Portland cement must contain at least 65% CaO and 22% SiO_2. The recommended composition of the product is given in table 1.1.

Primary components	[%]
CaO	65
SiO_2	22
Secundary components	
Al_2O_3	5
Fe_2O_3	3
MgO	1
K_2O-Na_2O	1

Table 2.1: Composition of Portlandcement (Design & Mix, 2015)

2.1.1.2 Slag

Slag is a glass-like by-product left over after a sudden cooling of a melting subvolcanic rock obtained by melted iron ore in the blast furnace. The chemical components of slag are listed below in table 1.2.

Primary components	[%]
CaO	40
SiO_2	35
Al_2O_3	15
MgO	10
Secundary components	
MnO	<1
Fe_2O_3	<1
Sulfides	
CaS	<1

Table 2.2: Composition of slag cement (Design & Mix, 2015)

The resistance of slag in marine environment is no subject in this master degree and will further not be treated.

2.1.2 Aggregates

Aggregates are inert granular materials and include sand, crushed stone and gravel. These ingredients can be natural, artificially manufactured or recycled. The natural type of aggregates consists of minerals, involves mining and has been processed mechanically. Artificial manufactured aggregates have mineral sources as well and they are subject to an industrial process with thermal or other treatments. Recycled aggregates are the result of treated or processed inorganic material that have ever been used for construction.

The aggregates are classified according to the density. Light aggregates belong the category with a maximal density of 2000 kg/m^3 or 1200 kg/m^3 in bulk. The use of these particles is described by the European standard EN 13055-1. Due to the low density, they are only allowed in specific structures. The 'normal' aggregates start with a minimum density of 2000 kg/m^3. A maximum boundary is not defined in the European standard EN 12620. However, EN 13242 specifies a maximum density of 3000 kg/m^3.

2.1.3 Water

Water is a mayor parameter in the hydration reaction and defines the concrete strength of the cured substance. Mixing water is the quantity of water that comes in contact with cement, impacts slump of concrete and is used to determine the water to cement ratio (w/c). Water absorbed by aggregates does not participate in the hydration process. The quantity and quality of the mixing water has important effects on fresh concrete properties, such as setting time and workability, and influences the strength and durability of the hardened concrete.

2.1.4 Additives

Additives are optional ingredients in the concrete mixture, but are indispensable nowadays thanks to mayor benefits. There are several types of additives with different functions.

Plasticizers and superplasticizers are used to improve the processability for a certain water to cement ratio. Keeping this parameter low, results in higher strength capacities. After the first generation with lignosulfates and the second generation with polycondensates of melamine or naphthalene sulfonates, a new generation of products was born: polycarboxylate ethers (PCE). Besides a fundamental difference in the chemical composition of PCE, a dispersion effect by steric hindrance between particles is created.

Another common type of additives are accelerators or retarders. These products allow to adjust the velocity of hydration to a strict time schedule or extreme temperatures. In case of cold weather, accelerators can increase the hydration speed and consequently the temperature. Retarders are useful in warm climates. Calcium chlorides ($CaCl_2$) used to be a common accelerator, but is discouraged by EN 206-1 due to the chloride content. Chloride-free additives are the proposed alternatives. The most effective product considered is calcium and sodium nitrate, being a corrosion inhibitor.

3 Exposure of reinforced concrete to marine environment

3.1 Types and causes of concrete deterioration

Concrete is the most widely used construction material, not only because of its low cost, but also due to its high durability. But material limitations, design and construction practices, and severe exposure conditions can cause concrete to deteriorate, which may result in aesthetic, functional, or structural problems. For this reason, a sound design, execution and curing are unbearable to achieve a long lifespan.

Concrete can deteriorate for a variety of reasons, and concrete damage is often the result of a combination of factors. The following summary discusses potential causes of concrete deterioration and the factors that influence them. There is a distinction between physical and chemical deterioration. (Portland Cement Association, 2002)

3.1.1 Physical deterioration

3.1.1.1 Difference of temperature

Concrete exists of the matrix cement paste and aggregates. In case of different thermal expansion coefficients, the expansion for a certain temperature change will be different. This expansion will cause tension because of the bonding between both parts. The expansion of the aggregates is higher than the expansion for the matrix cement in case of a temperature increase. A temperature decrease leads to tensile stresses in the aggregates with risk of cracks between the elements. (D'Hulster & Duwijn, 2016)

3.1.1.2 Frost

The structure of concrete implies pores which are completely or partial filled with water. The transformation of water into frost involves a volume increase of 9 % which results in high compressive stresses if this expansion is restrained. Due to conductivity, superficial layers freeze in the earliest stadium. When frost occurs underneath the material is obstructed by the frozen layers' present in the external layers. The resulting compression stress can lead to cracks in the outer layer. Because of these cracks, water, chlorides, carbon dioxide and other substances can enter the structure with less effort fostering deterioration at a higher level. (D'Hulster & Duwijn, 2016)

3.1.1.3 Abrasion and erosion

Abrasion damage occurs when the surface of concrete is unable to resist wear caused by rubbing and friction. The phenomenon occurs in an environment of water or free air. As the outer paste of concrete wears, the fine and coarse aggregates are exposed and abrasion and impact will cause additional degradation that is related to aggregate-to-paste bond strength and hardness of the aggregate. Another type of erosion is cavitation. This effect is the formation of vapour cavities in a liquid which is the consequence of turbulent movements of the liquid. When subjected to higher pressure, the voids implode and can generate an intense shock wave. The impact can induce vibes and delamination of the structure. (Portland Cement Association, 2002)

3.1.2 Chemical deterioration

Chemical deterioration is divided in two categories. The first category is deterioration by leaching, supplemented by deterioration due to expansion.

Harmful substances enter the material via diffusion or capillary absorbing and react with the compounds of concrete. Examples of harmful substances are sulphates, alkalis, salts and several organic substances. (D'Hulster & Duwijn, 2016)

The process of leaching is caused by the solvation of concrete ingredients by harmful substances. Calcium hydroxide ($Ca(OH)_2$) in specific is an accessible target. This process leads to erosion inside the material and reduces the strength. Sulphates, such as magnesium sulphate or sodium sulphate, are harmful for the concrete strength. The results of the chemical reactions between the sulphates and the hydration have a larger volume and may lead to cracks in the concrete mass.

Influencing parameters are temperature, pH-value, amount of harmful substances, permeability of the concrete and the type of cement. Common harmful substances are present in groundwater, mined materials, such as aggregates, sea water or an industrial source.

3.1.3 Determination of deterioration types

The different types of deterioration can be excluded on basis of the environment. The possible combinations are indicated in the following table 3.1. More information about the exposure classes is given in 3.3.1.

Type of aggression	**Zone of Exposure**				
	Atmosphere	Splash and spray zone	Tidal zone	Submerged in water	In soil
Corrosion due to chlorides	•	•	•		
Cracks due to freeze-daw cycles	•	•	•		
Erosion due to abrasion		•	•		
Contact of chemical products			•	•	•
Growth of flora			•	•	•

Table 3.1: Types of deterioration that can arise in different exposure zones (Fernández Prada, n.d.)

3.2 Corrosion of reinforcement steel

3.2.1 The corrosion process

Corrosion of reinforcing steel and other embedded metals is the leading cause of deterioration in concrete. Especially in marine environment, where the process is accelerated by chloride ions. When steel corrodes, the resulting rust occupies a volume of approximately six times the volume of the original material. This expansion creates tensile stresses in the concrete, which can eventually cause cracking, delamination, and spalling, see figure 3.1 and 3.2. Moreover, the bars lose their ductility and tensile strength. The rate at which the corrosion process progresses depends on a number of factors, mainly on the 'micro-climate' that immediately surrounds the structure.

Fig 3.1: Concrete affected by corrosion (Portland Cement Association, 2002)

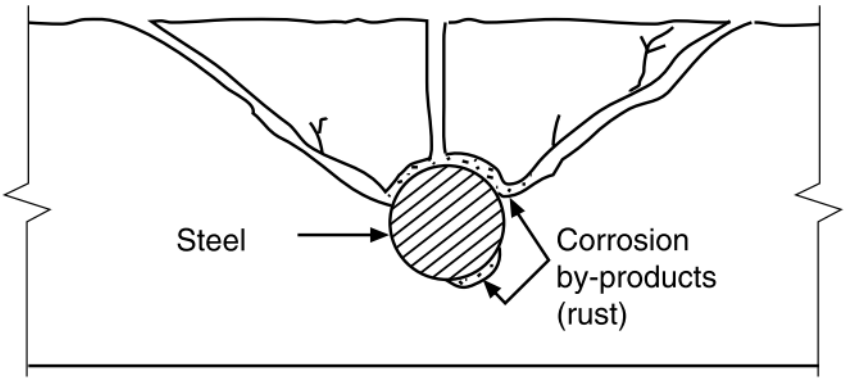


Fig 3.2: Cracking, delamination, and spalling caused by corrosion (Portland Cement Association, 2002)

Steel corrodes because it is not a naturally occurring material. Iron ore is melted and refined to produce steel. Heating the material will transform iron ore into steel and adds energy to the metal. The stored energy causes a thermodynamically unstable position under normal atmospheric conditions and will be released and reverted back to its natural state which is iron oxide or rust. This process is called corrosion. Magnesium, aluminium and iron are metals with a relatively high storage of energy compared to metals such as copper, silver and gold with a more stable position. The table in figure 3.3 lists a few metals in order of decreasing amounts of energy required to convert them from their oxides to metal.

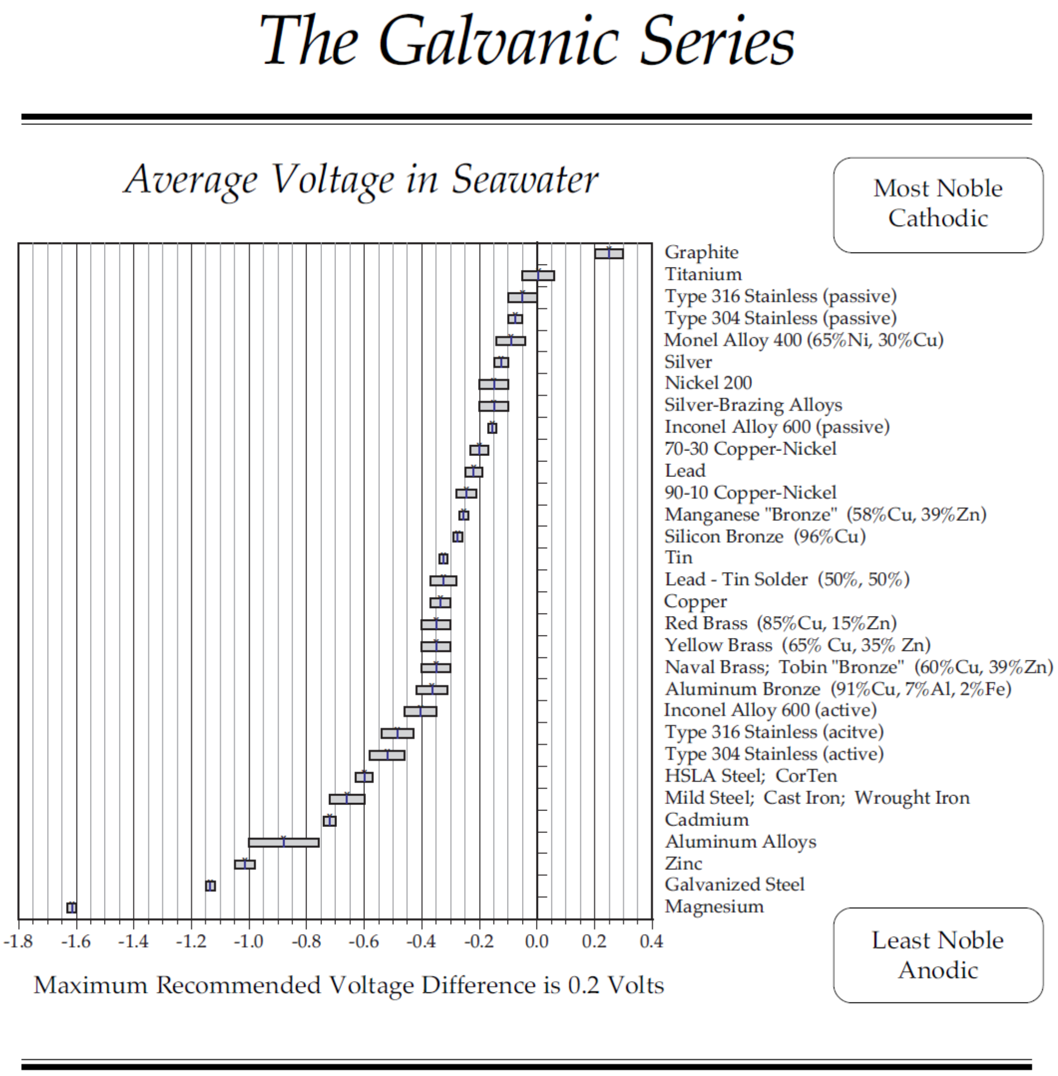


Figure 3.3: The galvanic series

For corrosion to occur the simultaneous presence of metal, moisture and oxygen are required. In chemical terms, they exist of four elements: at least two metals or two locations on a single metal at different energy levels should be present, in addition of an electrolyte and a metallic connection. In reinforced concrete the rebar may have many separate areas at different energy levels. Concrete acts as the electrolyte, and the metallic connection is provided by the rebar itself.

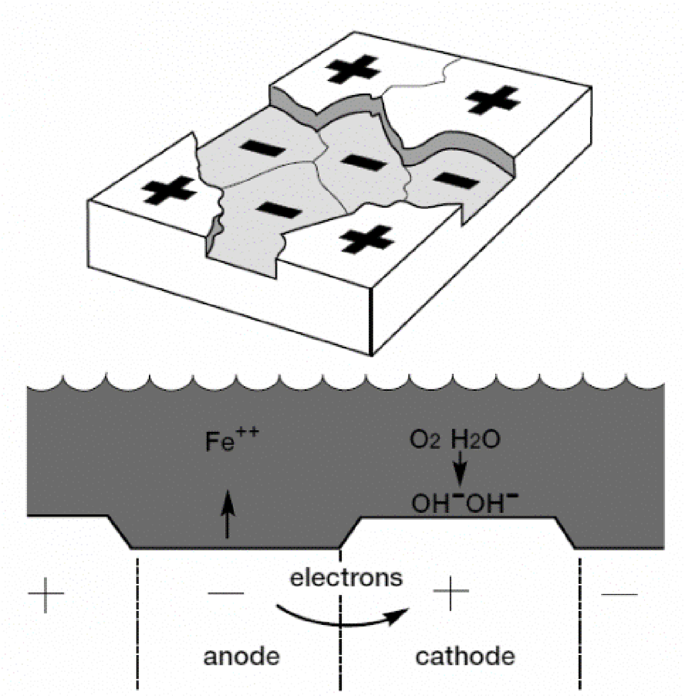


Figure 3.4: Chemical analysis of corrosion ("Steel construction," n.d.)

Figure 3.4 shows a corroding steel bar embedded in concrete divided into positive and negative fields. At active sites on the bar, called anodes, iron atoms lose electrons and move into the surrounding concrete mass as ions. This process is called a half-cell oxidation reaction, or the anodic reaction, and is represented as:

$$2Fe \rightarrow 2Fe^{2+} + 4e^{-} \qquad (1)$$

The electrons remain in the bar and flow to sites called cathodes, where they combine with water and oxygen in the concrete. The reaction at the cathode is called a reduction reaction. A common reduction reaction is:

$$2H_2O + O_2 + 4e^{-} \rightarrow 4OH^{-} \qquad (2)$$

To maintain electrical neutrality, the ferrous ions migrate through the concrete pore water to these cathodic sites where they combine to form iron hydroxides, or rust:

$$2Fe^{2-} + 4OH^{-} \rightarrow 2Fe(OH)_2 \downarrow \qquad (3)$$

This initial, precipitated hydroxide tends to react further with oxygen to form higher oxides. The increase in volume, as the reaction products react further with dissolved oxygen, leads to internal stress within the concrete. This may be sufficient to cause cracking and spalling of the concrete cover equal to the main protection to chloride penetration.

Presence of the four corrosion elements is not sufficient for significant deterioration. Reinforcing steel bars in concrete are protected from corrosion by a thin oxide layer that forms on their surface due to the highly alkaline environment of the surrounding concrete with a pH-value of 12 to 13. This passive film can strongly reduce the corrosion process to a rate of 1/1000 and prevents metal atoms from dissolving. The alkalinity is due to $Ca(OH)_2$, which splits into ions as showed in equation (4) and the protective layer is formed by iron oxides (Fe_2O_3 and Fe_3O_4). (Portland Cement Association, 2002)

$$Ca(OH)_2 \leftrightarrow Ca_2 + (solv) + 2OH^-(solv) \qquad (4)$$

3.2.2 The carbonation process

Carbonation is the reaction between the calcium-based phases in concrete and carbon dioxide from the air, which enters the concrete via the pore network. In the reaction with calcium hydroxide, calcium carbonate is formed. The process is described in the following equations.

$$2NaOH + CO_2 \rightarrow Na_2CO_3 + H_2O \qquad (5)$$

$$2KOH + CO_2 \rightarrow K_2CO_3 + H_2O \qquad (6)$$

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \qquad (7)$$

This reaction reduces the pH of the pore solution to as low as 8.5 at the expense of the passive film on the steel. This can lead to corrosion of the steel bars similar to the effects of the migration of chloride ions. Figure 3.5 shows the carbonation of concrete in case of uniform corrosion and in case of localised corrosion.

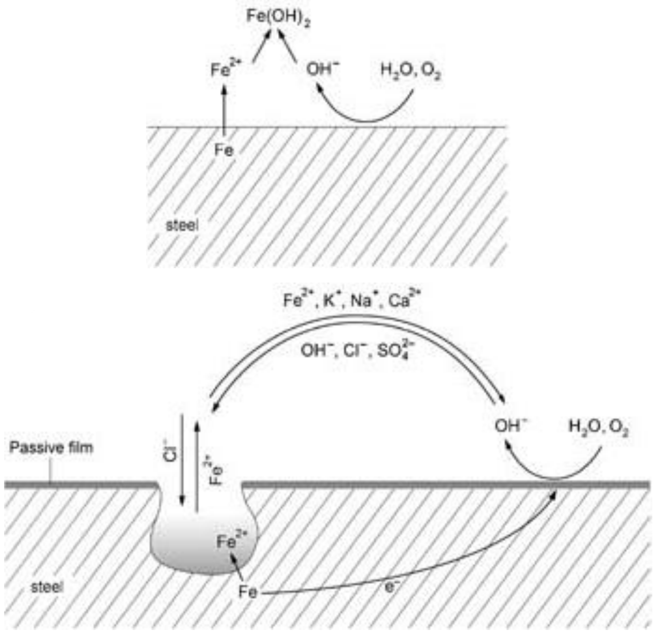


Fig 3.5: Carbonation of concrete for (a) uniform corrosion and (b) localised corrosion

Carbonation is a slow process. In high-quality concrete, it has been estimated that carbonation will proceed at a rate up to 1.0 mm per year. The amount of carbonation is significantly increased in concrete with a high water-to-cement ratio, low cement content, short curing period, low strength, and highly permeable or porous paste.

Influencing environmental parameters are relative humidity, temperature and CO_2 concentration in decreasing order. The highest rates of carbonation occur when the relative humidity is maintained between 50 % and 75 %. Below 25 % relative humidity, the degree of carbonation that takes place is considered insignificant. Above 75 % relative humidity, moisture in the pores restricts the penetration of carbon dioxide (CO_2) (ACI 201 1992). Vulnerable locations to carbonation-induced corrosion are areas of building facades that are exposed to rainfall, shaded from sunlight, and have low concrete cover over the reinforcing steel. (Portland Cement Association, 2002)

3.2.3 Penetration of chlorides

The penetration of chloride ions in the concrete structure will result in a chemical reaction with present iron ions. During the initial stage chlorides are responsible for the weakening of the passive layer on the steel bars. The chlorides enter the concrete structure and dissolve into chloride ions in the pore water. At the moment of reaching the critical concentration in the pore water, the depassivation is launched. The negative chloride ions react with the positive iron chloride as described in equation (8).

$$Fe^{2+} + 2Cl^{-} \rightarrow FeCl_2 \quad (8)$$

The iron chloride reacts with the negative hydroxyl ions and results in negative chloride ions and iron hydroxides. Finally, these iron hydroxides react with oxygen resulting in rust. The reactions are illustrated in figure 3.6.

$$FeCl_2 + 2OH^- \rightarrow 2Cl^- + Fe(OH)_2 \tag{9}$$

$$4Fe(OH)_2 + O_2 \rightarrow 2Fe_2O_3 + 4H_2O \tag{10}$$

During the pathway, chloride ions function as a catalyst and increase the rate of the chemical reaction while the required activation energy is decreased. Since they are not consumed in the catalysed reaction, catalysts can continue to act repeatedly which results in pitting corrosion. This in contrast to uniform corrosion in case of carbonation. Putting corrosion diminishes the cross section of the rebar producing local, increased stresses. Such effect can lead to failure of the rebar.

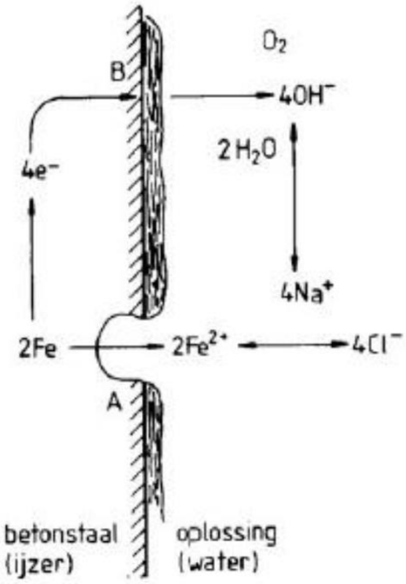


Figure 3.6: Putting corrosion by chlorides (Lammertijn, 2007)

3.3 Standardisation

3.3.1 Exposure classes

The durability of concrete depends on the environment. The European norm NBN EN 206 (2014) divided the different types in classes and subclasses ranked by increasing risk of corrosion, as seen in table 3.2. The fourth class deals with the severe marine environment. The classes are indicated with following characters: X (eXposure); followed by a character which refers to the type of deterioration: C = Carbonation, D = De-icing salt, S = Sea water, F = Freeze/Thaw Attack, A = Chemical Attack.

Class designation	Description of the environment
1/ No risk of corrosion or attack	
X0	For concrete without reinforcement or embedded metal: All exposures except where there is freeze/thaw, abrasion or chemical attack. For concrete with reinforcement or embedded metal: very dry
2/ Corrosion induced by carbonation	
Where concrete containing reinforcement or other embedded metal is exposed to air and moisture, the exposure shall be classified as follows:	
XC1	Dry or permanently wet
XC2	Wet, rarely dry
XC3	Moderate humidity
XC4	Cyclic wet and dry
3/ Corrosion Induced by chlorides other than from sea water	
Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including de-icing salts, from sources other than from sea water, the exposure shall be classified as follows:	
XD1	Moderate humidity
XD2	Wet, rarely dry
XD3	Cyclic wet and dry
4/ Corrosion induced by chlorides from sea water	
Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from seawater or air carrying salt originating from sea water, the exposure shall be classified as follows:	
XS1	Exposed to airborne salt but not in direct contact with sea water
XS2	Permanently submerged
XS3	Tidal, splash and spray zones
5/ Freeze/thaw attack with or without de-icing agents	
Where concrete is exposed to significant attack by freeze/thaw cycles while being wet, the exposure shall be classified as follows:	

XF1	Moderate water saturation, without de-icing agent
XF2	Moderate water saturation, with de-icing agent
XF3	High water saturation, without de-icing agent
XF4	High water saturation, with de-icing agent or sea water
6/ Chemical attack	
Where concrete is exposed to chemical attack from natural soils and ground water, the exposure shall be classified as follows:	
XA1	Slightly aggressive chemical environment
XA2	Moderately aggressive chemical environment
XA3	Highly aggressive chemical environment

Table 3.2: Exposure classes ("NBN EN 206," 2014)

Design parameters as maximum water-cement ratio (w/c), minimum strength class, minimum cement content or minimum air content are defined for each exposure class in NBN EN 206. These values assume a design working life of 50 years and may differ according to the national appendixes.

The fourth class 'Corrosion Induced by chlorides from sea water' is divided in three subclasses ranked according to the level risk of corrosion:

- Exposed to airborne salt but not in direct contact with sea water (XS1);
- Permanently submerged (XS2);
- Tidal, splash and spray zones (XS3).

These classes will be treated more into detail in chapter 3.4.1.2.

3.3.2 Concrete cover

The concrete cover c_{nom} is the perpendicular distance between the surface of the reinforcement bars to the nearest concrete surface. This is the main protection of the reinforcement and buffers the ingress of contaminants, such as chlorides. For this reason, a strict calculation method is imposed by the European standards for every exposure class. The calculation starts with the equation (11).

$$c_{nom} = c_{min} + \Delta c_{dev} \tag{11}$$

Where

c_{nom}	Concrete cover	[mm]
c_{min}	Minimum concrete cover	[mm]
Δc_{dev}	Allowed deviation on the concrete cover	[mm]

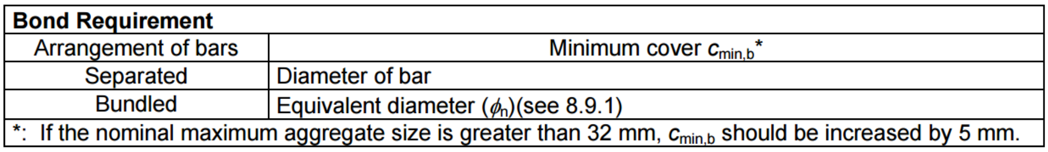
The minimum concrete cover c_{min} is determined by equation (12) and bounded by the following parameters:

$$c_{min} = max\,\{c_{min,b};\ c_{min,dur} + \Delta c_{dur,y} - \Delta c_{dur,st} - \Delta c_{dur,add};\ 10\ mm\} \tag{12}$$

Where

$c_{min,b}$	Minimum concrete cover	[mm]
$c_{min,dur}$	Minimum concrete cover due to environmental conditions	[mm]
$\Delta c_{dur,y}$	Additive safety element	[mm]
$\Delta c_{dur,st}$	Reduction of minimum cover for use of stainless steel	[mm]
$\Delta c_{dur,add}$	Reduction of minimum cover for use of additional protection	[mm]

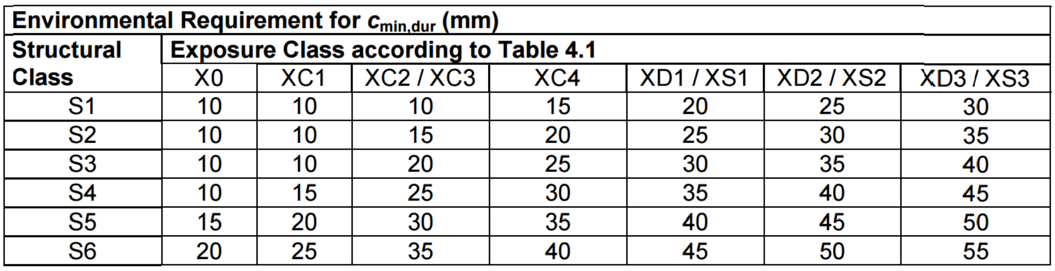
The absolute minimum value for c_{min} is 10 mm. $c_{min,b}$ is determined by environmental conditions and should be deduced of the table in figure 3.1.



Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $c_{min,b}$ should be increased by 5 mm.	

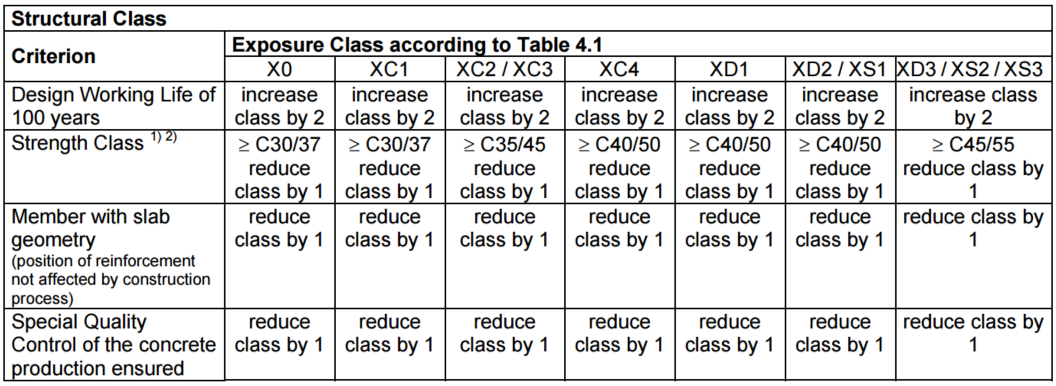
Figure 3.1: Minimum cover, $c_{min,B}$, requirements with regard to bond (NBN EN 1992-1-1)

The parameter $c_{min,dur}$ involves the structural class of the whole structure. The higher the design working life, the more concrete cover should be applied. Specifications are given in the table of figure 3.2 and may vary depending on the national appendix. Depending on criteria, such as design working life, strength class, member with slab geometry and special quality control of the concrete production, structural classes should be increased or reduced. A table is added to figure 3.3.



Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Figure 3.2: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080 (NBN EN 1992-1-1)



Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class 1) 2)	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Figure 3.3: Recommended structural classification (NBN EN 1992-1-1)

3.4 Chlorides

3.4.1 Origin

Chlorides in a concrete structure may come from two sources. One of them is as a contaminant of ingredients, such as water, cement or aggregates, in the fresh concrete or as a chemical additive. Another source is external factors which enter the concrete mass via several transport mechanisms. In this case, chlorides occur as ions in a liquid faze. The content of chlorides entering the structure depends on the environment in close surrounding. Common sources are de-icing salts along road infrastructures and sea water close to the shore. The following paragraphs describe the various transport systems of chlorides in concrete.

3.4.1.1 Chloride content concrete

The chloride content of the concrete ingredients is limited to prevent corrosion. Chlorides can occur in any raw material needed for the concrete mixture: cement, aggregates, mixing water, additives and fillers. Logically, aggregates exploited from marine environment will contain more chlorides. Limited values are mentioned in the standards, but not for all the available products. To assess the maximum content of chlorides, one must sum up the contribution of the chloride content of all the added ingredients separately. The maximum chloride content is expressed by the mass of cement and indicated by a chloride content class in table 3.3.

Type	Chloride content class	Maximum Cl^- content by mass of cement [%]
Not containing steel reinforcement or other embedded metal with the exception of corrosion resisting lifting devices	Cl 1,00	1,00
Containing steel reinforcement or other embedded metal	Cl 0,20 Cl 0,40	0,20 0,40
Containing prestressing steel reinforcement in direct contact with concrete	Cl 0,10 Cl 0,20	0,10 0,20

Table 3.3: Maximum chloride content of concrete (NBN EN 206)

It is not allowed to add calcium chloride and chloride based admixtures to the concrete mixture containing any type of steel reinforcement or other steel elements.

3.4.1.2 External supply of chlorides

The European standard imposes three types of exposure to chlorides of sea water, see chapter 3.3.1, in increasing order of aggressiveness:

- Exposed to airborne salt but not in direct contact with sea water (XS1);
- Permanently submerged (XS2);
- Tidal, splash and spray zones (XS3).

The first category (XS1) implies the interaction between chlorides from marine aerosol with reinforced concrete. The marine aerosol is mainly generated along the seashore by breaking waves, where air and sea water are mixed with lots of energy. The result is a high concentration of marine particles in the atmosphere subject to air flow. When wind speed increases, there is an increase in both the number of generated, marine particles and the percentage of larger drops in the aerosol spectrum. As a result, salt concentrations increase exponentially with wind speed. (Meira, Andrade, Padaratz, Alonso, & Borba, 2007).

Parts of concrete structures that are permanently submerged in sea water belong to the second category (XS2). These parts are exposed to the highest concentration of chlorides compared to the remaining exposure classes, however, this type is not considered as the environment with the highest risk of corrosion. This due to lack of oxygen under the water surface. Still, the corrosion process may occur consuming the dissolved oxygen present in the concrete structure. The O_2 molecules are transported by diffusion through the wet concrete cover. The deeper under the water level, the less the concentration of oxygen and the lower the risk for corrosion. Examples of structures in this category are substructures of a bridge, drilling platforms and quay walls.

The cyclic wet and dry category is evaluated with the highest risk of corrosion, having all the main factors in supply for the chemical reaction in repetitive cycles. This category is called 'Tidal, splash and spray zones' (XS3). The progress of corrosion for reinforced concrete in this environment is found three to five times faster than equal parts submerged in sea water. The more aggressive the environment, the more severe and the less superficial the deterioration occurs (Ille, Radić, & Bleiziffer, 2013). This is illustrated in figure 3.4. Structural examples are multi-storied condominiums, coastal protecting blocks above the average sea level. This area is usually bounded by the low tide mark and stretches out over 2 to 4 m depending on local factors. Apart from the high concentration of chloride ions, the electrical conductivity of the concrete in this region is significant higher than in regions belonging to the other categories. (Sohanghpurwala & Scannell, 1994)

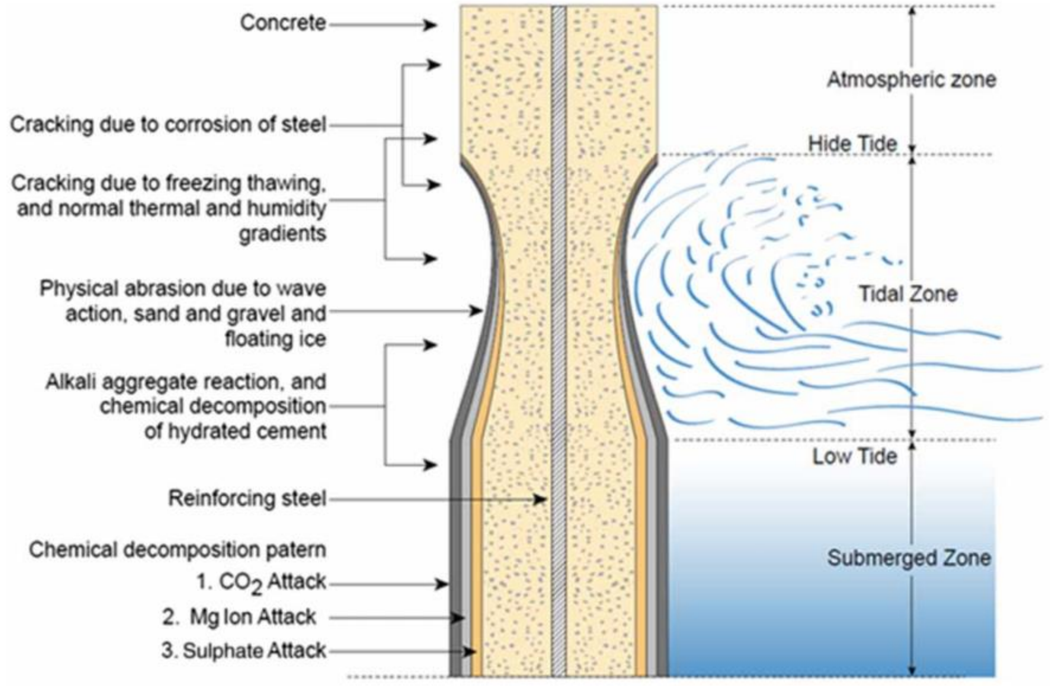


Figure 3.4: Types of deterioration for the three categories of marine environment (Ille et al., 2013)

3.4.2 Free and bound chlorides

Chloride binding is a complicated phenomenon during chloride transport in concrete. When the pores of the concrete are filled with water, they create an optimal transport system for the chlorides. Once inside the structure, a part of the chloride ions will bind with the matrix cement. These are called bound chlorides. A small fraction is bond physically by the cement which means the chlorides are adsorbed by C-S-H. Another part is chemically bound to the cement hydration products. The remaining particles are free chlorides. (Yuan, Deng, Shi, & De Schutter, 2013) It is generally assumed that only free chloride ions can promote the corrosion process of steel reinforcement. On the other hand, free chlorides can be gradually transformed into bound chlorides.

3.4.3 Chloride threshold level

The chloride threshold level (CLT) is commonly defined in two different ways. From a scientific point of view, CTL can be defined as the chloride content required for depassivation of the steel. From a practical engineering point of view, CTL is usually the chloride content associated with visible deterioration of the reinforced concrete structure. (Neves, Vicente, Castela, & Montemor, 2015)

This parameter only involves free chlorides as bound chlorides which do not cooperate in the corrosion process. The relationship between free and bound chloride ions over a range of chloride concentrations at a given temperature are known as the chloride binding isotherms. There are several types of binding isotherm to describe the relationship. Further investigation showed that no single binding isotherm can accurately measure this relationship. (Yuan et al., 2013)

The CLT depends of several parameters which makes it impossible to define one precise number that activates the corrosion process. Figure 3.5 shows the idea behind the risk of corrosion in relation to the cement mass, environmental factors and the state of the concrete. The CLT strongly depends on carbonation. In case of carbonated concrete, there is no passivating layer to affect. Another accelerating factor is the release of chemically bonded chlorides, which do not participate in the corrosion process, to the concrete which increases the content of free chlorides. (Dooms, Pollet, & Mosselmans, 2008)

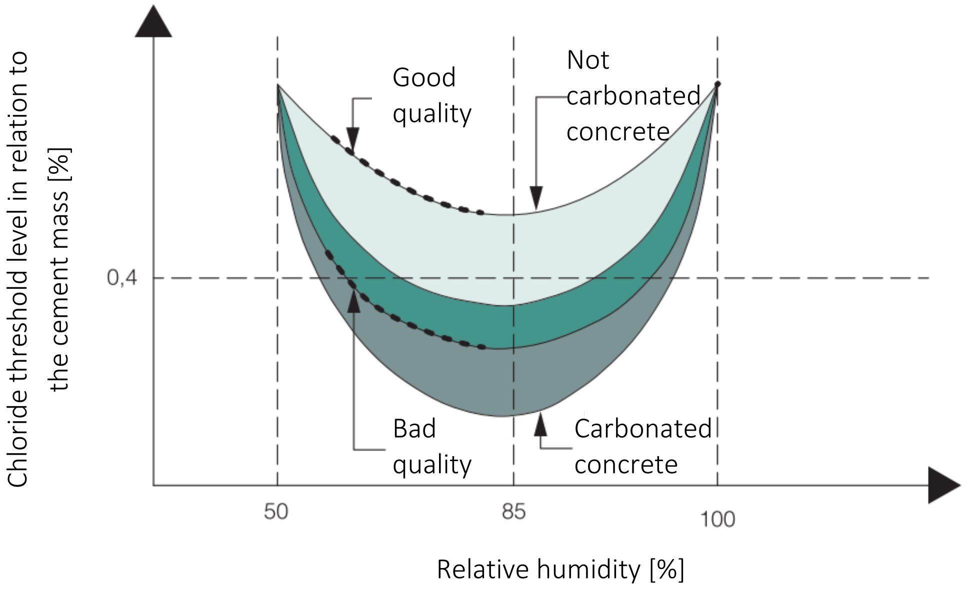


Figure 3.5: Chloride threshold level in relation to the cement mass according to environmental factors (Dooms et al., 2008)

Table 3.4 and Table 3.5 represent an overview of the CLT in relation to the cement mass in case of not carbonated concrete. One can make a distinction between two situations:

- Chlorides added to the concrete mixture;
- Chlorides that have entered the cured concrete.

Higher values of the CLT are found for chlorides that have been added in the concrete mixture. This phenomenon is a logical result of the fact that those ions will partly bond chemically to the cement particles.

[%] added Cl-ions in relation to the cement mass	[%] added Cl-ions in relation to the concrete mass	Risk of corrosion
< 0.6	< 0.075	Low
0.6 - 1.0	0.075 - 0.125	Average
> 1.0	> 0.125	High

Table 3.4: Chloride threshold level in relation to the cement- and concrete mass in not carbonated concrete for chlorides added to the concrete while mixing. (Dooms et al., 2008)

[%] entered Cl-ions in relation to the cement mass	[%] entered Cl-ions in relation to the concrete mass	Risk of corrosion
< 0.4	< 0.05	Low
0.4 - 1.0	0.05 - 0.125	Average
> 1.0	> 0.125	High

Table 3.5: Chloride threshold level in relation to the cement- and concrete mass in not carbonated concrete for chlorides who have entered the cured concrete. (Dooms et al., 2008)

3.5 Transport mechanisms

3.5.1 Diffusion

Diffusion is a net movement of particles stimulated by a difference in concentration of the particles in a solvation. Due to marine environment, a difference in chemical potential will occur between the outer face and the inner structure and induce a migration of ions towards the region of low potential. It is considered the predominant mode of transporting chloride ions in saturated concrete and for this reason the transport mechanism will be covered into detail.

3.5.1.1 General principle

A liquid system with diffusants, i.e. chloride ions, is shown in
Figure 3.6. The chemical potential of the chloride ions is given in the following equation:

$$\mu = \mu_0 + RTln(\gamma \cdot c) \tag{13}$$

Where

μ	Chemical potential	[J/mol]
μ_0	Standard chemical potential	[J/mol]
R	Gas constant, 8.314	[J/mol K]
T	Absolute temperature	[K]
γ	Activity coefficient	[-]
c	Concentration of free chloride in general, by volume of solution	$[kg_{Cl}/m^3]$

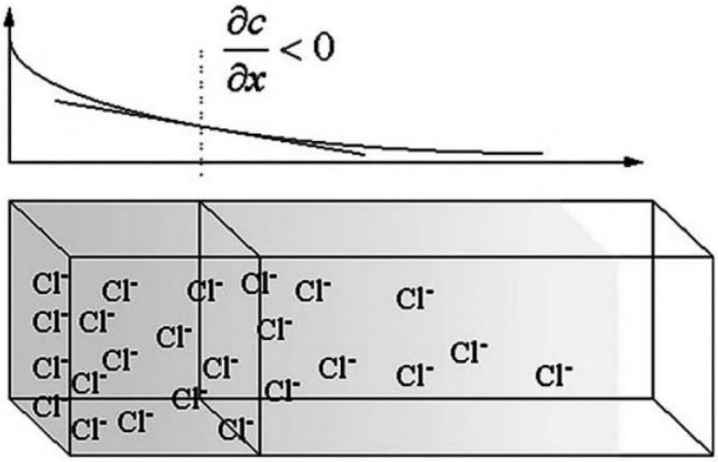


Figure 3.6: Illustration of the concentration gradient of the diffusion of chloride ions in a pure solution system (Luping et al., 2012)

The chemical potential gradient allows ions to move in one direction towards the gradient. As chloride ions or anions cannot exist alone, they are surrounded by cations. Between the ions appears an electrical field ϕ' which tends to draw back the anions. One can describe the average velocity of the chloride ion movement v as follows: (Luping, Nilsson, & Basheer, 2012)

$$v = -B\frac{\partial \mu}{\partial x} + u'\frac{\partial \Phi'}{\partial x} = -BRT\left(\frac{\partial \ln c}{\partial x} + \frac{\partial \ln \gamma}{\partial x}\right) + u'\frac{\partial \Phi'}{\partial x} \tag{14}$$

or

$$v = -BRT\left(1 + \frac{\partial \ln \gamma}{\partial \ln c}\right) + u'\frac{\partial \Phi'}{\partial x} \tag{15}$$

Where

v	Average diffusion velocity of ions	[m/s]
B	Proportionality factor	[-]
x	Distance variable	[m]
ϕ'	Electrical potential	[V]
u'	Ion mobility	$[m^2/(s*V)]$

One combines BRT = D where D is called the diffusion coefficient that describes the flow rate of the ions through a unit area of solution. This physical parameter is known as flux.

$$J_d = c \cdot v = -D \frac{c\,\partial \ln(y)}{\partial x}\left(1 + \frac{\partial \ln y}{\partial \ln c}\right) + cu' \frac{\partial \Phi'}{\partial x} \tag{16}$$

J_d represents the diffusion flux of ions and x the travelled distance in equation (16). Flux is the rate of flow of particles through a specified area per unit of time. In this case only flux in one dimension has been taken in account.

$$J_d = -D \frac{\partial c}{\partial x}\left(1 + \frac{c}{\gamma} \cdot \frac{\partial \gamma}{\partial c}\right) + cu' \frac{\partial \Phi'}{\partial x} \tag{17}$$

The free chloride concentration (∂c/∂x) is always negative, as the concentration decreases for increasing distance. This is illustrated in figure 3.6. As a result, the diffusion flux remains a positive value. The activity coefficient γ and the counter electrical field ϕ’ are two parameters depending on the free chloride concentration and other ions present in the solution. Hence, the following terms are difficult to solve.

$$\frac{c}{\gamma}\frac{\partial y}{\partial c} \quad and \quad cu' \frac{\partial \Phi'}{\partial x} \tag{18}$$

For this reason, these terms are often neglected. The simplified equation is the general form of Fick’s law also known as Fick’s first law. This equation describes a steady-state equilibrium of the chloride concentration in every point of the structure at any moment. The indices of the diffusion coefficient emphasize the steady-state term. (Ustabas, 2012)

$$J_d = -D_{ssd} \frac{\partial c}{\partial x} \tag{19}$$

De flux of chloride depends on de depth in the concrete structure. The principle of mass conservation states that a change of flux in an infinitesimal small piece of concrete dx will induce a change of chloride content, as shown in figure 3.7. The change of the total amount of chlorides per unit of time will be equal to the change of flux per unit of distance.

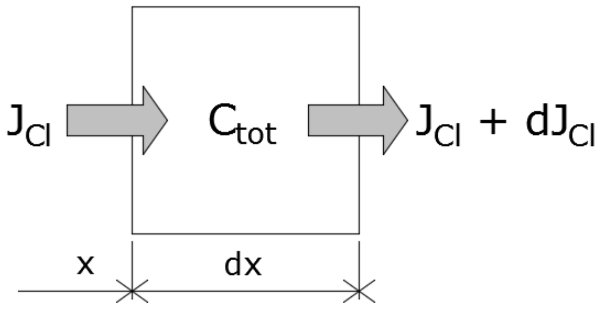


Figure 3.7: Illustration of the change of flux in dx

$$\frac{\partial J}{\partial x} = \frac{\delta C}{\delta t} \tag{20}$$

Substitution of equation (19) in (20) provides:

$$\frac{\delta}{\delta x}(-D_{nssd}\frac{\delta C}{\delta x}) = -\frac{\delta C}{\delta t} \tag{21}$$

Fick's second law treats the transport of material by diffusion according to surface concentration and assumes the following: x > Δx and D is constant.

$$\frac{\delta C}{\delta t} = D_{nssd}\frac{\delta^2 C}{\delta x^2} \tag{22}$$

Due to negligence of the diffusion of bound chlorides, some further adaptions are required. Bound ions do not participate in the diffusion process but the particles covered some distance to the location where the bonding took place. For this reason, all ions need to be taken in account. The equation describes the migration of particles through a homogenic medium, although concrete is a heterogenic medium. The ions only move through the pores of the structure. A last remark is the fact that the movement of negative ions in one direction induces a flow of positive ions in the opposite direction, which is known as electricity current. The phenomenon reduces the diffusion of negative ions and has been neglected in the equation due to complexity. (D'Hulster & Duwijn, 2016)

A common adaption of Fick's second law describes the penetration of chloride ions in a non-steady-state. A non-steady-state method is based on a chloride ingression profile and a steady-state method is based on a constant flow of chlorides through a specimen. The first method acts under following boundaries.

- $C = C_i = 0$ for $x > 0$ and $t = 0$: initial concentration of chlorides
- $C = C_s$ for $x = 0$ and $t > 0$: concentration of chlorides at the surface

The assumptions deliver the following equation:

$$\frac{C(x,t)}{Cs} = 1 - erf\left(\frac{x}{\sqrt{4D_{nssd}t}}\right) \tag{23}$$

For an initial chloride concentration apart from 0, equation (23) is formulated as follows:

$$C(x,t) = Ci + (\mathrm{Cs}\Delta\mathrm{x} - \mathrm{Ci}) * (1 - \mathrm{erf}\left(\frac{x - \Delta x}{\sqrt{4D_{nssd}t}}\right)) \tag{24}$$

Where

C(x,t)	Chloride content at average depth x and exposure time t	[% of m_c]
C_i	Initial chloride content	[% of m_c]
$C_{s\Delta x}$	Calculated surface chloride content at the convection zone	[% of m_c]

*Δ*x	Depth of the convection zone	[m]
D	Non-steady-state chloride diffusion coefficient	[m²/s]
erf	Error function	[-]
t	Time	[s]
x	Depth below the exposed surface to the mid-point of the ground layer	[m]

(Álava, Tsangouri, De Belie, & De Schutter, 2016)

3.5.1.2 Error function

The error function is an integral function and has its origin in statistics and stochastic calculations. The error function and the complementary function are defined as follows: (Cortés, Company, Jódar, & Ponsoda, 2005)

$$erf(x) = \frac{2}{\sqrt{\pi}} \int_0^x e^{-t^2} dt \tag{25}$$

$$erfc(x) = \frac{2}{\sqrt{\pi}} \int_x^\infty e^{-t^2} dt \tag{26}$$

$$erf(x) + erfc(x) = 1 \tag{27}$$

The error function is calculated with power series, as shown in equation (28):

$$erf(x) = \frac{2}{\sqrt{\pi}} \sum_{n=0}^{\infty} \frac{(-1)^n x^{2n+1}}{n!\,(2n+1) \rightarrow} \tag{28}$$

Both functions, the error function and the complementary function, are displayed graphically in figure 3.8. Extreme values for parameter x can result in a simplification of the equation (25). The results are shown in table 3.6.

	$\boldsymbol{erf(x) = \frac{2}{\sqrt{\pi}} \int_0^x e^{-t^2} dt}$	$\boldsymbol{erfc(x) = \frac{2}{\sqrt{\pi}} \int_x^\infty e^{-t^2} dt}$
x → 0	0	1
x → ∞	1	0

Table 3.6 Extreme values of the error function

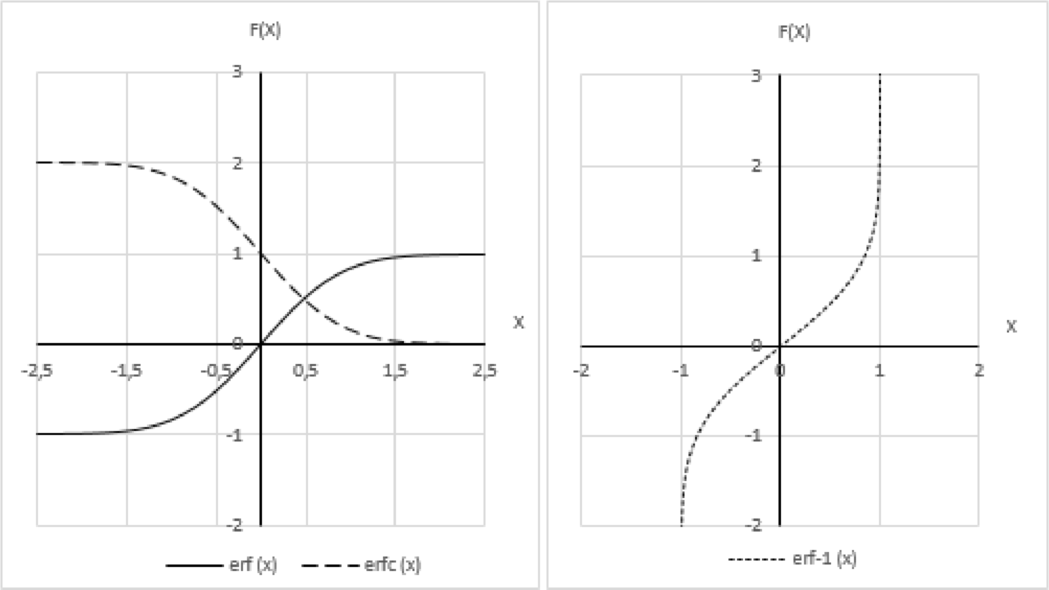


Figure 3.8 Graphical representation - erf(x) and erfc(x)

3.5.1.3 Ageing

The influence of the age of the concrete on diffusion is not clear yet. The microstructure tends to compact in function of time by a continuous hydration of the cement. On the other hand, the environment tends to block the pores. Both processes reduce the diffusion coefficient and affect the durability of the concrete in a positive way.

In mathematical terms, the diffusion coefficient will be lower for t_2 than for t_1 if $t_1 < t_2$. The equation is as follows:

$$D_{nssd}(t) = k_e D_{nssd}(t_0) \left(\frac{t_0}{t}\right)^{\propto_A} \tag{29}$$

With

$$k_e = e^{\left(b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_{real}}\right)\right)} \tag{30}$$

Where

k_e	Environmental factor	[-]
b_e	Regression factor, between 3500 K and 5500 K. The variable is normal distributed and has a mean value of 4800 K and a standard deviation σ = 700 K	[K]
T_{ref}	Referention temperature: 273.15 K	[K]
T_{real}	Temperature of the structure and the environment	[K]
α_A	Ageing exponent for an apparent diffusion coefficient for method	[-]
t	Time of exposure	[year]
t_0	Time reference: 28 days or 0.0767 year	[year]
$D_{nssd}(t)$	Apparent diffusion coefficient as mean value during the period from t_0 to t	[m²/s]

$D_{nssd}(t_0)$ Apparent diffusion coefficient on a reference moment 't_0' [m²/s]

A clear distinction between the used diffusion coefficients is needed. $D_{nssd}(t_0)$ is the diffusion coefficient measured at time t_0 with test method NT Build 492 set by Nordtest. During a period t_0, the diffusion coefficient changes. This makes $D_{nssd}(t_0)$ a constant, mean value for the diffusion coefficient over the period t_0. The same applies for $D_{nssd}(t)$ which is the constant, mean value over the period t. Both factors are apparent diffusion coefficients and indicate the exposure time starting at the time of the removal of the formwork. (Luping et al., 2012)

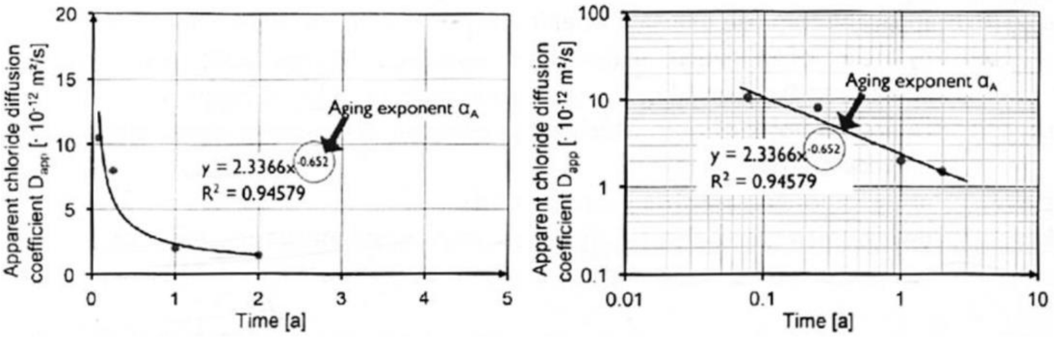


Figure 3.9: Graphic illustration of $D_{nssd}(t)$ en $D_{nssd}(t_0)$ (Fédération internationale du béton, 2006)

Figure 3.9 shows the relation between both diffusion coefficients. The graph on the left displays the function on a normal scale. The graph on the right shows the function on a double logarithmic scale. To make predictions of the progress, it is needed to find the ageing exponent α_A which can be defined with diffusion coefficients on two different moments. These parameters can be determined with experimental tests on existing structures or in the laboratory. Diffusion tests are time-consuming and very expensive. Fib Bulletin 76 (2015) imposes that the duration of tests should be conducted over a period of at least 2 years, to achieve an acceptable accuracy of the parameters.

The test method will be treated into detail in chapter 4.1. This method has been developed to cope with the time-consuming effect and describes an accelerating test. The result is a more economical and rapid migration test.

3.5.2 Capillary suction

Sorption is the result of capillary movement in the pores of concrete due to surface tension. The capillary suction can only take place in partially dry concrete as the tension disappears in completely dry or saturated concrete.

In accordance with the equation of Washburn (1921), the capillary absorption mechanism of a porous material can be simplified to a one-dimensional process as a set of vertical, cylindrical capillaries. Assuming the set of capillaries perpendicular to the surface of the liquid to be absorbed, the mass of fluid absorbed per unit area normal to the direction of flow normalized mass m_s can be expressed as follows: (Navas, Alc, Fern, & Mart, 2011)

$$m_s = \rho \left(\frac{\gamma}{\mu} \frac{\cos \vartheta}{2} \right)^{1/2} \left(\frac{\varepsilon'}{\tau} r^{1/2} \right) t^{1/2} \tag{31}$$

Where

ρ	Fluid density	[kg/m³]
γ	Surface tension	[kg/s²]
μ	Viscosity	[kg/(s·m)]
θ	Contact angle	[rad]
ε'	Effective porosity	[-]
r	Average pore radius	[m]
t	Time	[s]
τ	Tortuosity factor	[-]

The tortuosity is a dimensionless correction parameter which connects the properties derived from the theoretical model with the experimental properties and takes imperfections as the micro-heterogeneities of the material and the fact that the pores are not completely straight, uniform and vertical into account. The tortuosity factor is defined positive as it slows down the absorption process.

Another decreasing factor to be considered into the tortuosity parameter is the 'bottle neck' which consists of two types. The first one is when the pore size is very close to the water molecule size preventing the molecule to enter inside the pore. The second type is when the pore size is very large and exceeds the limit of capillary rising. The mechanism stagnates due to the pressure generated by water interfacial tension and the water-material affinity. Consequently, the tortuosity factor serves as a corrective parameter including all those circumstances decelerating the rise of the liquid in the material regarding the ideal model.

If all the characteristic parameters of the liquid and the interface are grouped into a single constant C and the material is assumed to consist of N different types of pores, it is possible to express equation (31) as the following equation: (Navas et al., 2011)

$$m_s = C \sum_{i=1}^{i=N} \left(\frac{\varepsilon'}{\tau} r^{1/2} \right) t^{1/2} = \left(\sum_{i=1}^{i=N} K_i \right) t^{1/2} = K \cdot t^{1/2} \tag{32}$$

Where K_i is the rate constant of the absorption process for each pore type and K is the global kinetics constant. Considering this, it is possible to obtain an expression of the slope of the curve of normalized mass versus the square root of time, according to figure 3.11, as follows: (Navas et al., 2011)

$$\frac{\partial m_s}{\partial t^{1/2}} = K = C \sum_{i=1}^{i=N} \left(\frac{\varepsilon'}{\tau} r^{1/2} \right) t^{1/2} \tag{33}$$

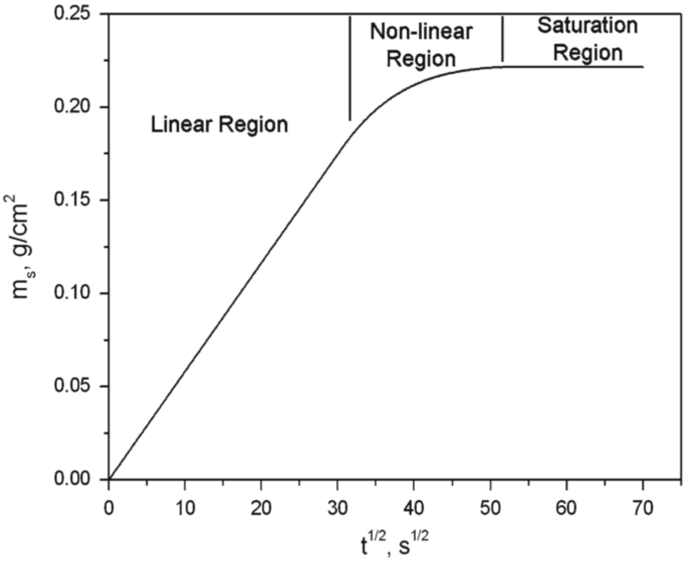


Figure 3.10: Theoretical representation of the normalized mass versus square root of time with the three zones of the capillary absorption curve

In accordance with equation (33), pores with a larger diameter absorb water faster than those with a smaller diameter. For samples with finite height, pores with a larger diameter fill more quickly. Hence, the element corresponding with those pores in the sum becomes zero, the slope of the graph decreases and the non-linear region of the absorption curve (zone 2) begins. This process whereby the slope and the absorption kinetics decreases, continues as the remaining pores are filled. Finally, when all the pores are saturated, the sum in equation (33) is not valid anymore and the saturated region of the absorption curve is reached. Considering this behaviour, it is deduced that the difference between two consecutive values in the non-linear zone of the slope of the absorption curve can be specifically associated with a certain type of pore. This value is determined by the following equation, which defines the difference between two consecutive slope values: (Navas et al., 2011)

$$k_j = K_j - K_{j-1} = C \left(\frac{\varepsilon'_j}{\tau_j} r_j^{1/2} \right) \tag{34}$$

Where

$$C = \rho \left(\frac{\gamma}{\mu} \frac{\cos\theta}{2} \right)^{1/2} \tag{35}$$

As previously indicated, the contact angle is incorporated in the constant C. In the literature, the water contact angle is zero on a clean quartz surface which usually requires polishing or chemical or heating treatments. However, the surface can possibly be contaminated, as for example, by the adsorption of several chemical compounds from air, increasing the values of the angle with 20 – 80°. If the contact angle is higher than 0°, this effect is considered in tortuosity factor, because this factor is a parameter to fit the results of the methodology with the experimental data. (Navas et al., 2011)

3.5.3 Hydrostatical pressure difference

Adding hydrostatical pressure of a solvation containing chlorides on a concrete surface, results in accelerated penetration due to the difference in pressure. Velocity and depth of the chloride penetration increases exponentially for increasing pressure and the bonding of chloride with concrete decreases under pressure. (Jin, Zhao, Gao, & Hou, 2013)

3.5.4 Thermal migration

Higher temperature will result in more kinetic energy for the chloride ions. When a specimen of concrete with a constant, initial chloride concentration is heated, chloride ions will move to the colder parts of the concrete. Experimental tests proved that non-steady-state and the steady-state migration coefficient three to eleven times and three to seven times increases when the temperature is adjusted to 95°C. (Yang & Weng, 2011)

3.5.5 Influencing effects

Several external effects can stimulate or reduce the chloride transport. Some factors are less demanding to control than others and are useful to prevent the reinforced bars from corrosion.

3.5.5.1 Stress Levels

External loads can affect the process of chloride ingress by modifying the microstructure of concrete. Experimental tests showed that the chloride concentration and the chloride diffusion coefficient decreased significantly with the increase of the compressive stress and increased with the increase of the flexural stress. (Wang, Lu, Jin, & Bai, 2011)

3.5.5.2 Temperature

The higher the temperature, the more chlorides enter the concrete mass. The empirical law is valid for capillary suction and diffusion. This last item is a thermodynamic movement so heat provides more energy for the transport of the ions. (D’Hulster & Duwijn, 2016)

3.5.5.3 Age

During the lifespan of the concrete, the hydration process continues and reduces the volume of the free water and the porosity. These factors hamper the transport of the chlorides, as explained in chapter 3.5.1.3.

3.5.5.4 Cracks

Cracks occur due to shrinkage, thermal and mechanical loadings and create an easy entrance for chlorides in the concrete structure. According to the crack width and the crack depth, the chloride diffusivity can strongly increase. A sound design may enhance the distribution and reduce the size of the cracks. (Du, Jin, Zhang, & Li, 2015)

3.5.5.5 Concentration of the chlorides

The gradient of concentration of the chlorides increases for a high concentration of chlorides at the surface of the material. Consequently, the diffusion process is more stimulated and the transport capacity increases. Depending on the location or situation, several types of concentrations have been created with an estimation of the expected concentration of chlorides. The categories in marine environment consist of tidal, splash and spray zones. Examples of types on land are de-icing salts and exposure to chemical products. The quantity of concentration is expressed in mass percentage in relation to the cement mass.

3.6 Electrical migration

3.6.1 Introduction

Experimental tests are essential to achieve a better understanding of the way chloride-induced corrosion of reinforcing steel come about. Several long- and short-term laboratory tests have been created to assess the long-term chloride transport. Long-term experiments are very time-consuming, laborious and expensive and they are more and more evaded as short-term experiments eliminate those disadvantages. These experiments make use of an electrically-accelerated test, such as the Rapid Chloride Migration Test (RCM). It has been developed by Tang (1996) and is released as a guideline in NT Build 492 (1999).

3.6.2 The chloride transport model

The movement of ions can be accelerated by a potential difference in the same way as electrons move through a circuit due to voltage. Chloride ions are negative and are attracted by the positive pole (anode) and positive ions are attracted by the negative pole (cathode).

The extended chloride transport model for the RCM test has been developed by Tang (1996) and is described as follows: (Spiesz & Brouwers, 2015)

$$\frac{\partial c}{\partial t} = \frac{D_0}{1 + \frac{\partial c_b}{\partial c}} \left(\frac{\partial^2 c}{\partial x^2} - \frac{zFU}{RTL} \frac{\partial c}{\partial x} \right) = D_{RCM} \left(\frac{\partial^2 c}{\partial x^2} - \frac{zFU}{RTL} \frac{\partial c}{\partial x} \right) \tag{36}$$

Where

D_{RCM}	Chloride diffusion coefficient	[m²/s]
z	absolute value of ion valence, for chloride: z = 1	[-]
F	Faraday constant, F = 9.648 × 10^4	[C/mol]
U	Absolute value of the applied voltage	[V]
R	Gas constant, R = 8.314	[J/(K·mol)]
T	Temperature	[K]
L	Thickness of the specimen	[m]
x	Average value of the penetration depths	[m]
t	Test duration	[s]
D_0	Intrinsic chloride diffusion coefficient	[m²/s]
c_d	Bound chloride concentration at which the colour changes, c_d ≈ 0.07 N for OPC concrete	[N]
c	Free chloride concentration	[N]

The term ($\partial c_b/\partial c$) is a constant value which implies that no binding or instantaneous linear chloride binding under equilibrium conditions is involved in the chloride migration process. Another term, D_{RCM}, considers the binding in an implicit manner and is described in the following paragraphs. (Spiesz & Brouwers, 2015)

Tang (1996) proposed a solution of the calculation of the non-steady-state migration coefficient of equation (37):

$$D_{RCM} = \frac{RTL}{z\,FU} \frac{x_d - \propto \sqrt{x_d}}{t} \tag{37}$$

With

$$\propto = 2\sqrt{\frac{RTL}{z\,FU}} \cdot erf^{-1}\left(1 - \frac{2c_d}{c_0}\right) \tag{38}$$

Where

x_d	Chloride penetration depth	[m]
α	Laboratory constant	[-]
c_d	Chloride concentration at which the colour changes, c_d ≈ 0.07 N for OPC concrete	[g/dm³]
c_0	Chloride concentration in the catholyte solution, c_0 ≈ 2 N	[g/dm³]

The equation can be simplified as $erf^{-1}\left(1-\frac{2\cdot 0.07}{2}\right)=1.28$ (NT Build 492, 1999):

$$D_{nssm}=\frac{0.0239(273+T)L}{(U-2)t}\left(x_d-0.0238\sqrt{\frac{(273+T)L\,x_d}{U-2}}\right) \tag{39}$$

Where

D_{nssd}	Non-steady-state migration coefficient, x 10^{-12}	[m²/s]
U	Absolute value of the applied voltage	[V]
T	Average value of the initial and final temperatures in the anolyte solution	[°C]
L	Thickness of the specimen	[mm]
x_d	Average value of the penetration depths	[mm]
t	Test duration	[h]

Another model has been developed by Spiesz et al. (2012) and extended the transport model for non-linear chloride binding and concentrations non-equilibrium. The equation is written as follows:

$$\varphi\frac{\partial c}{\partial t}+v\frac{\partial c}{\partial x}=-k\left(c-\left(\frac{C_b}{K_b}\right)^{\frac{1}{n}}\right) \tag{40}$$

$$(1-\varphi)\rho_s\frac{\partial C_b}{\partial t}=k\left(c-\left(\frac{C_b}{K_b}\right)^{\frac{1}{n}}\right) \tag{41}$$

Where

φ	Porosity	[-]
v	Ionic migration velocity	[m/s]
k	Chloride mass transfer rate	[l/s]
ρ_s	Specific density of concrete	[g/dm³]
K_b	Chloride binding capacity	[dm³/g]
n	Chloride binding intensity	[-]
C_b	Bound chloride concentration	[g/g]

The equations (40) and (41) represent respectively the free and the bound chloride concentrations. The ionic migration velocity v is specified as:

$$v=\frac{D_{eff}zFU}{RTL} \tag{42}$$

Where

D_{eff}	Effective chloride diffusion coefficient	[m²/s]

The effective chloride diffusion coefficient in equations (40) is rather used to treat the binding term independently and to express the diffusion coefficient in the terms of the flux per unit area of concrete than only the pore solution. (Spiesz & Brouwers, 2015)

3.6.3 Ageing

Specimens of large amounts of structures with different ages have shown a decreasing trend in the diffusion coefficient that was apparently time depending (CUR Bouw & Infra, 2007). The decrease in velocity of diffusion of chlorides over time is due to the hydration of the cement which reduces the volume of the capillary pores, especially for blast furnace slag or fly ash, and due to curing of the concrete which reduces the pore water volume. The reduction of the diffusion coefficient over time is described in the following empirical relation (43):

$$D_{RCM}(t) = D_{RCM}(t_0)\left(\frac{t_0}{t}\right)^n \tag{43}$$

Where

n	Ageing exponent	[-]
t	Time of exposure	[year]
t_0	Reference of time, often t_0 = 28 days or 0.0767 year	[year]
$D_{RCM}(t)$	RCM-value at time t	[m²/s]
$D_{RCM}(t_0)$	RCM-value at time t_0	[m²/s]

Table 3.7 provides the ageing exponent for diverse types of cement. The values are reserved for specimens in standard conditions according to EN 206-1. These involves a room temperature of 20°C and a curing time of minimum 6 months.

Type of cement	**n_{Cl}**
CEM I	0.25
CEM I met 25-50% S	0.30
CEM III 50-80% S	0.40
CEM I met 21-30% FA	0.80
CEM V/A (25% S + 25% FA)	0.60

Table 3.7: Ageing exponent n_{Cl} for each type of cement

With

S	Slag
FA	Fly ash

4 Experiments

4.1 RCM permeability test

4.1.1 Specifications

The accelerated test developed by Whiting has been approved as standardized method by ASTM (test C1202) and AASHTO (test T227). (Dooms et al., 2008)

4.1.2 Method

A concrete cylinder with a thickness of 5 cm is fitted in a migration cell. One chamber contains a solvation of NaOH (0.1 M) and the other chamber contains sodium chloride (NaCl, 3 % per weight). An electrode has been added to each chamber as seen in figure 4.1. Then, an electric field of 60 V between both electrodes is applied. The total charge transferring through the specimen is measured for 6 hours and expressed in coulomb. Measuring the change of concentration in the anodic chamber is a direct way to define the chloride flux. The experiment is semi-quantitative as there is a distinction between high, average, low, very low and neglectable permeability. The test defines that a larger number of coulombs represents a higher permeability of the concrete to the chlorides. Although the test does not have any direct information about the diffusion of the chlorides, it is considered that the diffusion is related to the total, measured charge. One should keep in mind that the non-steady-state migration coefficient cannot be directly compared with chloride diffusion coefficient obtained from other tests. (Andrade, 1993)

4.1.3 Critical review

The evolution from long term experiments to the Rapid Chloride Permeability Test generates strong benefits, such as time reduction, accompanied by disadvantages. Although the fact has been standardized, investigators still recognize uncertainties:

- The total measured electric current is the result of the migration of all the ions and thus, not only chloride ions;
- The measurement starts before it reaches its steady-state;
- A high voltage can increase the temperature (especially in case of high permeability of the concrete) which stimulates the diffusion process;
- Chemical reactions with the electrodes can change the solvation in the anodic (positive electrode) and the cathodic (negative electrode) chamber. For example, the electrolysis of water generates O_2 and H_2. The use of non-corrosive electrodes, e.g. Pt, can exclude any other form of electrolysis in the cell. When the voltage exceeds a certain level, oxidation of chloride ions may occur.

Controlling the voltage on a lower level is recommended to reduce the heat production. (Andrade, 1993)

4.1.4 Processes in the migration cell

There are two electrode processes. One of them is the metal dissolution which generates oxides at the anolyte. This is only the case when the anode is an oxidable metal, such as copper. In the case of graphite electrodes, the process will be:

$$C \rightarrow CO + CO_2 \quad (44)$$

The other electrode process is the evolution of gases in case of electrolysis of water, in both anolyte and catholyte, generating O_2 and H_2. This evolution will stir the solution in both chambers. The chemical reactions at each electrode are as follows:

- Positive electrode

$$2H_2O \rightarrow 2O_2 + 4H^+ + 4e^- \quad (45)$$

- Negative electrode

$$2H_2O + 2e^- \rightarrow H_2 + 2OH^- \quad (46)$$

Not only water electrolysis, but also oxidation of Cl^- may happen if the voltage is high enough to produce Cl_2 evolution.

- Positive electrode

$$2Cl^- \rightarrow Cl_2 + 2e^- \quad (47)$$

Besides, an oxygen reduction takes place as follows:

- Negative electrode

$$2O_2 + H_2O + 4e^- \rightarrow 4OH^- \quad (48)$$

All these reactions tend to maintain the total electroneutrality of the migration cell as expected by one of the fundamentals of electrochemical reactions.

The third process happening in a cell is the movement of ions in the electrolyte to carry the electricity through the cell and to close the electrical circuit. Therefore, migration is developed and diffusion may appear if this migration leads to concentration differences. As it was mentioned, beside chlorides, all the remaining ions take part in migration in a proportion what is known as their ‘transport or transference number’.

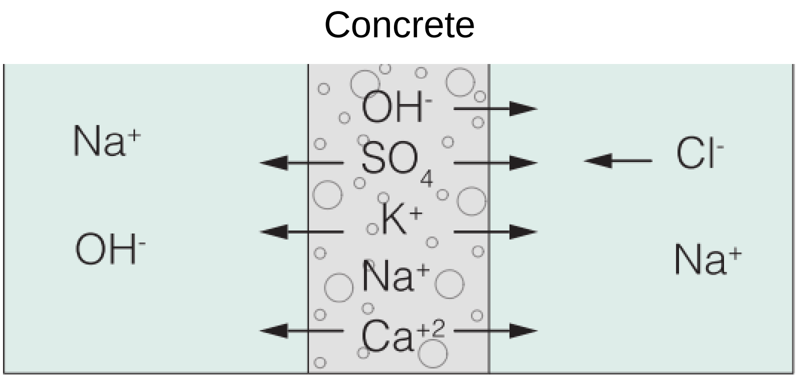
The transference number of an ion moving under the action of an external electrical field is defined by the proportion of the current carried by this ion in relation to the current carried by the rest of the ions. It is formulated as:

$$t_j = \frac{i_j}{i} = \frac{z_j c_j \lambda_j}{\sum zc\lambda} = \frac{\lambda_j}{\Lambda} \tag{49}$$

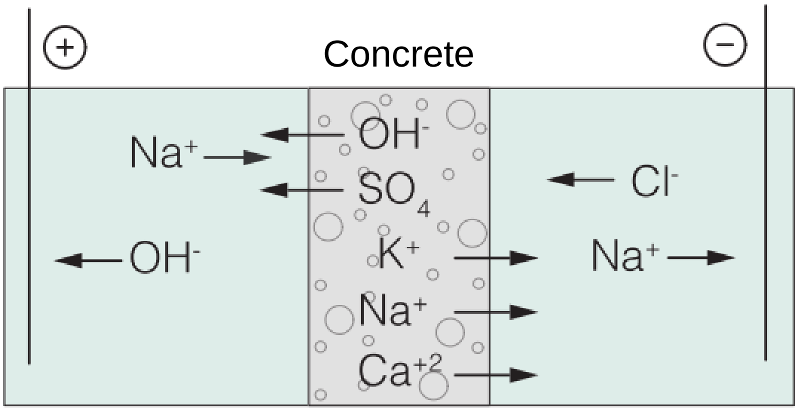
Hence, the transference number is a function of the ionic mobility or the equivalent conductivity. This means that OH^- ions will carry much more proportion of current than Cl^- ions, while acknowledging the ionic conductivity of OH^- is 198.5 cm²/ohm eq and the one of Cl^- is 76.34 cm²/ohm eq. In case of concrete, it means that the main proportion of the current would be taken by OH^- ions and not by the Cl^- ions. The hydroxides might behave as a supporting electrolyte.

For this reason, it is important to calculate the chloride transference numbers when conducting the Rapid Chloride Permeability Test to specify the pure chloride transport quantity. In addition, the chlorides may interact with the C_3A molecules during transport through the concrete. This effect postpones the stationary flow until all reactive sites are saturated. (Andrade, 1993)

A. Diffusion



B. Migration



C. Diffusion + migration

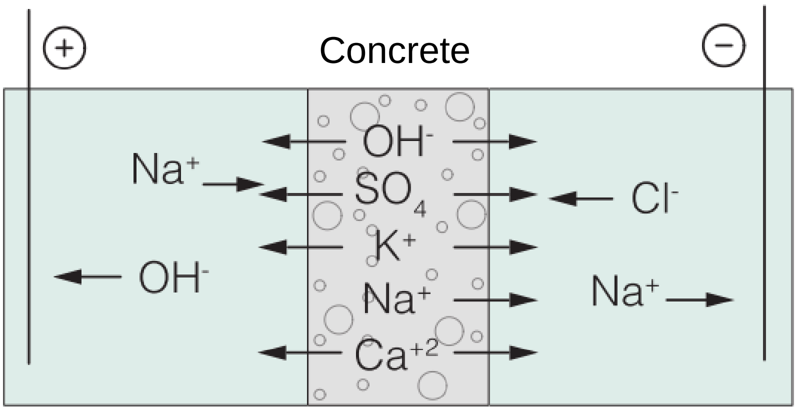


Figure 4.1: Illustration of the mass transport in a concrete cylinder mounted in the migration cell (Dooms et al., 2008)

4.2 Diffusion test

4.2.1 Specifications

The diffusion test has been standardized by Nordtest: NT Build 443 (Application, 1995). The purpose is to define the resistance against chloride ingress using the diffusion coefficient.

The procedure is valid for water-saturated specimens, manufactured and saved in the laboratory, as well as for drilled specimens in situ with a minimum age of 28 days. Visible imperfections are not allowed on the specimens. The cylinder-shaped cores have a diameter of minimum 75 mm and should be larger than the triple equivalent of the size of the biggest aggregate. The length should be minimum 100 mm. At least three specimens should be tested. (Application, 1995)

4.2.2 Method

The specimens are exposed to a NaCl solvation with a high concentration (165 ± 1 g NaCl per dm^3 solvation), which initiates an accelerated ingress of the chlorides. Afterwards, one works layers ordered to ground off the concrete sheets parallel to the exposed face of the specimen. With the concrete layers, the chloride content can be specified according to the standardized method by NordTest: NT Build 208 (Content & Volhard, 1996). This is a method which enables to define the chloride content in cured concrete by solvating the specimen in nitric acid followed by a titration. The chloride content in relation to the dry concrete mass is calculated as follows:

$$m_{Cl^-} = C_m = 3.545 \frac{V_1 N_1 - V_2 N_2}{m} \tag{50}$$

Where

m	Mass of the concrete	[g]
V_1	Volume of the silver nitrite solution	[ml]
N_1	Concentration of the silver nitrite solvation	[mol/l]
V_2	Volume of the ammonium thiocyanate solution	[ml]
N_2	Concentration of the ammonium thiocyanate solvation	[mol/l]

The factor at the beginning of the equation refers to the molecular mass of chloride atoms which is 35.45 g/mol. The difference is due to units:

$$35.45 \left[\frac{g}{mol}\right] \cdot 0.001 \left[\frac{mol}{ml}\right] \cdot 100\ [\%] = 3.545 \left[\frac{g\%}{ml}\right] \tag{51}$$

When using an automatic titrator, equation (50) should be adjusted because the back titration is not necessary anymore. The automatic equipment is able to dose the volume of silver nitrite needed to bind with all the chlorides.

$$m_{Cl^-} = C_m = 3.545\frac{V_1N_1}{m} \tag{52}$$

The theory behind the titration is the following: an excess of silver nitrite ions is added to the solvation of chloride ions. The reaction results in silver chloride.

$$Ag^+ + Cl^- \rightarrow AgCl \tag{53}$$

The excess of silver reacts with the added thiocyanate solvation.

$$Ag^+ + SCN^- \rightarrow AgSCN \tag{54}$$

The excess of thiocyanate ions is indicated by the formation of a red iron(III)-complex.

$$Fe^{3+} + SCN^- \rightarrow FeSCN^{2+} \tag{55}$$

The solubility of AgCl is higher than AgSCN. This means that the solid silver can participate again in the solvation with the added thiocyanate. To prevent this reaction, benzyl alcohol or nonanol should be added.

The initial chloride content C_i can be specified via an identic specimen of the same concrete composition. For the drilled core, one takes a specimen of 20 g that is as far as possible of the standardized sieve with a mesh opening of 1 mm. The chloride content of the resulting concrete particles can be specified via the method of NT Build 208 as described earlier.

The parameters C_s and D_e are defined by the equation (56) with a non-linear analysis. The first measurement is omitted as shown in figure 4.2.

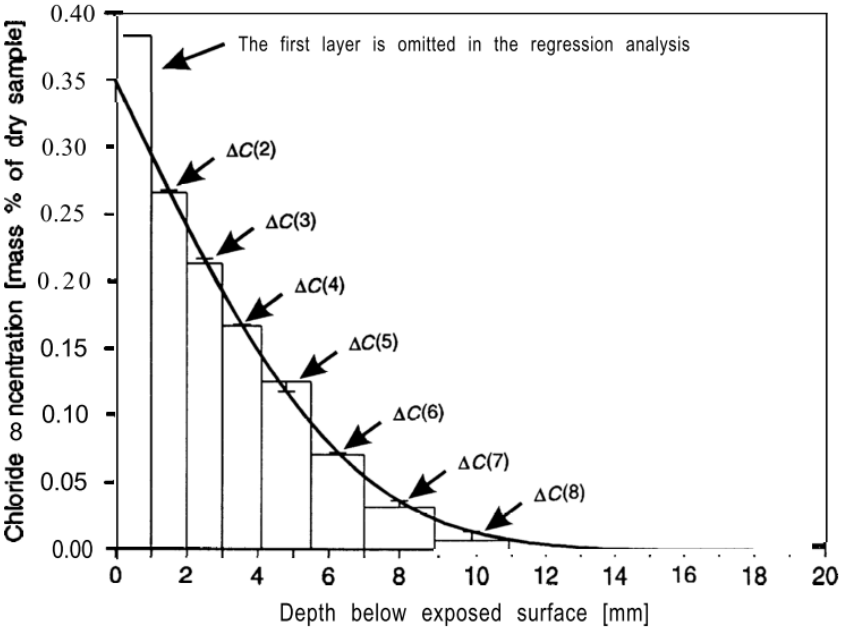


Figure 4.2: Non-linear regression analysis (Application, 1995)

$$C(x,t) = C_s - (C_s - C_i) \cdot erf\left(\frac{x}{\sqrt{4D_e t}}\right) \tag{56}$$

Where

C(x,t)	Chloride content at average depth x and exposure time t	[% of m_c]
C_s	Boundary conditions for the exposed surface in relation to the dry concrete mass	[% of m_c]
C_i	Initial chloride content	[% of m_c]
erf	Error function (verwijzing naar formule)	[-]
x	Depth under the exposed surface	[m]
D_e	Effective chloride transport coefficient	[m²/s]
t	Duration of exposure	[s]

The non-linear regression is conducted with the least-squares method with S as follows:

$$S = \sum_{n=2}^{N} \Delta C^2(n) = \sum_{n=2}^{N} \left(C_m(n) - C_{(x,t)}(n)\right)^2 \tag{57}$$

The penetration parameter K_{Cr} can be determined with the use of equation (58) for $C_s > C_r > C_i$. C_r is stated as the 0.05 mass percentage.

$$K_{Cr} = 2\sqrt{D_e} \cdot erf^{-1}\left(\frac{C_s - C_r}{C_s - C_i}\right) \tag{58}$$

Where

K_{Cr}	Penetration parameter	[m/√(s)]

C_r	Reference of chloride content in relation to the dry concrete mass	[% of m_c]
erf^{-1}	Inverse error function	[-]

(Application, 1995)

4.3 Migration test

4.3.1 Specifications

The instructions for the migration test are standardized by NordTest: NT Build 492 (NT Build 492, 1999). It is a quick way to determine the resistance against chloride ingress using the chloride migration coefficient D_{nssm} in a non-steady-state migration. This kind of migration involves a change in the chloride content in function of time.

The experiment is valid for samples manufactured and saved in the laboratory as well as for drilled cores in situ. The cylinder-shaped specimens have a minimal diameter of 100 mm and a thickness of 50 mm. At least three samples should be tested to ensure a certain accuracy. (NT Build 492, 1999)

4.3.2 Method

Due to an electric potential, the chloride ions are forced to enter the cylinder via the axial axes. The catholyte solution is 10 % NaCl by mass in tap water (100 g NaCl in 900 g water) and the anolyte solution is 0.3 mol/l NaOH in de-ionised water (approximately 12 g NaOH in 1 l water). The test set-up is shown in figure 4.3 with set-up parameters in figure 4.4. The specimens are placed in a rubber sleeve and secured with two stainless clamps. The whole is placed on a plastic support in a reservoir with 12 l catholyte solution. The free space of the rubber sleeve is filled with 300 ml anolyte solution in which the anode takes place. The anode and cathode are connected respectively to the positive and the negative pole of the DC potential. (NT Build 492, 1999)

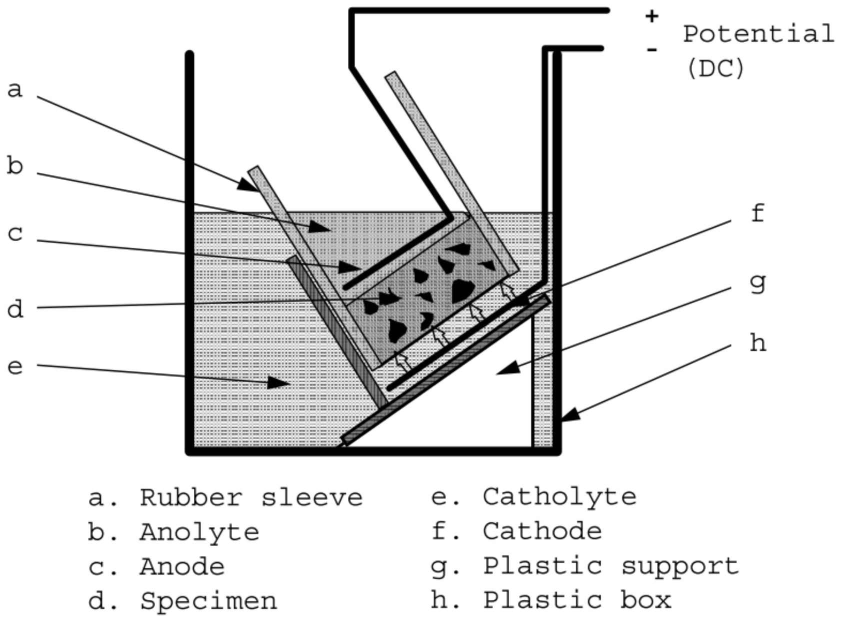


Figure 4.3: Set-up for the migration test (NT Build 492, 1999)

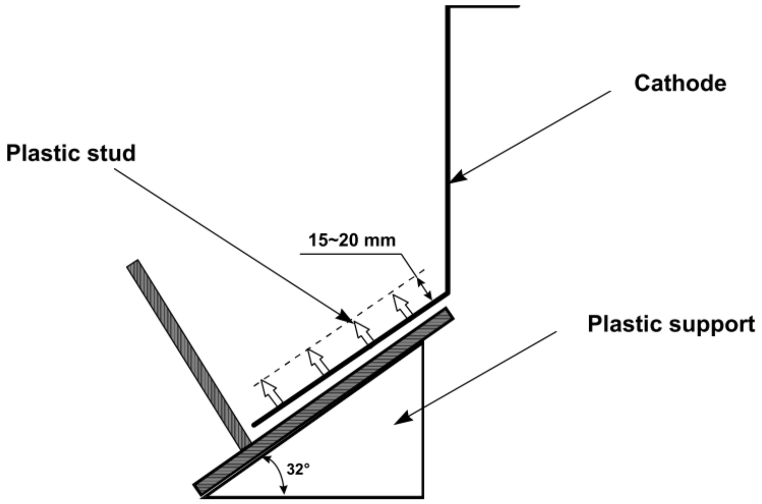
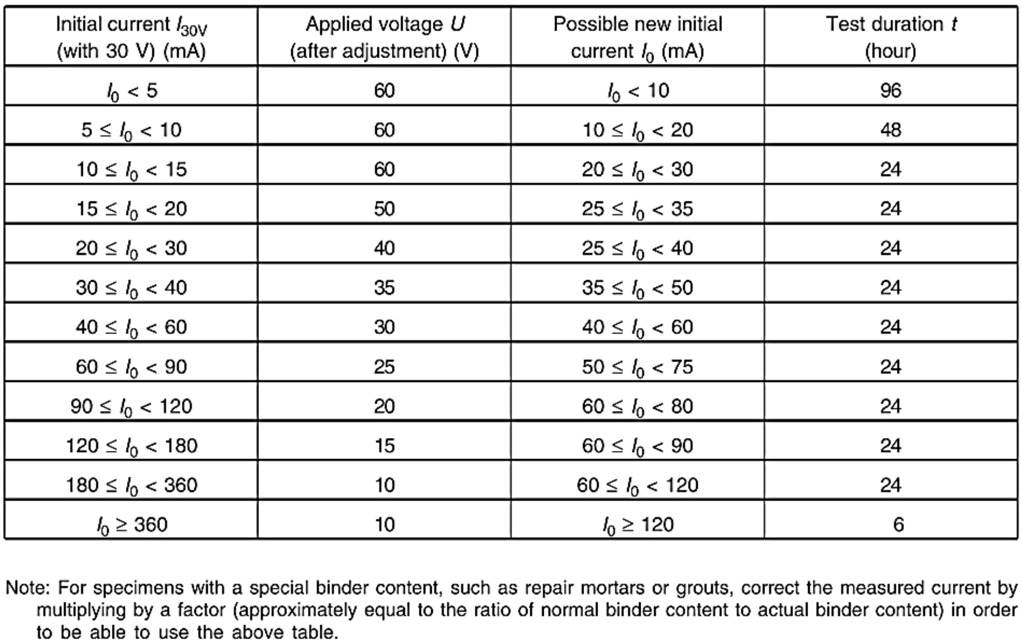


Figure 4.4: Specifications for the set-up of the migration test (NT Build 492, 1999)

The test starts with a tension of 30 V throughout the cylinder. Depending on the measured current intensity, the voltage should be adapted according to the table in figure 4.5. The new measured current will define the duration of the experiment. The temperature at the beginning and at the end of the anolyte solvation will be measured and should be maintained between 20 and 25°C.



Initial current $I_{30\text{V}}$ (with 30 V) (mA)	Applied voltage U (after adjustment) (V)	Possible new initial current I_0 (mA)	Test duration t (hour)
$I_0 < 5$	60	$I_0 < 10$	96
$5 \leq I_0 < 10$	60	$10 \leq I_0 < 20$	48
$10 \leq I_0 < 15$	60	$20 \leq I_0 < 30$	24
$15 \leq I_0 < 20$	50	$25 \leq I_0 < 35$	24
$20 \leq I_0 < 30$	40	$25 \leq I_0 < 40$	24
$30 \leq I_0 < 40$	35	$35 \leq I_0 < 50$	24
$40 \leq I_0 < 60$	30	$40 \leq I_0 < 60$	24
$60 \leq I_0 < 90$	25	$50 \leq I_0 < 75$	24
$90 \leq I_0 < 120$	20	$60 \leq I_0 < 80$	24
$120 \leq I_0 < 180$	15	$60 \leq I_0 < 90$	24
$180 \leq I_0 < 360$	10	$60 \leq I_0 < 120$	24
$I_0 \geq 360$	10	$I_0 \geq 120$	6

Note: For specimens with a special binder content, such as repair mortars or grouts, correct the measured current by multiplying by a factor (approximately equal to the ratio of normal binder content to actual binder content) in order to be able to use the above table.

Figure 4.5: Test voltage and duration for concrete specimen with normal binder content (NT Build 492, 1999)

At the end of the imposed test duration, one can remove the specimens out the migration cell. The sample should be cleaned with water and subsequently split in two halfs. Both parts should be sprayed with $AgNO_3$ solvation 0.1 mol/l. About fifteen minutes later, the white silver chloride precipitation on the split surface is clearly visible. The penetration depth can be measured from the centre to both edges at intervals of 10 mm as shown in figure 4.6. A few cautions should be taken during the measurements:

- If the penetration front is obviously blocked by the aggregate, move the measurement to the closest front where there is no blocking. Alternatively, the depth can be ignored in case of sufficient valid depths.
- If there is a penetration with a significant deviation in relation to the other fronts, the concerning front may be excluded.
- No measurements shall be taken in less than 10 mm of the axial edges of the cylinder. The purpose is not to involve possible seepage due to non-homogeneous saturation.

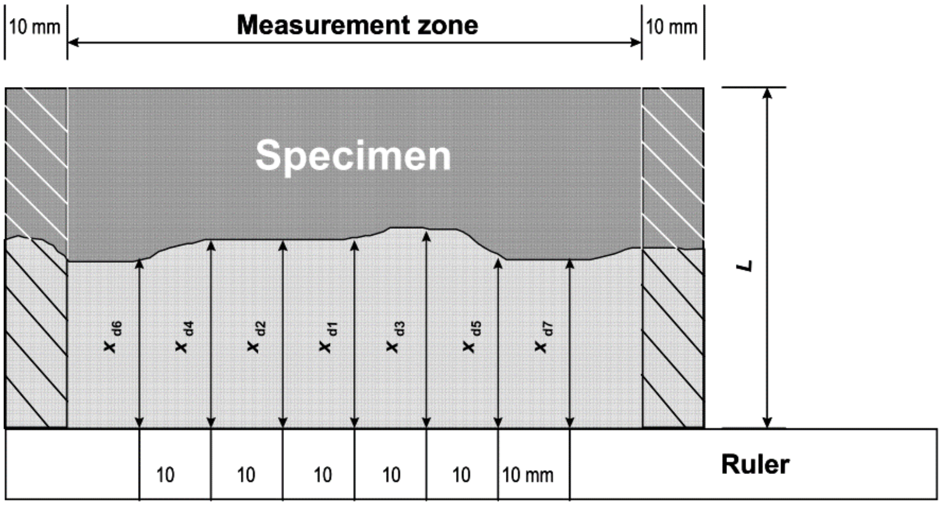


Figure 4.6: Illustration of measurement for chloride penetration depths (NT Build 492, 1999)

4.3.3 The chloride migration coefficient

The chloride migration coefficient D_{nssm} in a non-steady-state is calculated with the following equation (59): (NT Build 492, 1999)

$$D_{nssm} = \frac{RT}{zFE} \cdot \frac{x_d - \propto \sqrt{x_d}}{t} \tag{59}$$

With

$$E = \frac{U-2}{L} \tag{60}$$

$$\propto = 2\sqrt{\frac{RTL}{z\,FE}} \cdot erf^{-1}\left(1 - \frac{C_d}{C_0}\right) \tag{61}$$

Where

D_{NSSM}	Chloride migration coefficient for a non-steady-state	[m²/s]
z	Absolute value of ion valence, for chloride: z = 1	[-]
F	Faraday constant, F = 9.648 × 10^4	[J/(V·mol)]
U	Absolute value of the applied voltage	[V]
R	Gas constant, R = 8.314	[J/(K·mol)]
T	Temperature	[K]
L	Thickness of the specimen	[m]
x_d	Average value of the penetration depths	[m]
t	Test duration	[s]
erf^{-1}	Inverse error function	[-]

C_d	Bound chloride concentration at which the colour changes, $c_d \approx$ 0.07 mol/l for OPC concrete	[mol/l]
C_0	Chloride concentration in the catholyte solution, $C_0 \approx 2$ mol/l	[mol/l]

5 Inspection

5.1 Introduction

A clever design of a concrete structure is indispensable for a long lifetime. However, not every exterior effect can be foreseen. To guarantee the durability of reinforced concrete one needs to review the current state on a regular basis. There are distinct types of inspection starting already at the mixing process of the concrete to a quick routine check or a profound investigation of the finished structure.

5.2 Concrete production

The inspection starts at the mixing process of the concrete, usually in concrete plants. These are factories where raw materials and consumables are collected to mix diverse types of concrete. The transport, quantities and mixing processes are kept under close supervision as they have a major influence on the quality of the product.

5.2.1 Aggregates

Sand and aggregates are usually transported by ship or by axes, as most of the concrete plants are located along waterways. During loading and unloading, one should be aware of maintaining a homogenic grading curve and avoiding contamination. The grapple should store the coarse material layer ordered to prevent segregation. Moreover, any contact with water is forbidden as it could influence the total amount of water in the final product.

For the dosing of the aggregates, it is recommended to use weight dosing. Only for light aggregates, volume dosing may be a sound alternative.

5.2.2 Cement

The most important property of hydraulic binders is the capability to react with water. Consequently, any contact with water or moisture during transport or storage must be avoided. Furthermore, distinct types of cement should be stored in different silos.

Dosing of cement takes place in separated weight dosing containers. The required accuracy of the equipment is about 1.5 % in relation to the target value.

5.2.3 Water

Water is brought to a water weighing system via a pipeline network. Using other water than tap water, the water will be stored in bunkers in order to control the quality before adding it to the mixture.

Water is usually dosed by a measurement system of calibrated flow meter. The water content may also be measured automatically via the electrical resistance of the concrete mixture or via the power of the mixer in function of the consistency of the liquid concrete.

5.2.4 Consumables

The supply of consumables, such as air entertaining agents, plasticisers, accelerators or retarders, is usually provided via roads in containers of 1000 l. A clear manner of storage should be applied in a separated space. Due to frost, many consumables can lose their functioning in whole or in part. For this reason, the products should be kept in frost-free rooms. To prevent sagging of heavy particles, the liquids should be stirred from time to time, mechanically or by air inflation.

The market provides several methods to dose consumables. Measuring cylinders are an option for liquid consumables, which allows to control the dosed quantity visually. Powdered products, such as dyes, are often added manually. Consumables in small amounts are added in the mixing water.

5.2.5 The mixing process

The purpose of the mixing process is to divide the dosed ingredients as equal as possible to obtain a homogenic mixture. Beside using the correct type and dose of the ingredients, the water-cement ratio should be kept as low as possible assisted by consumables. This in order to optimize the strength and to reduce permeability and consequently the ability for chlorides to enter the structure. During the process of pouring, a sound dispersion of the aggregates around the reinforcement bars should be ensured. Moreover, a good compaction in assistance of a vibrator is important which can reduce the permeability up to 9 % (Ritzon, 2004). All these regulations are essential to pursuit high durability and resistance against corrosion.

5.3 Routine inspection

Routine inspection must ensure that the natural ageing process is succeeded closely. When any form of damage is noticed in the beginning faze, worse deterioration may be prevented by taking appropriate measures. Routine inspection is the first step of the evaluation of concrete structures. The diverse types consist of a visual inspection, basic tests and a report of these actions.

5.3.1 Visual inspection

The visual inspection is meant to detect all signs of concrete deterioration and identify the possible sources of the problem. During the inspection, the focus is on the following aspects:

- The inspection of the information and documentation about the history of the structure;
- Old forms of protection, coatings and impregnations;
- The original appearance of the concrete surface;
- The differences in colour at the concrete surface;
- The presence, appearance and localization of cracks;
- The superficial deterioration of the concrete surface;
- Damage to the concrete itself;
- Exposed reinforcement bars;
- Deformation of the structure;
- The presence of moisture or water, leakage, etc.

Subsequently, this data should be illustrated in graphs with marks for the delamination and cracks in the concrete material. The details about the width of the cracks are necessary when improvements with injections are proposed and measures of the width of the crack should be checked on a regular basis with the aim of succeeding the evolution of the deterioration.

5.3.2 Condition survey

The visual inspection is restricted to the visible aspects of deterioration. Although, concrete deterioration may also remain out of sight for a long time within the concrete structure. Thus, serious defects may occasionally progress over a large area before visible signs, such as cracking, spalling or staining, emerge. However, external factors, e.g. fire overload or earthquake, may produce immediate visible damage while they may also cause internal damage which may remain latent for a certain time. The conclusion is that visible aspects are no reference for the current state of the whole concrete mass and the visual inspection of the structure but should be assisted by basic tests to obtain numerical results. (Tay & Tam, 1995)

The basic tests can be executed in situ with small and portable equipment. However, the test results may vary due to the weather conditions and the working environment. One must be aware that the degree of deterioration is affected by many factors including the original variability of material properties. Consequently, the locations for the samples should be chosen randomly to make them statistically representative. Also, a large number of tests is recommended to enable a higher reliability on the statistical evaluation. As some tests are destructive, expensive or time-consuming, the number of samples is limited and should be planned and used with sound engineering judgement.

5.3.2.1 The rebound hammer

The rebound hammer the sclerometer, see figure 5.1, is made of steel and contains a spring-loaded plunger. The moment the compression spring is released, the plunger plus the hammer mass accelerates in a hollow barrel with a fixed and reproducible velocity at the end of the hammer guide. One measures the distances of the setback of the hammer on a linear scale, attached to the housing of the instrument. This is the method for an analogue device. In case of a digital test hammer, the appliance is equipped with an electronic transducer which converts the rebound values into a reading on the digital display. Moreover, a range of statistics is drawn up and could be downloaded by a computer.

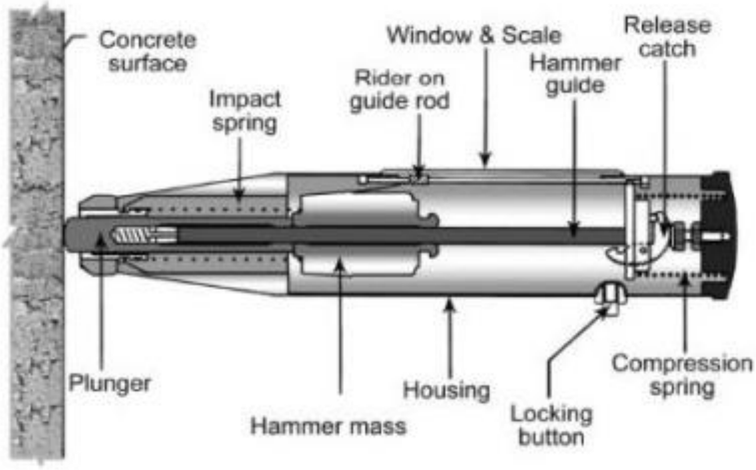


Figure 5.1: Illustration of the rebound hammer

The distance of the setback is called the sclerometer-index and gives an indication of the concrete quality. Empirical relations are used between the sclerometer-index and the compression strength of the concrete to interpret those values. An example is graphically illustrated in figure 5.2.

The test anvil has a diameter of approximately 150 mm with an impact area hardened to Binell 500 or Rockwell 52C. Test areas exhibiting carbonation, high porosity or honeycombing are not recommended. The test surface shall be ground smooth with an abrasive stone when heavily textured, soft or covered with loose mortar.

The test procedure has been standardized by ASTM C 805 Standard Test Method for Rebound Number of Hardened Concrete and imposes the following method. Hold the instrument firmly and perpendicular to the test surface. Push the button gradually until the hammer impact has taken place. Estimate the rebound number before recording the result to verify the test. Take ten readings from each test area. No two impact tests should be closer together than 25 mm. Evaluate the impression made on the surface after impact, and if the impact crushes or breaks through a near-surface air void, disregard the reading and conduct another test.

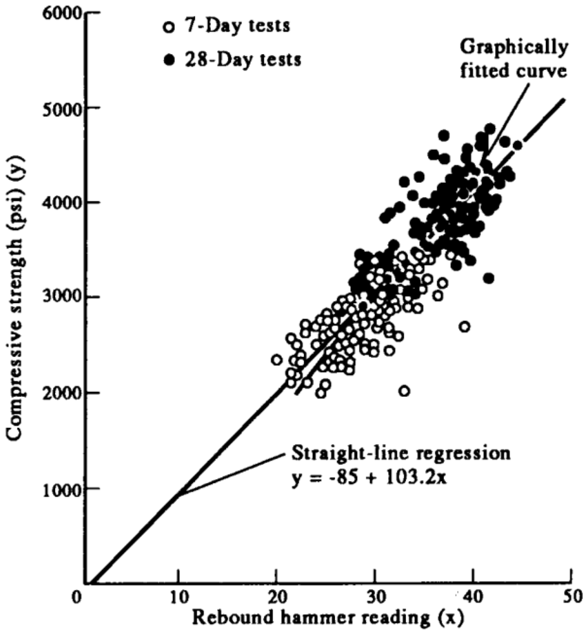


Figure 5.2: Relationship of compressive strength of concrete to rebound hammer test reading (Series 149) (Tay & Tam, 1995)

5.3.2.2 The penetration test

The probe of the penetration test consists of a hardened steel rod and is driven into the concrete mass. The exposed length of the probe represents the relative resistance of the concrete to the impact of the projectile. This penetration test is not entirely non-destructive since it leaves surface perforations and often a small conical area of broken concrete when the probe is withdrawn. The instrument is capable to reproduce the same velocity and functions similar to the rebound hammer, see the previous chapter **Fout! Verwijzingsbron niet gevonden.**. However, it measures the resistance of the concrete below the surface, while the rebound hammer tests the hardness of the surface. One must be aware that this experiment strongly varies on the heterogenic properties of the concrete.

5.3.2.3 Ultrasonic Pulse Velocity Test

The Ultrasonic Pulse Velocity Test or UPV test is designed to identify and map voids, cracks and other forms of deterioration in the concrete. The methodology relies on direct arrival of compressional waves with frequencies ranging from 50 to 150 kHz. The device consists of a vibrating transducer located on one side of the concrete member and a receiving transducer on the other side. The ultrasonic energy transfers through the concrete mass and its velocity is affected by the concrete density, compressive strength, presence of reinforcing steel, cracks and voids. Nevertheless, the method is recommended to be used only for comparative and qualitative condition studies than for quantitatively estimating concrete strength. An technique scheme of the experiment is added in figure 5.3.

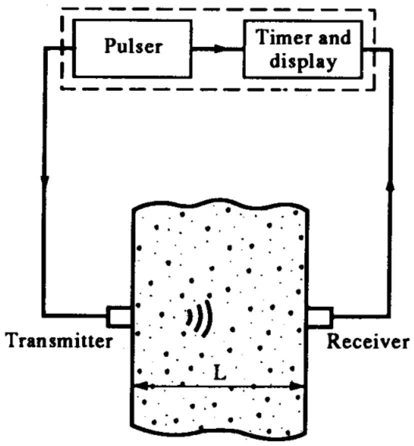


Figure 5.3: A technique scheme for the UPV experiment

5.3.2.4 The pachometer

The coating, the localization and orientation of the reinforcement bars are estimated by a pachometer, as shown in figure 5.4. This is a non-destructive magnetic device and operates by generating a magnetic field and measuring the reaction between the magnetic field and the metal. By the intensity of the response, one can determine the location and thickness of the concrete cover. For the depth measurements, an accuracy of 0.254 cm (0.1’) is feasible. The test equipment is an economical technique compared to radiography and radar and only requires access to one side of the concrete slab or wall.

There are distinct kinds of testing devices to measure the diameter of standard reinforcement bars. The capability to distinguish individual rebars depends on the size, the coating and the distance in between. The smaller the distance between two rebars, the more difficult it is to identify the rebars separately and the lower the accuracy of the measurement of the coating. To estimate the accuracy, one can execute exploratory drillings and compare the results to the signals of the pachometer.

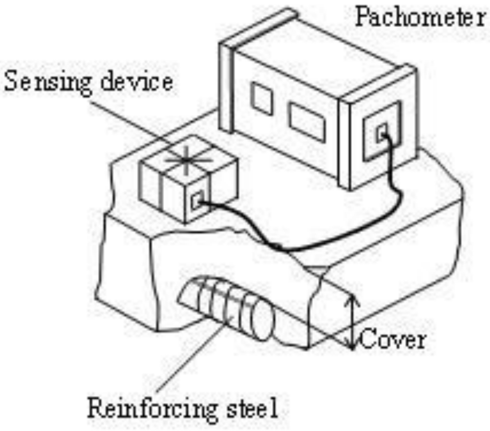


Figure 5.4: A technique scheme for the pachometer experiment

5.3.2.5 Drill method for the carbonation depth

Variables, such as type of exposure or cement, after-treatment and the degree of compaction of the concrete, have a profound influence on the carbonation depth. All the testing methods for this parameter are destructive. The carbonation depth is measured by dividing a drill core in halves and spraying a pH-indicator on the fresh fracture surface, e.g. phenolphthalein. Subsequently, the colourless zone indicates the carbonated parts and the remaining pink zone stands for the non-carbonated zone.

Another drill method is more economical and is executed as follows. The drilled powder is collected on a sheet of paper that is imbued by phenolphthalein. The moment the powder turns pink, the drilling process is stopped and the depth of the drilled hole is considered the carbonation depth. This method is less accurate and requires a larger number of tests. Moreover, the presence of coarse aggregates increases the error probability enormously.

With the obtained results of the in situ tests, in relation to the coating, the risk of corrosion due to carbonation of the concrete can be estimated.

5.3.2.6 Conclusion

Various test approaches have been established and it is recommended to use several of them to improve the confidence in the estimation of the current state. Non-destructive tests are preferred before partly destructive tests to evade additional damage. The evaluation of the visual inspection and of the results of the basic tests should lead to one of the following conclusions:

- There are no further measures needed;
- A more profound investigation of the concerning deterioration parts is required, before further steps are taken;
- An immediate action, such as prohibition on use or access, limited charge, local enforcement, etc. are demanding, regarding to a compromise of the safety of the structure and the users.

5.3.3 Profound inspection

The aim of a profound inspection is twofold. On one side, it should supplement the routine inspection when demanding or in case of the establishment of concerning aspects. On the other hand, more in-depth tests are carried out in order to judge the current state of the structure, the life span and the structural resistance.

5.3.4 Non-destructive tests on chloride ingress

Recently, external techniques and embedded sensors have been developed for measuring chloride ingress in concrete structures. They consist of the following methods:

- Ion selective electrode (ISE);
- Electrical resistivity (ER);
- Optical fiber sensors (OFS).

5.3.4.1 Ion selective electrode

The Ion Selective Electrode (ISE) or potentiometric sensor has been studied in electrochemistry for several years thus, the knowledge about the method is extensive. Moreover, it is capable to distinguish chloride ions from other ions without being destructive or chemical tests. The method involves embedded sensors to enable a closed circuit with reinforcement bars. In this way, the change in potential difference can be measured without affecting the environment. The system is based on the Nernst equation:

$$E = E^0 + \frac{RT}{nF}\ln(a_{Cl^-}) - E_{ref} \tag{62}$$

Where

E	Measured voltage	[V]
E^0	Standard electrode potential (generally Ag/AgCl)	[-]
n	Number of electrons	[-]
E_{ref}	Standard potential of the reference electrode	[V]

In order to measure the voltage, it is necessary to complement the ISE with a reference electrode (RE). A technique scheme is added in figure 5.5.

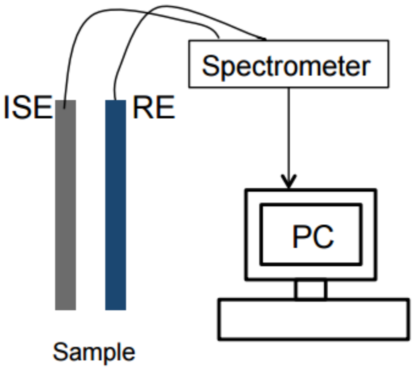


Figure 5.5: A technique scheme for the ISE experiment

6 Protection systems

6.1 Introduction

Protection systems are numerously available on the market and have only one purpose: extension of the life span of the structure and its material. The various kinds of damage and deterioration as well as the main causes have been investigated for a long time. The result has been converted in part nine of the European standard EN 1405 where a distinction has been made between eleven principles. Principles 1 to 6 treat the defects in the concrete material and the remaining principles are related to the damage as a consequence of corroded reinforcement. A list of the eleven principles is added below:

- Principle 1: Protection against ingress (PI);
- Principle 2: Moisture control (MC);
- Principle 3: Concrete restoration (CR);
- Principle 4: Structural strengthening (SS);
- Principle 5: Increase in physical resistance (PR);
- Principle 6: Resistance to chemicals (RC);

- Principle 7: Preserving or restoring passivity (RP);
- Principle 8: Increasing resistivity (IR);
- Principle 9: Cathodic control (CC);
- Principle 10: Cathodic protection (CP);
- Principle 11: Control of the anodic areas (CA).

(“EN 1504,” 2005)

For this study and in function of the application on the harbour ‘Marina Internacional’, the focus is put on principle 3 about concrete restoration (CR).

6.2 Repair mortars

6.2.1 Introduction

Repair mortars are based on a binder, mixed with graded aggregates and often include admixtures and additions. They are used for different functions, such as substitution of deteriorated concrete, filling cracks, protection of reinforcement and equalizing of surfaces. Principle 3 of the European standard EN 1504-3 describes the requirements for the repair mortars for structural and non-structural repairs. The repair mortars are classified by the type of binder.

6.2.2 Types of repair mortars

6.2.2.1 Repair mortars based on hydraulic binders

Repair mortars containing hydraulic binders protect the reinforcement by creating an alkaline environment around the reinforcement. To improve the properties of these mortars, one can add polymers or additives and fibres. The addition of polymers offers the following advantages:

- Compensation of shrinkage;
- Increased tensile strength;
- Improved adhesive properties;
- Reduced permeability for moisture and carbon dioxide;
- Enhanced frost resistance;
- Augmented chemical resistance.

Beside this, polymers increase thermal expansion, which results in more tension.

Repair mortars on basis of hydraulic binders, modified by polymers with a ratio of 5 — 20 % of polymers in relation to the cement mass, are indicated as PCC or Polymer Cement Concrete mortar. The remaining mortars with less than 5 % of polymers in relation to the cement mass are marked with CC or Cement Concrete mortar.

6.2.2.2 Resin bound repair mortar

Resin bound repair mortars consist of resin, which is the mayor ingredient, a filler and one or more components in order to accelerate the curation or the polymerisation and it has the function of a catalyst.

In comparison to repair mortars with hydraulic binders, the resin bound repair mortars have higher mechanical resistance, for tensile and compressive strength, which develops in an earlier stadium. Moreover, they obtain a higher thermal expansion coefficient, a lower modulus of elasticity, a higher curation shrinkage and a reduced fire resistance. These mortars are common used for small, local repairs, eventually in case of exposure to chemical substances.

Resin bound repair mortars are indicated as PC or Polymer Concrete mortar. The most common used repair mortars are epoxy resin with amine as hardener. Less common types of hardeners are polyurethane. The addition of fillers and pigments is allowed.

6.2.2.3 Selection of the product

The selection of the type of repair mortar is not only based on the desired properties of the product. It should be similar to the properties of the original concrete, except for low quality concrete. Another important factor to take into consideration is shrinkage. It should be reduced to a minimum in order to prevent internal tension. Moreover, each product has a recommended way of application. Hence, in function of the scope of the repair works, whether or not labour-intensive methods are preferred.

EN 1504-3 made a distinction between repair mortars for structural and non-structural purposes. Moreover, each category is divided in two strength classes. Class R1 and R2 belong to non-structural use and class R3 and R4 belong to structural use. Table 6.1 gives a resume of the criteria for repair mortars based on hydraulic binders imposed by European standard EN 1504-3. One can notice that the requirements for non-structural repairs are less strict compared to the structural repairs.

Type of repair mortar	Class	Compression strength [N/mm²]	Adhesion strength [N/mm²]
Non-structural	R1	≥ 10	≥ 0.8
	R2	≥ 15	≥ 0.8
Structural	R3	≥ 25	≥ 1.5
	R4	≥ 45	≥ 2.0

Table 6.1: Criteria for repair mortars according to EN 1504-3

6.2.3 Preparation of the subsurface

Before applying any protection, the deteriorated concrete and the corroded steel should be removed carefully. The method about preparation of the subsurface is described by the European standard in EN 1504-10.

6.2.3.1 Removal of the concrete

First deteriorated areas should be investigated and marked by an expert. When confirmed, the edges of these areas should be marked with a saw cut of 5 mm deep. In case of a maximal diameter D_{max} of the aggregates equal to 2 mm or more, the saw cut should be minimum three times D_{max}. Parts with inadequate adhesion or with pore concrete quality, should be removed until sound quality concrete is met with a minimum depth of 5 mm. The recommended equipment for the removal is a jackhammer on air pressure. The use of flame gunning is forbidden. The saw cuts should have angle of minimum 90° and maximum 135° to avoid delamination of neighbouring, good quality concrete. A course surface should be formed to ensure a strong, mechanical bond between the original material and the recovery mortar.

The depth of the concrete that should be removed, depends on the concrete cover:

- When the restored concrete cover will exceed 20 mm and the circumference of the reinforcement bar is substantial in the healthy zone of the concrete, it is not necessary to clear the whole bar. The half of circumference of the bar should be removed as shown in the first image of figure 6.1.

- In case the restored concrete cover will exceed 20 mm and the circumference of the reinforcement bar is mainly situated in the deteriorated concrete part, the whole bar should be cleared up to 10 mm behind the bar over a length of minimum 20 mm parallel to the rebar. This is illustrated in the second image of figure 6.1.
- For a restored concrete cover that will not exceed 20 mm, the whole rebar should be cleared, independent of the percentage of the circumference that has been deteriorated by corrosion. This is explained by the image in figure 6.1.
- When the whole concrete surface is covered with a layer of repair mortar with a thickness that exceeds 20 mm or with additional protection coatings, the concrete should be removed in the zones with delamination according to the described method in the first and second case. For a thickness of the repair protection layer between 5 and 20 mm, the same method is applied as when whole protection layers are not present.

Locations where the reinforcement crosses the edge between deteriorated and not-deteriorated concrete, the bar should be cleared up to the not-deteriorated zone with a length equal to the diameter of the rebar and minimum 20 mm.

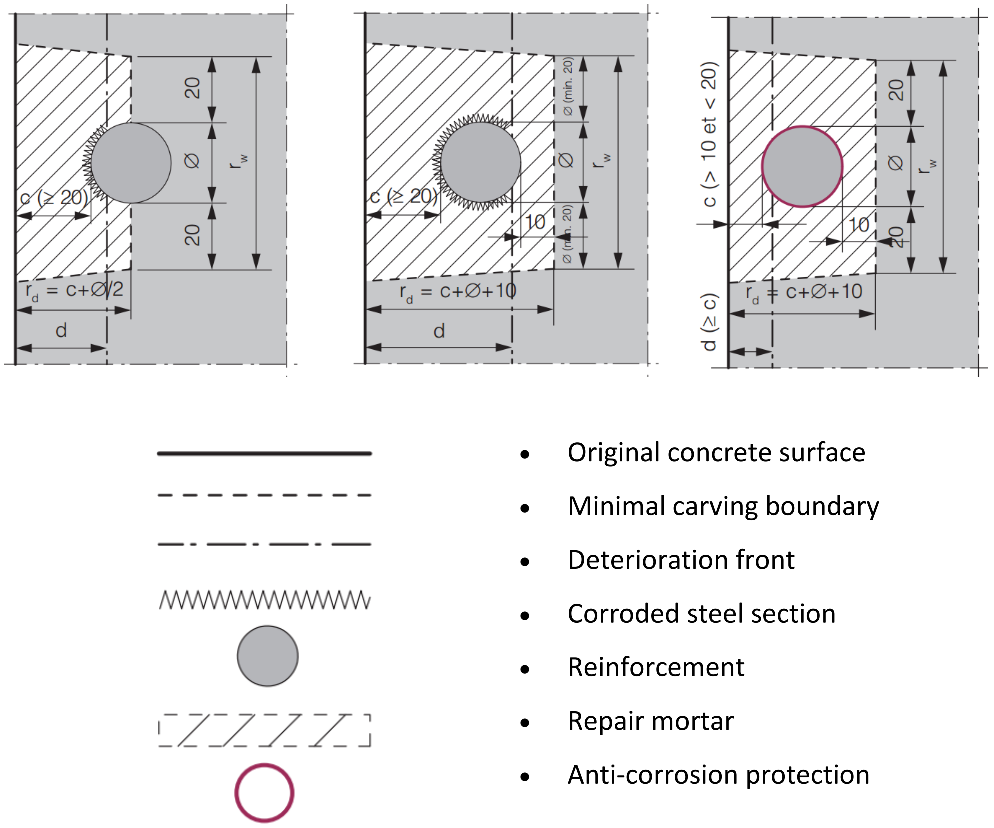


Figure 6.1: Illustration of the original section and the section to be removed with related parameters

Where

d	Depth of the deteriorated zone, measured from the original concrete surface	[mm]

c	Original concrete cover	[mm]
Ø	Nominal reinforcement diameter	[mm]
r_d	Minimum depth to be removed	[mm]
r_w	Minimum diameter of zone to be removed	[mm]

To ensure a sufficient length of the cleared rebar, a whole corroded bar should be cleared up to the maximum of the following values: 15 mm or the maximal diameter D_{max} of the aggregates of the repair material, plus five.

6.2.3.2 Cleaning of the reinforcement

The corroded steel on the rebars should be removed, as they contaminate the repair mortar and reduce the adhesion between steel and concrete. There are several cleaning systems:

- Grit blasting: fine, hard particles, such as coal slag blasting grid, are sprayed with high velocity due to compressed air. The shape, size and composition of the particles define the intensity of the process. This method causes dust formation and waste.
- Water jetting: water is sprayed under high pressure, with 500 to 2000 bar. This technique is recommended for chloride contaminated reinforcement.
- Milling and brushing: mechanical surface processing which is recommend for coarse cleaning, prior to the application of a steel primer. It is unable to reach the backside of the rebar.

With advanced methods, such as grit blasting or sand blasting, a near white surface can be achieved. The two cleaning procedures produce different steel surfaces, which have an influence on the corrosion resistance of the repaired structure. It was found that rebars cleaned by sand-blasting reach a higher resistance of corrosion compared to mechanical cleaning. This could be caused by the more thorough cleaning. (Al-Dulaijan et al., 2002)

After the removal of the deteriorated concrete and the corroded reinforcement, the rebars should be subject of a sound investigation to examine whether the reinforcement is still able to fulfil the original function. In case of visible reduction of the section, such as scaling or pitting corrosion, a structural investigation should be conducted by an authorized engineer. Depending on the results, one can conclude to change the rebars or to add reinforcement.

6.2.3.3 Preparation for the repair mortar

After removal, the concrete surface should be cleaned to eliminate organic liquids, such as oil, grease, cement grout, etc. to enable sufficient course properties and a good bonding of the repair mortar. A recommended method for this cleaning process is the use of sandblasting with oil-free compressed air.

After this preparation, the subsurface should have a surface cohesion which is equal to or improved compared to the adhesion value required for the repair mortar. When this condition cannot be fulfilled, the surface cohesion should be equal to the tensile strength of the concrete.

At least two hours before the application of the repair mortar, the subsurface should moistened. When applicating, the subsurface should still be moistened, without showing any glance sight.

6.2.3.4 Insertion layer

In the next step, an insertion layer can be applied to obtain a strong bonding between the repair mortar and the substrate. The use of the product is strongly recommended when the application area takes place on a vertical surface or on a downside of a horizontal surface. The layer is often based on the same binder as the used repair mortar. For repair mortars based on hydraulic binders, it is not always recommended to use an insertion layer. The layer should be applied by experts for the following reason: if the layer is not applied properly, then it may have a detrimental effect on the insertion.

6.2.4 Application of the repair mortar

In this master thesis and in function of the application on the harbour 'Marina Internacional', the following techniques of application shall be covered:

- Manual application;
- Pouring application;
- Dry spraying application.

6.2.4.1 Environmental parameters

The temperature of the environment and the concrete structure during application is bounded. The minimum and maximum temperature is added to the technical information sheet of the product. The recommended values are respectively 5 °C and 25 °C, unless specified otherwise by the producer.

6.2.4.2 Technical parameters

The maximum layer thickness of the repair mortar based on hydraulic binders is given in the technical information sheet of the product. In general, it depends on the maximal diameter of the aggregates D_{max} and is often equal to a factor ten of this parameter. Locally, layers with higher thickness may be allowed. Moreover, the use of a mesh reinforcement may increase the allowed thickness of the layer.

In addition, the minimum layer thickness is added to the technical information sheet of the repair mortar and usually rates a factor three of D_{max}. The nominal layer thickness should also be found in the same information sheet.

6.2.4.3 *Manual application*

This technique is common for local and small repairs, as large areas would be very labour-intensive. Common used products are PCC mortars, because the polymers create a sound compaction and reduce the shrinkage. Beside of that, CC mortars have a better processability due to substances, such as micro silica. One should consider that these mortars have an inferior resistance against chlorides. Another applicable type of product for this technique is PC mortar. This repair mortar shows many differences with concrete in terms of mechanical properties and should not be used for large repairs.

The repair mortar and its consistency are selected depending on the position of the application area: a vertical surface, the upside or the downside of a horizontal surface. Still, the quantities should fit the imposed boundaries.

The repair mortar is applied with a trowel and later pressed to compact the vicious material, as seen in figure 6.2. Eventually, a formwork may be used for the finishing process. The resulting surface should be finished properly to achieve a smooth surface, which will prevent the water infiltration and fulfil aesthetical requirements. The focus should be on the edge of the surface to be repaired. An excessive treatment will not improve properties as it may even result in detachment of the layer.

Figure 6.2: Manual application of the repair mortar with a trowel

6.2.4.4 *Pouring application*

This technique is beneficial for large areas or volumes of repair and for structures with a dense mesh reinforcement. The properties of the repair mortar should be tailored to the properties of the concrete substrate in order to reduce shrinkage. PCC mortars are preferred, due to beneficial properties in terms of shrinkage and tensile strength. In case of larges volumes, segregation or early curation may appear.

The repair mortar should have a good processability when the product is applied by pouring or pumping. Moreover, a fitting and mortar tight formwork is needed which should be anchored in a sound way to withstand the hydraulic pressure. To prevent segregation, pouring repair mortars should not be vibrated. One must pay attention to the venting process of the poured material and to the prevention of enclosed air bubbles. Interruption in the pouring process can cause those enclosed air bubbles and an incomplete filling of the formwork. Figure 6.3 illustrates the pouring process.

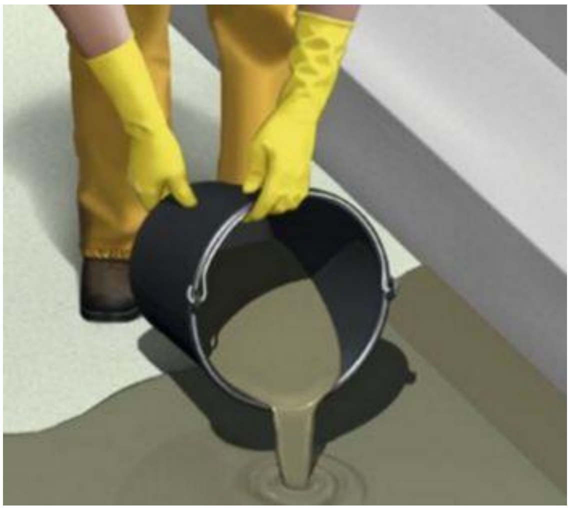


Figure 6.3: Pouring application of the repair mortar with formwork

6.2.4.5 Spraying application

For the spraying application, the repair mortar is sprayed on the substrate with oil-free compressed air. This technique is a rapid and economical way of application for very large volumes with limited use of formworks. Locations that are hard to reach are easy accessible with the supply hose. The requirements for the products and the instructions for the application are respectively described in the European standard EN 14487-1 and EN 14487-2.

Due to the high pressure, the product is applied with a high density. Beneficial consequences are a low water permeability and water absorption, good freeze and daw properties, reduced formation of cracks after curing and an increased wear resistance. In preparation of the surface, an insertion layer is not necessary. A decent pre-moistening should be sufficient. During the spraying process, a part of the applied material rebounds, in particular the course aggregates. Because of this, the final mixture will contain more fine particles than in the original mixture. As a result, the adhesion improves and the shrinkage increases. This side effect may be prevented by adding appropriate additives. Moreover, the grading curve of the aggregates is decisive for the appearance of the finishing surface and the layer thickness of the repair mortar.

There are two kinds of application methods: a dry spray method and a wet spray method. For this master thesis and in function of the application on the harbour ‘Marina Internacional’, only the dry spray method is covered. In addition, this method has a wider availability of products on the market.

Dry spray method

The most common mixtures for the dry spray method are based on hydraulic binders. Thanks to the addition of polymers, the risk of shrinkage cracks can be reduced. Besides of that, PCC mortars may be used where dry polymers have been added to in advance. This type is less common due to the excessive cost.

For this method, the dry mixture is added to a vat where a flexible hose is stowed through the substance by compressed air. The moment before the mixture leaves the hose, the water is added. Consequently, the substance is mixed with a swirling motion in the nozzle. The correct dosing of the water quantity is of mayor importance and should be adjusted continuously by experienced staff. This involves that the dry mixture should be driven with a constant velocity through the nozzle. An illustration of the equipment is given in figure 6.4.

As water or additives are not required during transport or processing of the mixture, a very low water-cement ratio can be achieved up to 0.30. In practice, this ratio cannot be determined with high accuracy. Because of the low W/C ratio, the mixture has a very low slump which enables application on the ceiling. To prevent dust formation, one may moisten the dry mixture with a mass of water up to 2 % in relation to the mass of the dry mixture. Moreover, additives may offer a solution for this issue.

A disadvantage of the method is the high part of rebounded aggregates, compared to the wet spray method.

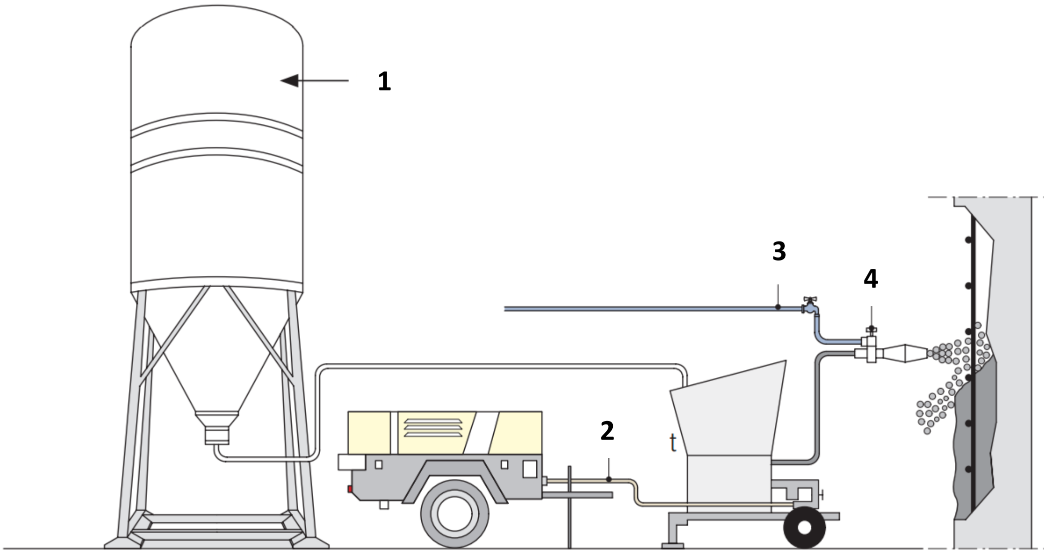


Figure 6.4: Illustration of the process of the dry spray method

With

1	Dry mixture
2	Compressed air
3	Water

4 Nozzle

6.2.4.6 *Post-treatment*

After the application of the repair mortar, an adapted post-treatment should be conducted to ensure a durable repair. When using resin bound mortars, the surface should be protected against moisture and direct sunrays. In case of repair mortars with hydraulic binders, dehydration should be avoided. Dehydration may be prevented by keeping the surface moistened with wet cloths or curing compounds. This last product may never be used between two seductive layers of mortar. By selecting the curing compound, eventual later protecting coatings should be taken into account as it could influence the adhesion.

6.2.4.7 *Levelling of the surface*

In most cases, it is desired to obtain a smooth and levelled surface after the repair for aesthetical reasons and to facilitate the application of protection coatings. This result may be obtained by applying an equalizing or smoothening mortar on the repaired surface. These mortars contain hydraulic binders and are applied in very thin layers, hence, a sound moistening of the substrate and an adapted post-treatment are indispensable. When a protection coating on the finished surface is requested, it may serve as a post-treatment of the smoothening layer.

Application to the yachting harbour ‘Marina Internacional Torrevieja’

7 Investigation

7.1 Introduction and location

‘Puerto Deportivo Marina Internacional’ is the name of a local yachting harbour in Torrevieja. The harbour is operated by Club Nautico and used for recreative activities. The harbour was built in 1980, two years after ‘Real Club Nautico Torrevieja’. The newest harbour ‘Puerto Deportivo Marina Salinas’ was built in 2005. The city is surrounded by two salt lakes, as showed in figure 7.1, which have currently a production of one million tons together annually. The industry started more than one century ago and has international export for several purposes: de-icing salt, fishery salt and salt for industrial purposes. Another important economical factor for the city is tourism, mainly sustained by the three yachting harbours.

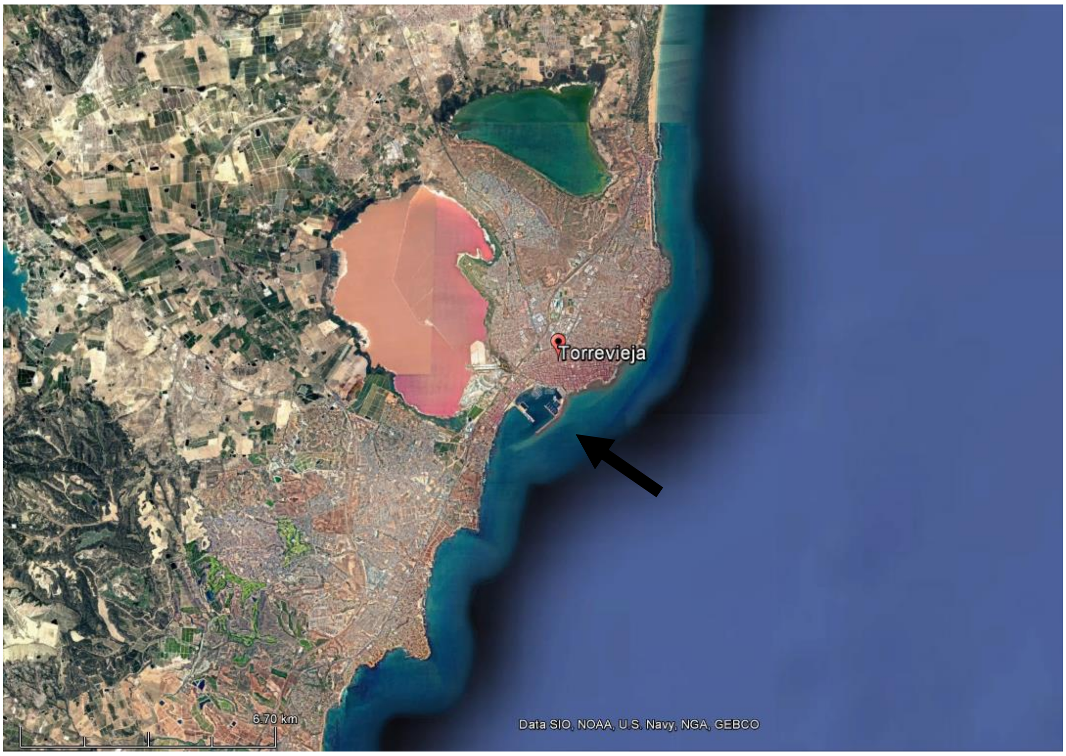


Figure 7.1: Overview of the harbour ‘Marina Internacional’ (source: Google maps) - scale: 1/100 000

Figure 7.2 gives a clear overview of distinct parts of the whole harbour of Torrevieja. The area is surrounded by a breakwater of 1440 m which makes a bend with angle of 161° after about 600 m. A second breakwater on the other side has a length of 788 m. The function of the dike is related with the explotation of the saltmines. The structure is equipped with a conveyor to transport the salt to the place of loading and unloading. The mayor functions of the harbour are salt industry and recreative yachting while fishery has a smaller share.

Figure 7.2: Overview of the harbour 'Marina Internaciona'l (source: Google maps) - scale: 1/10 000

1	Puerto Deportivo Marina Internacional
2	Real Club Nautico
3	Puerto Deportivo Marina Salinas
4	Breakwater
5	Quay of salt

The harbour 'Marina Internacional' has been investigated for this master thesis and from this point all the information given treats about this harbour.

The harbour consists of a terrestrial area with parking lots, a nautical shop, a hotel and restaurants, see appendix 1. From that point, the main entrance to the berths is constructed straight into the water. This road is accessible for vehicles of clients and staff and provides sanitary facilities, a seamen headquarters and a captaincy. The principal dock connects thirteen pontoons which give access to the 896 berths. These berths are divided into categories according to the size of the yachts. Larger ships have more draught and are situated in the south part of the harbour. A satellite photo has been added in figure 7.3.

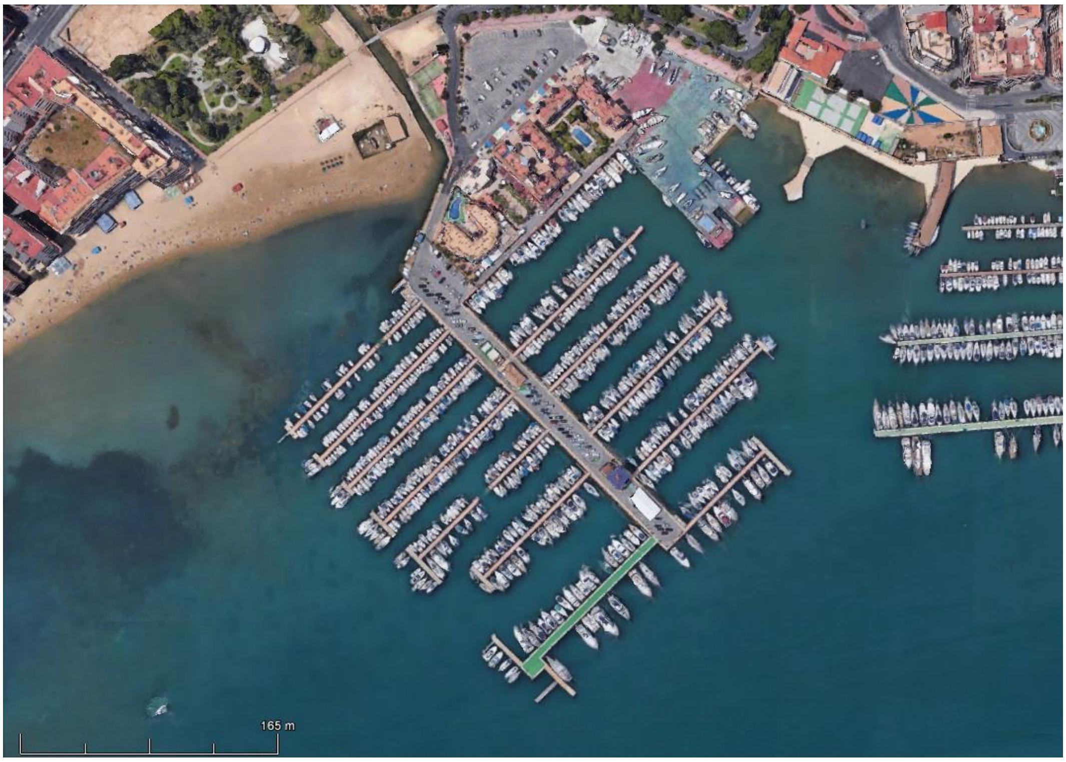


Figure 7.3: Overview of the harbour 'Marina Internacional' (source: Google maps) - scale: 1/2500

7.2 Analysis of the harbour

7.2.1 General state and history

7.2.1.1 Principal road

The principal road of the harbour is covered with asphalt on top and protected with blocks of natural stone on both sides. There is one stroke of the concrete structure underneath visible above the natural blocks which did not have any traces of corrosion. This stroke has an average height of 1,4 m above the water surface and is no part of the wetting and drying cycle under normal circumstances. The contact with chlorides will mainly consist of sprays and wind in sea atmosphere. However, the deterioration of the lower strokes behind the blocks of natural stone could be more severe as the seawater fills up the holes between the blocks. This stroke will be part of the wetting and drying cycle which is considered the most ideal environment to accelerate the corrosion process. Unfortunately, it was impossible to investigate or measure this part due to the heavy blocks obstructing the surface. The cubes of natural stone did not show any traces of deterioration.

7.2.1.2 Pontoons

For the investigation, the focus was put on the pontoons as they have the most progress in the corrosion process. They are part of the highest exposure class to chloride ingress: tidal, splash and spray or XS3 according to the European standards. The damages vary from brown-coloured stains with small cracks to parts without any concrete cover and with severe corrosion of the reinforcement bars, as they have direct contact with the sea atmosphere. In this case, the structure is no subject to a programme of improvement. The pontoons are supported by concrete columns at the ends of the slabs and by steel columns in the middle to ensure a temporary, safe construction. The columns were in a good state under as well as above the water surface. They used to be covered with a layer of paint with corrosion protective abilities, but it has been solvated by the aggressive environment. A new protective paint should be applied to anticipate corrosion. The lower part of the beam that is always submerged in water, will not show any traces of corrosion as oxygen, which is one of the fundamental elements in the process, is not present. The columns support on a square steel slab of about 0,5 by 0,5 m of the bottom of the shore. Every two months, the supports are checked and adjusted by a diver to avoid unsupported parts. The columns may move or fall because the compression between the bottom and the pontoon can vary due to settlements. In 2014, one slab broke and fell into the water. This part, together with two others has been replaced by a precast and prestressed concrete and will be reviewed in chapter 7.3.2.1.

7.2.1.3 Sea level

At the moment of the visit, the average difference in altitude between the underside of the pontoons and the water level was about 23 cm. This measurement was token at 10 o'clock in the morning when the tides are between ebb tide and high tide. This is a very small difference knowing that the maximal difference between both tides is about 18 cm. Beside the tides, the sea level is not a constant value. The highest levels occur at the end of summer and the lowest levels occur at the end winter. This effect is mainly caused by the volume change due to temperature differences. Another concerning factor, is an increase of the sea level because of global warming. All the parameters suggest that the pontoon slabs should be installed higher to decrease the contact with seawater and thus, chlorides.

7.2.1.4 Yachts

Not only the immobile structure of the harbour must handle corrosion problems, also yachts are mainly made of metals, such as steel or aluminium, and need protection systems. The most common method is the addition of cathodic protection consisting of a galvanic anode planted on the hull of the ship. The anode is made of metals such as magnesium, aluminium and zinc. Magnesium has the most negative electro potential and is more used in environments with high resistivity, for instance soil or fresh water. Salt water has a lower resistivity and requires anodes of zinc or aluminium. The frequency of changing of anodes depends on the application where the anodes have been used. They should be checked every year during the dry dock maintenance. Nowadays, more complex systems with a closed loop can measure the graduation of corrosion and adjust the supply of ions. This promising technique can avoid overprotection and is designed especially for large yachts.

7.2.2 Documentation

The harbour did not dispose over any historical or technical information about the construction of the pontoons, except the floor plan of the architect, which was used for the design and classification of the harbour. The pontoons were created onshore next to the harbour. After curation, they were installed on the concrete columns. This information was obtained orally.

7.3 Evaluation of the structure

7.3.1 Pontoons

The pontoons were the subject of the experiments in this master thesis. They are composed by isostatic concrete slabs (1060 x 300 x 60 cm) and supported by concrete columns at the ends over a stroke of 0,5 m. The joints between the distinct parts are open and do not transfer any forces to adjacent parts. There is a pipeline on both sides of the pontoon with electrical cables and fresh water supply. Every 10 m, tapping points are attached to the structure to supply the yachts. Besides, anchor points are mounted with bolts to keep the yachts in its berth. The finishing layer of the running surface is brushed in order to achieve a good grip in all-weather circumstances.

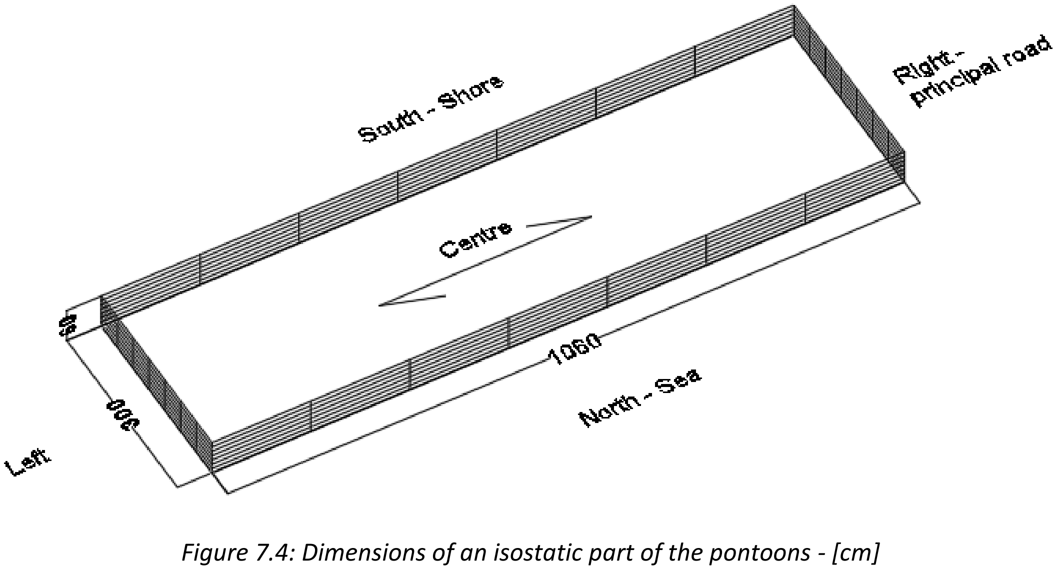


Figure 7.4: Dimensions of an isostatic part of the pontoons - [cm]

7.3.2 Visual inspection

A common and logic step of the investigation, is a visual inspection, see chapter 5.3.1. It should give an estimation of the current state of the structure and is conducted by experts. For this master thesis, the inspection was done by the author, who has a standard academical experience in this matter.

Afterwards, all the pontoons were assessed by the damage due to corrosion. The applied method was as follows. Every isostatic part of the pontoons was analysed on both sides, north and south. For each side, an indication was given with the rate of corrosion. The different rates are listed in table 7.1, accompanied by representative photos in figure 7.5. The map with colour indications for the whole harbour is added in appendix 2.

🟩	(1) No stains of corrosion on the surface
🟨	(2) Concrete cracks and stains of corrosion on the surface
🟥	(3) Concrete cover is strongly reduced due to corrosion
▨	(4) Reinforcement bars are visible

Table 7.1: Indication with the corrosion rate of the pontoons.

Figure 7.5: Photos of the corrosion rate matching the four indications

The inspection was conducted by the author itself. Each pontoon was assessed from on the running surface of the structure itself, by leaning forward two times on each side. The final indication was based on the worst part of the surface of one side. This time-consuming process could have been conducted more efficiently in case there were no yachts parked besides the pontoons. Hence, it would have been possible to inspect the concrete slabs from a motorboat which gives you a better overview and, moreover, the opportunity to observe the downside of the pontoons. The sides are only a fraction of the corrosion rate of the whole slab as the surface has the same conditions as the lower part of the sides for the corrosion process. For this reason, one should be careful to draw conclusions based on this investigation. Figure 7.6 shows some photos of the state of one isostatic part of the pontoons. Large areas of reinforcement appear without concrete cover.

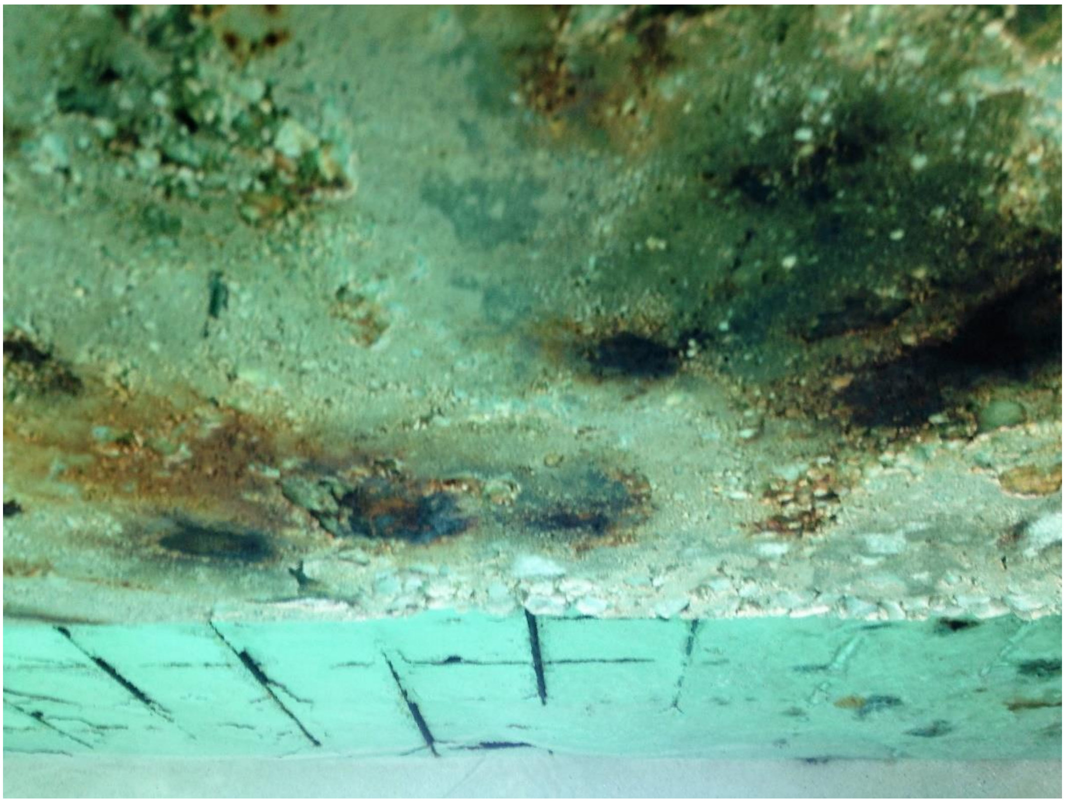


Figure 7.6: Photos of the current state of the downside of the pontoons

7.3.2.1 Replaced slabs

The first category of the legend is an indication for no stains of corrosion on the surface. This category was only used for replaced slabs. Three separated parts of the pontoons were changed as the damage due to corrosion was not sustainable. One of them, part 6 of pontoon 5, collapsed in 2014. The slabs are exchanged by a prestressed and prefabricated elements. The vertical section contains hollow circles to reduce the dead weight. Another beneficial is the decreased height of the profile, due to prestressed system. This gives the opportunity to increase the height difference between the slab and the sea level. A visual inspection revealed some concerning facts: the reinforcement wires of the prestressed elements are cut at the same level of the concrete surface which means there is no concrete protection over the wires. Even with a high concrete quality, chlorides have direct contact with the end of the strings and corrosion problems will occur rapidly. Photos of the new, prestressed slabs are given in figure 7.7. In the corner is one unprotected reinforcement wire visible. It was impossible to take pictures of the full vertical section of the slab, as it was restricted by the neighbouring, concrete beam.

Figure 7.7: Photo of the prestressed, prefabricated slab of the pontoon

7.3.3 Motivation of selection

One isostatic part has been chosen to conduct the detailed investigation. The regarding piece is part of pontoon number five and is the third one in a row starting from the principal road. It belongs to the (3) category of corrosion, indicated during the visual inspection. Every part of the pontoons was available for investigation as the tests were non-destructive, do not disturb the silence or prevent passage.

The scientific determination was made as follows. The categories with graduation of the corrosion process were made on a basis of visible damage due to corrosion. This damage was always situated around the base reinforcement, thus the edge of the pontoon between the side surface and the downside surface. The experiments, however, can only take place on a plane surface or above this damage. After thirty years of service in the tidal, splash and spray zone, the concrete shows visible damage, such as delamination, cracks and in particular coarse air voids. This deterioration creates a direct entrance for chlorides into the concrete cover. Regardless of the current state of the reinforcement, the concrete cover should be removed in its whole to eliminate the chloride content in the structure. For this reason, it was not chosen to detect the concrete for every category. The measurements taken serve as a reference for all the pontoons.

In case of disposal of a resistivity meter, measurements for each corrosion category would be recommended.

7.3.4 Rebound hammer

7.3.4.1 General information

The current value of the compressive strength of the concrete can be estimated with a concrete test hammer or sclerometer. The test procedures are described by ASTM C 805 - 97 Standard Test Method. The method has been explained in chapter 5.3.2.1. General data about the experiment is listed below:

- Date: 24 May 2017;
- Testing equipment, see figure 7.8:
    - Concrete test hammer (Original SCHMIDT - model N/NR), produced in Zurich, calibration certificate of ISO 9001 standard with instrument number: 157185, calibrated on 4 March 2005 by Proceq SA Switzerland;
    - Grinder;
    - Chalk;
    - Ruler;
- The concrete mixture proportions of the test areas were unknown;
- The curing conditions of the test areas were unknown;
- The type of exposure of the test areas: tidal, splash and spray or XS3;
- Air temperature at time of measurements: 25 °C.

Betonprüfhammer
SCHMIDT
Concrete Test Hammer
KALIBRIER-ZERTIFIKAT
CALIBRATION CERTIFICATE

Instrument No.: 157185
Datum/Date: 04.03.05
Prüfer/Responsible: Cemal UZUN

ISO 9001
proceq
PROCEQ SA SWITZERLAND

Figure 7.8: Test equipment and calibration certificate

7.3.4.2 Location and method

The slab has been tested at nine different locations of which three on both sides and three on the upper side. Both ends and the bottom were restricted areas and have not been tested.

The test points were distributed over the whole surface with varying coordinates. Each area was first prepared with a grinding wheel to erase imperfections, stains of mortar and moss. The depth of the grinded surface was less than 1 mm. Then a grid was marked with chalk as showed in figure 7.9. Photos of the preparation are added in figure 7.10. On each point of the grid, a test was conducted. The distance between the dots is 63.5 mm or 2.5' according to ASTM C 805 Standard Test Method. The reason to test each point of the grid instead of one test point is as follows: concrete is heterogenic material containing elements with high compressive resistance, such as aggregates and reinforcement bars, complemented by the binding material with a lower resistance. To anticipate this effect, the average is taken of the results of each point of the grid.

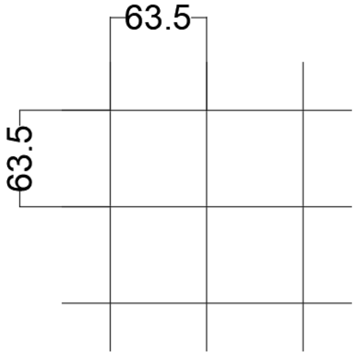


Figure 7.9: Dimensions of the test grid for the rebound hammer - [mm]

Figure 7.10: Preparation of the surface for the rebound hammer

1	Original surface
2	Grinded surface
3	Application of the test grid on the surface

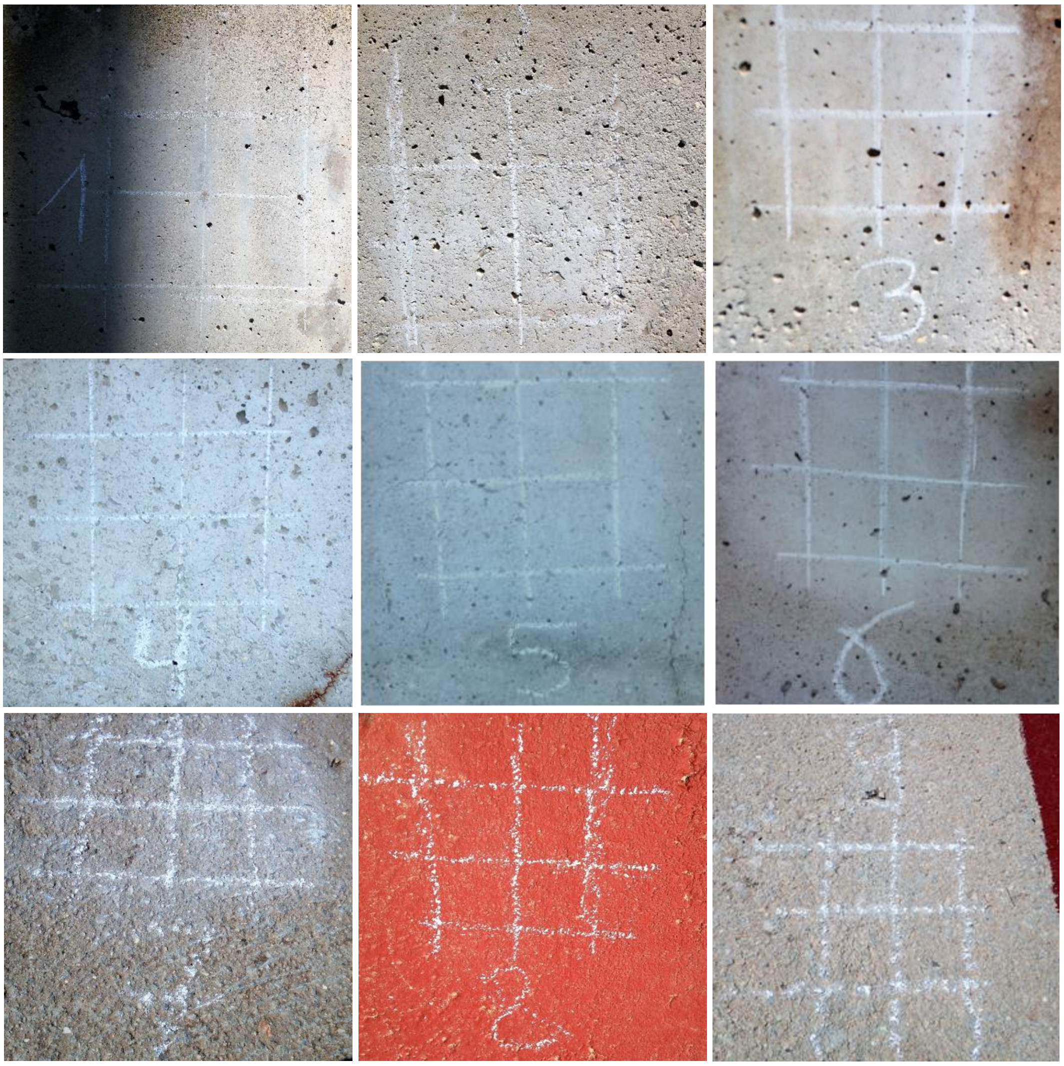


Figure 7.11: Photos of all the prepared areas for the rebound hammer test

The test areas are shown in the photos of figure 7.11. The areas on the sides of the slabs, from one to six, were relative plane. However, large air voids were present at the surface. The measurements were always token at least 1 cm further than such a void to avoid influenced results.

7.3.4.3 Results

The following tables show the results of the nine test areas. The number recorded on the linear scale of the equipment is the rebound number. This parameter does not have any units and should be converted to compressive strength via empirical graphs, which is seen in figure 7.12. There are two graphs added to the hammer: one of them is the relation between the rebound number and the compressive strength of concrete on cylinders $F_{ck,cyl}$ [N/mm²]. The other graph shows the relation between the rebound parameter and the compressive strength of concrete on cubes $F_{ck,cub}$ [N/mm²]. All the results have been converted to $F_{ck,cyl}$. There are three graphs for each position of the hammer during the test: perpendicular to a vertical surface and perpendicular to a top and a bottom surface. A rebound hammer test conducted on an upper surface will have a lower rebound number than on a downside surface for the same concrete quality due to gravity. The tables 7.2 to 7.11 contain the relevant information and results of each test area. Results marked in dark grey were out of range of the graph. In this case, the maximum result in range has been recorded.

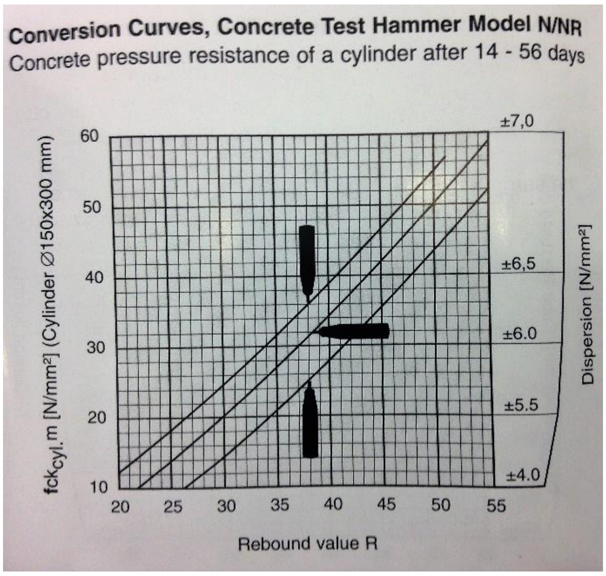


Figure 7.12: Conversion curves for the rebound hammer - model N/NR

Vertical surface - South - Right

	Rebound R			F_{ck} - [N/mm²]	
50	50	42	50	50	36
54	56	44	56	58	40
50	50	48	50	50	46

Table 7.2: Results rebound hammer test - Vertical surface - South - Right

Vertical surface - South - Center

	Rebound R			F_{ck} - [N/mm²]	
40	36	40	32	28	32
38	40	40	32	32	32
40	40	40	32	32	32

Table 7.3: Results rebound hammer test - Vertical surface - South - Center

Vertical surface - South - Left

Rebound R			F_{ck} - [N/mm²]		
48	44	46	46	40	44
38	48	48	32	46	46
46	42	42	44	36	36

Table 7.4: Results rebound hammer test - Vertical surface - South - Left

Vertical surface - North - Right

Rebound R			F_{ck} - [N/mm²]		
44	48	44	40	46	40
44	46	50	40	44	50
50	44	44	50	40	40

Table 7.5: Results rebound hammer test - Vertical surface - North - Right

Vertical surface - North - Centre (1)

Rebound R			F_{ck} - [N/mm²]		
46	42	40	44	36	32
34	40	32	26	32	22
46	40	38	44	32	32

Table 7.6: Results rebound hammer test - Vertical surface - North - Centre (1)

Vertical surface - North - Centre (2)

Rebound R			F_{ck} - [N/mm²]		
40	40	42	32	32	36
38	40	28	32	32	19
42	34	38	36	26	32

Table 7.7: Results rebound hammer test - Vertical surface - North - Centre (2)

Vertical surface - North - Left

Rebound R			F_{ck} - [N/mm²]		
40	38	40	32	32	32
32	28	40	22	19	32
38	34	42	32	26	36

Table 7.8: Results rebound hammer test - Vertical surface - North - Left

Horizontal surface - Upside - Right

Rebound R			F_{ck} - [N/mm^2]		
50	42	26	54	40	18
28	36	30	22	32	24
38	32	30	36	26	24

Table 7.9: Results rebound hammer test - Horizontal surface - Upside - Right

Horizontal surface - Upside - Centre

Rebound R			F_{ck} - [N/mm^2]		
52	40	32	58	38	26
30	40	40	24	38	38
40	26	34	38	18	30

Table 7.10: Results rebound hammer test - Horizontal surface - Upside - Centre

Horizontal surface - Upside - Left

Rebound R			F_{ck} - [N/mm^2]		
36	26	26	32	18	18
32	26	24	26	18	16
26	22	26	18	12	18

Table 7.11: Results rebound hammer test - Horizontal surface - Upside - Left

For the calculation, the average has been taken of the concrete compressive strength on cylinders. The results differing more than 6 units of the average value should be discarded. These values have been marked in dark grey. In case of three or more discarded records, a new test must be taken in the same testing area. In the following tables 7.12 to 7.21, the records out of range were marked in dark grey. Average (a) includes all the results and average (b) includes all the results within range. In case of three or more results out of range, this last parameter is should not be used as a reference for the concrete strength and is marked in dark grey.

Vertical surface - South - Right

F_{ck} - [N/mm^2]			Average (a)	48.44
32	32	32	Min value	42.44
26	26	40	Max value	54.44
18	18	18	Average (b)	46.00

Table 7.12: Statistics results rebound hammer - Vertical surface - South - Right

Vertical surface - South - Centre

F_{ck} - [N/mm^2]			Average (a)	31.56
32	28	32	Min value	25.56
32	32	32	Max value	37.56
32	32	32	Average (b)	31.56

Table 7.13: Statistics results rebound hammer - Vertical surface - South - Centre

Vertical surface - South - Left

F_{ck} - [N/mm^2]			Average (a)	41.11
46	40	44	Min value	35.11
32	46	46	Max value	47.11
44	36	36	Average (b)	42.25

Table 7.14: Statistics results rebound hammer - Vertical surface - South - Left

Vertical surface - North - Right

F_{ck} - [N/mm^2]			Average (a)	43.33
40	46	40	Min value	37.33
40	44	50	Max value	49.33
50	40	40	Average (b)	41.43

Table 7.15: Statistics results rebound hammer - Vertical surface - North - Right

Vertical surface - North - Centre (1)

F_{ck} - [N/mm^2]			Average (a)	33.33
44	36	32	Min value	27.33
26	32	22	Max value	39.33
44	32	32	Average (b)	31.00

Table 7.16: Statistics results rebound hammer - Vertical surface - North - Centre (1)

Vertical surface - North - Centre (2)

F_{ck} - [N/mm^2]			Average (a)	30.78
32	32	36	Min value	24.78
32	32	19	Max value	36.78
36	26	32	Average (b)	32.25

Table 7.17: Statistics results rebound hammer - Vertical surface - North - Centre (2)

Vertical surface - North - Left

F_{ck} - [N/mm²]			Average (a)	31.67
32	32	32	Min value	25.67
22	19	32	Max value	37.67
32	26	36	Average (b)	31.67

Table 7.18: Statistics results rebound hammer - Vertical surface - North - Left

Horizontal surface - Above - Right

F_{ck} - [N/mm²]			Average (a)	30.67
54	40	18	Min value	24.67
22	32	24	Max value	36.67
36	26	24	Average (b)	30.33

Table 7.19: Statistics results rebound hammer - Horizontal surface - Above - Right

Horizontal surface - Above - Centre

F_{ck} - [N/mm²]			Average (a)	34.22
52	40	32	Min value	28.22
30	40	40	Max value	40.22
40	26	34	Average (b)	36.40

Table 7.20: Statistics results rebound hammer - Horizontal surface - Above - Centre

Horizontal surface - Above - Left

F_{ck} - [N/mm²]			Average (a)	19.56
32	18	18	Min value	13.56
26	18	16	Max value	25.56
18	12	18	Average (b)	17.67

Table 7.21: Statistics results rebound hammer - Horizontal surface - Above - Left

The test in the centrum of the north side of the pontoon returned three records out of range and has been retaken with valid results. The tests on the upper surface have failed on the three different spots due to the rough finishing layer. Grinding off this layer was not allowed, as visible damage would remain. Repetitive tests on the same spot produced very fluctuating results hence only one full test per area was token and should not be used as a reference for the current compressive strength. In the end, six useable measures may be used to draw conclusions.

7.3.4.4 Observations

During the test, it was observed that some test areas sounded hollow when conducting the experiment. This could indicate delamination or air voids underneath the surface. A description of the surface and an acoustic observation by ear is added in the following list:

- Test area 1: mainly fine air voids visible;
- Test area 2: course air voids visible up to 2 mm;
- Test area 3: hollow sound for eight out of nine measures, mainly fine air voids visible accompanied by voids up to 5 mm;
- Test area 4: rough surface after grinding, mainly large air voids up to 1 mm;
- Test area 5: hollow sound for nine out of nine measures, mainly fine air voids visible;
- Test area 6: hollow sound for nine out of nine measures, mainly fine air voids visible, accompanied by voids up to 4 mm;
- Test area 7: course finishing layer;
- Test area 8: course finishing layer;
- Test area 9: course finishing layer, hollow sound for nine out of nine measures.

Test location	**F_{ck} [N/mm²]**	**Hollow sound**	**Visible air voids**	**Roughness surface**
1	46.00	no	fine	low
2	31.56	no	large	low
3	42.25	yes	fine	low
4	41.43	no	large	low
5	32.25	yes	fine	low
6	31.67	yes	fine	low

Table 7.22: Comparison between the valid records and the observations

Table 7.22 compares the valid records with the observations. Test location 1 has the maximum, measured value for F_{ck}: 46.00 N/mm². The observations support this fact in such a way that they form a combination of optimal parameters. No hollow sound was noticed, and thus no indication of air voids or delamination, fine and dispersed visible air voids and low roughness at the surface. When we compare test location 1, 3, 5 and 6, the only parameter that changes, is the hollow sound for the acoustic observation. Test location 1 has a higher record than the others, which could indicate a relation between the sound and the compressive strength. Test locations with large visible air voids do not differ significantly from the remaining records. The higher the amount of records the more representative these conclusions would be. Test locations 7, 8 and 9 are not included in the table.

7.3.4.5 Analysis results

The results of the concrete hammer test have been applied to a statistical analysis. The number of separated tests that have been taken is 90. 22 of them have been discarded by differing 6 units or more from the mean value. The provisional results are 68 succeeded tests. This data is added to Table 7.23.

Statistical parameters	F_{ck} [N/mm²]
Total number of tests	90
Total number of valid tests	68

Table 7.23: Numerical data of the separate tests of the concrete hammer tests

However, the separated tests cannot be used for statistical analysis according to ASTM C 805 - 97. One should continue with the mean value of the grid, only if two or less results are discarded. The values for 'Average (b)' only consider the results in range. These values will be called 'records' from now on to prevent confusion with the test results. As mentioned in chapter 7.3.4.3, 6 valid records are registered. Statistical parameters are given in table 7.24.

Statistical parameters	F_{ck} [N/mm²]
Number of valid records	6
Number of discarded records	4
Mean value of the records	37.53
Standard deviation of the records	6.44

Table 7.24: Statistical data of the valid mean values of the concrete hammer test

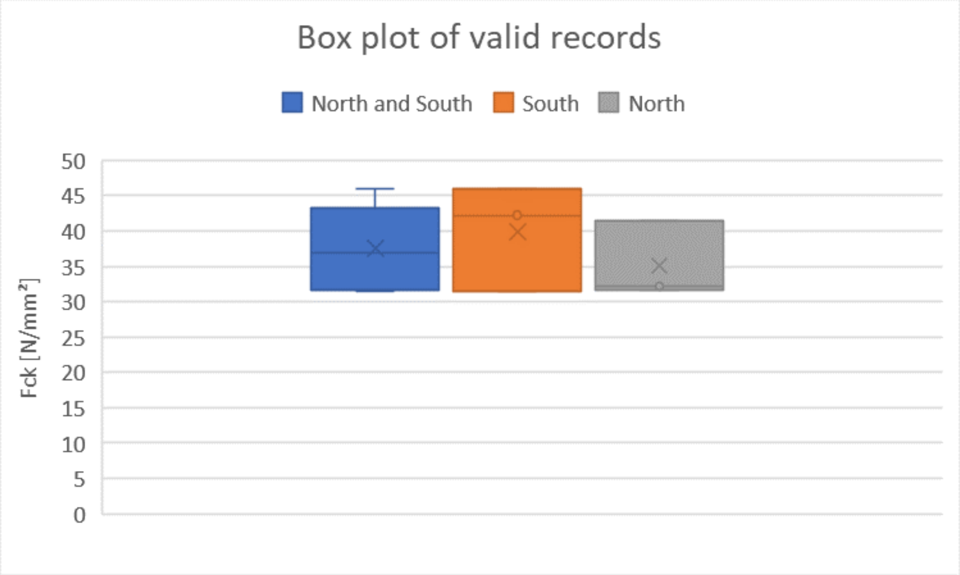


Figure 7.13: Box plot of the results of the concrete test hammer

A box plot in figure 7.13 gives an illustration of the statistical data in table 7.24. Although the extreme values have been discarded, the records are very dispersed and differ with more than 10 N/mm². A comparison with the design compressive strength could offer more conclusions about the decrease over the years of service in marine environment. Unfortunately, this information was not available. For further conclusions and the design of repair works, the mean value of the records, will be used as a reference for the current compressive strength of the concrete slabs.

7.3.5 Concrete cracks

7.3.5.1 General information

When a concrete structure is designed, a maximum value for the concrete cracks is determined on basis of the exposure class. These target values are found in the European standard EN 1992-1-1.

Information about concrete cracks was collected to acknowledge the state of the deteriorated concrete. The results are useful when injections are proposed for the repair works in order to estimate quantity and type of product needed. General data about the experiment is listed below:

- Date: 24 May 2017;
- Testing equipment, see figure 7.14:

    - Plastic strip with scaled lines: the range starts at 0.03 mm and ends at 3.00 mm with a logarithmic graduation;
    - Measuring tape;

- The concrete mixture proportions of the test surfaces were unknown;
- The curing conditions of the test surfaces were unknown;
- The type of exposure of the test surfaces: tidal, splash and spray or XS3;
- Air temperature at time of measurements: 25 °C.

Figure 7.14: Equipment for the measurements of concrete cracks

7.3.5.2 Location and method

For this investigation, both sides of the same isostatic part of the pontoon as for the concrete hammer test have been measured. The surface was divided in vertical strokes of 10 cm over the length of 1060 cm. Each stroke was examined on concrete cracks and only the crack with the largest width was recorded. Moreover, the appearance of corrosion stains has been noted.

The measurement was done as follows. The measuring tape was rolled out over the edge of the slab. Then, the intervals were examined, one by one, while being in a reclining position on the running surface of the pontoon. First, the crack width was measured with the plastic strip. Then, the whether or not appearance of stains was noted.

7.3.5.3 Results

There are two graphs added to the appendix. One of them shows the results for the side on the south, see appendix 3, and the other one shows the results for the north side, see appendix 4. For each graph, the results of crack width are displayed in combination with the whether or not appearance of corrosion stains. In case of corrosion stains, number one was noted and a zero was noted for a corrosion-free interval. The horizontal axis shows the distance x_m, which is the distance of the starting point to the centre of the interval.

7.3.5.4 Analysis results

The designed, maximum crack width depends on the exposure class of the structure. In this case, the exposure class is XS3. The designed crack width for reinforced concrete according to EN 1992-1-1 is 0.30 mm.

In table 7.25 is a statistical resume made of the results, which is graphically illustrated in a box plot, added to the appendix 5.

Statistical parameters	**North**	**South**
Number of intervals	106	106
Maximum width cracks - [mm]	4	6
Minimum width cracks - [mm]	0	0
Mean of width cracks - [mm]	0.62	1.17
Median of width cracks - [mm]	0.40	0.50
Standard deviation of width cracks - [mm]	0.64	1.45
Percentage of the amount of values exceeding 0.30 mm - [%]	56.60	58.49
Percentage of corrosion appearance - [%]	40.57	26.42

Table 7.25: Statistical data about the crack width and the appearance of corrosion stains

The measurement strip was in a used condition in such a way that the scaled lines were not tight printed anymore. For this reason, cracks smaller than 0.20 mm were recorded as 0 mm. In addition, the remaining recorded results could differ more from reality than the accuracy of the measurement strip, due to vague printed lines. Cracks with a size larger than 3 mm were recorded with a measuring tape with an accuracy of 1 mm.

For a comparison between the north and the south side, the mean values are not the best parameter as a few extreme results increase this value. This explains the high value for the standard deviation for the south side. Instead, we use the median values, which are more similar.

The percentage of values exceeding 0.30 mm is very high. Both sides have more than half of the intervals that contain cracks with a larger width than the design value. This is not a sustainable situation, especially for a such a severe environment. The percentage of the corrosion appearance differs with more than 15 %. One would expect a higher value for the south side, as it is oriented to the seaside and thus exposed to onshore wind.

The aim of combining the results of the crack width with the appearance of corrosion stains in one graph was to find a link between both parameters. Ultimately, there is no significant relation found. The appearance of corrosion on the north side is fluctuating quickly and occurs with the increasing and the decreasing of the crack width. The intervals with extreme values are only half corroded. This is not the case for the south side, where high values appear without visible corrosion, being aware of the lower percentage of occurring corrosion.

7.3.6 Conclusion

The investigation started with a visual inspection. The results vary from visible corrosion stains with concrete cracks to the loss of the whole concrete cover and strongly corroded reinforcement bars. The cause of the corrosion is the ingress of chlorides in the structure from the marine environment.

The experiment with the concrete test hammer gave an estimation of the current state of the concrete strength with a mean value of 37.53 N/mm². This value should be compared to the design value of the concrete slabs, however, this data was not available.

The collected details about the width of the cracks resulted in an exceedance of 56.60 % and 58.49 % for respectively the north and the south side of the pontoon. Moreover, 40.57 % and 26.42 % of the intervals showed corrosion stains. There was no link found between both types of deterioration.

Considering these results of the investigation, repair works are demanded to guarantee a sustainable and safe structure. For each category, an adjusted repair work is designed in chapter 0. Except for the slabs that have been replaced recently and are indicated as corrosion category green (1).

7.4 Management strategy

Repair works are demanded and during execution, the harbour will not be able to operate in normal conditions. The head of the harbour may decide, eventually in consultation with the contractor, between a few management options:

- Repair the whole or a part of the construction;
- Rebuild the whole or a part of the construction.

Demolition is not a part of the options, as the head of ‘Marina Internacional’ plans to continue the operation of the harbour, regardless of repair works. Some crucial factors to take into consideration during the process of decision are as follows:

- Targeted lifetime after repair works and protection;
- Required level of sustainability;
- Safety risks during the repair works;
- Possibility for further repair works in the future, such as accessibility and maintenance;
- The consequences and probability of a constructive mistake;
- The consequences and probability of partial failure.

Beside the factors in function of the harbour itself, it is recommended to consider the environment:

- Protection against sun and salt during the repair works;
- The impact on the environment or limitations imposed by the environment, e.g. the sea;
- Noise pollution and dust formation;
- The required time for the execution of the repair works.

All future inspections and repair works, that shall be conducted during the designed lifetime of the construction, should also be considered as part of the management strategy.

The management strategy is no part of this master thesis and will further not be treated.

8 Repair work

8.1 Design of the repair work

8.1.1 Design parameters

For the design of the repair works, the following parameters are taken in account:

- The type of the deterioration: corrosion of the rebars, delamination, cracks and failure of the concrete;
- The cause of the deterioration: ingress of chlorides with an exposure class XS3 to marine environment;
- The scope of the deterioration: present on the sides and on the downside surfaces of every isostatic part of the pontoons, as seen in figure 8.1 except for the replaced slabs;
- The results of the investigation and the division in categories.

Based on the information given in the previous list, a design of repair work is proposed for every category of corrosion. Except for the (1) category, which only includes replaced slabs.

Figure 8.1: Photos of the different surfaces of the deteriorated slabs

For the design of the repair works, constructive information of the structure is required. Unfortunately, there were no constructive plans or technical information disposable. Advanced pachometers are able to determine the location and the diameter of the embedded rebars, but this equipment was not available. In order to continue this master thesis, an estimation of the dimensions of the reinforcement bars has been made, based on measurements of exposed rebars. The measurements are only an estimation of the dimensions of the embedded bars, as corroded material expands. A two-dimensional plan of a section in the length and the width of the slab with estimated dimensions is added to the annex, see respectively appendix 6 and 7.

8.1.2 Selection of repair method

As mentioned in chapter 7.3.3, thirty years of service in the tidal, splash and spray zone with damage, such as delamination, cracks and in particular coarse air voids, creates a direct entrance for chlorides into the concrete cover. This theory is applicable to each category. Regardless of the current state of the reinforcement, the concrete cover should be removed in its whole to eliminate the mayor part of chloride content in the structure. For this reason, it is chosen to apply Principle 3 - Concrete restoration (CR) of EN 1504, which is covered into detail in chapter 6.2.

The depth of the deteriorated zone inside the structure and the percentage of corrosion of the rebars cannot be checked without destructive actions or special equipment. In practice, this should be examined by an expert during the repair works. To continue this master thesis, an estimation of the deteriorated zone and the corroded part has been made for each category.

The running surfaces do not show any kind of deterioration and are still in a sustainable state. The downside of the slab was not accessible for investigation, however, photos could be taken of the regarding surface of one slab, see figure 8.1. Due to few information of the downside of the slabs, the same ratio of deterioration as the sides will be taken.

8.1.3 Selection of the repair mortar

The selection of the type of repair mortar is based on several parameters as described in chapter 6.2.2.3 and proceeded with the help of a table with performance requirements for the product, imposed by EN 1504-3. For the repair works of the pontoons, a repair mortar for structural purposes of class R4 is needed for the following reasons. First, the substance will fill the space around the reinforcement and must guarantee a sound transmission of loads. Second, the design compressive strength of the repair mortar should not be lower than average compressive strength at the surface of the current state, which is 37.53 N/mm², see chapter 7.3.4.5. The most important parameters of the class R4, regarding the marine environment, are listed below:

- Chloride ion content ≤ 0.05 %;

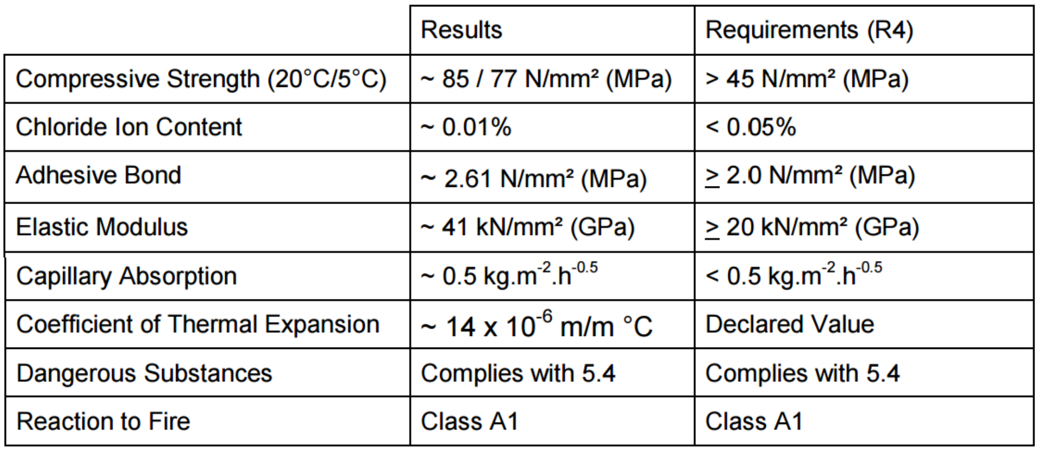
- Adhesive bond ≥ 2.0 N/mm²;
- Restrained shrinkage / expansion: ≥ 2.0 N/mm²;
- Capillary absorption $\leq 0.5\ kg \cdot m^{-2} \cdot h^{-0.5}$;

The type of application should be adjusted for:

- Large volumes;
- Structures with edges and corners;
- Layers up to 50 mm in order to limit the number of layers to two;
- Inclined surfaces.

The manual application is excluded, being too labour-intensive for large volumes. The dry spray application is preferred for large volumes and large surfaces. The shape of the pontoons implies a lot of edges and corners, which would make the spray process complex to ensure a constant layer on each surface. Moreover, the products of Sika designed for this application do not meet the requirements for class R4. The only application method left is the pouring method, also recommended for large volumes. However, the setup of the formwork may involve labour-intensive work.

Based on the requirements and the determination, the product Sika®MonoTop-614F has been chosen. This is a cementitious repair mortar with high flow, containing shrinkage compensators and high range water-reducing agents, meeting the requirements of Class R4 of BS EN 1504-3. The technical information sheet and the safety instructions have been added to the annex, see respectively appendix 8 and 9. The product is recommended for marine structures and exceeds the required layer thickness for this construction. It is based on Portland cement type CEM I and can be poured or pumped into a formwork. A comparison between the properties of the repair mortar and the requirements for class R4 is added in figure 8.2. All results meet the requirements.



	Results	Requirements (R4)
Compressive Strength (20°C/5°C)	~ 85 / 77 N/mm² (MPa)	> 45 N/mm² (MPa)
Chloride Ion Content	~ 0.01%	< 0.05%
Adhesive Bond	~ 2.61 N/mm² (MPa)	$\geq$ 2.0 N/mm² (MPa)
Elastic Modulus	~ 41 kN/mm² (GPa)	$\geq$ 20 kN/mm² (GPa)
Capillary Absorption	~ $0.5\ kg.m^{-2}.h^{-0.5}$	< $0.5\ kg.m^{-2}.h^{-0.5}$
Coefficient of Thermal Expansion	~ 14×10^{-6} m/m °C	Declared Value
Dangerous Substances	Complies with 5.4	Complies with 5.4
Reaction to Fire	Class A1	Class A1

Figure 8.2: Comparison between the properties of Sika®MonoTop-614F and the requirements of class R4

8.1.4 Columns

Before bringing concrete slabs back to its original position, the columns should be increased with height of about 30 cm. The purpose is to reduce the contact with the seawater and to anticipate on the rising sea level. The design and construction of the increase of this improvement is no part of this master thesis.

8.2 Repair work

8.2.1 Preparation of the construction site

Before the start of the construction, the following aspects must be considered for a smooth functioning of the repair process.

The slabs will be transported to a near construction site for the pouring process to prevent the ingress of chlorides during the curation period and to facilitate the access to each side of the slab. Before, the cables and installed equipment on the pontoons, should be removed. Consequently, the only activities in function of the repair works at the harbour are the removal of old slabs and accessories and the installation of repaired slabs with the related equipment and cables. Hence, nuisance will be reduced to a minimum. Also, the quality of the underground, where heavy transport machines shall pass, should be checked.

Before, during and after the repair works, a safe environment must be guaranteed. Regarding the scope and the location of the construction site, the safety instructions shall be imposed by a safety advisor.

8.2.2 Removal of the concrete cover

When the slabs are placed in sheltered workshop in an upside-down position, the removal process of the deteriorated concrete can start. Each slab should be treated according to its defined corrosion category.

The constructive plans with the dimensions for the removal for each category are added to appendix 10, 11 and 12. The edges are chamfered at an angle of 120° to prevent delamination of the neighbouring, good quality concrete and the adjacent surface of sound concrete should be roughened sufficiently to provide a mechanical key between the original material and the repair mortar. Also, the concrete substrate shall be free from dust, loose material, surface contamination and materials which reduce bond or prevent suction or wetting by repair materials. The following paragraphs describe the method of removal for each category.

8.2.2.1 Category (2)

Category (2) involves the structures with concrete cracks and stains of corrosion on the surface. Cracks, delamination and air voids are present all over the concrete surface of the sides with stains of corrosion around the base reinforcement.

The depth of deteriorated concrete cover is estimated up to half of the rebars. This means that less than 50 % of the circumference is corroded. The corroded side should be oriented towards the closest exposure surface of the structure. Based on these estimations and figure 6.1, the concrete cover should be removed at least up to half of the diameter of the rebars, in this case 6.3 mm of depth around the principal beams, 6.0 mm of depth for the bottom surface and 6.2 mm of depth for the short sides. When the depth of the deteriorated zone is found larger during the removal, the new depth of concrete to be removed should be defined by an expert. After the removal, the transverse reinforcement will be revealed in a whole and the main reinforcement up to the halve of its section.

8.2.2.2 Category (3)

Category (3) involves the structures with strongly reduced concrete cover due to corrosion. Cracks, delamination and air voids are present all over the concrete surface of the sides with high presence of corrosion stains around the base reinforcement.

The depth of deteriorated concrete cover is estimated up to the backside of the rebars. This means that less than 100 % of the circumference is corroded. The corroded side should be oriented towards the closest exposure surface of the structure. Based on these estimations and figure 6.1, the concrete cover should be removed at least up to 10 mm behind the rebars, in this case 8.0 mm around the principal beams, 7.4 mm for the bottom surface and 7.6 mm of depth for the short sides. When the depth of the deteriorated zone is found larger during the removal, the new depth of concrete to be removed should be defined by an expert. After the removal, the transverse and the main reinforcement will be revealed in a whole.

8.2.2.3 Category (4)

Category (4) involves the structures with partly loss of concrete cover due to corrosion. The reinforcement bars are visible and strongly corroded with cracks, delamination and air voids are present all over the concrete surface of the sides.

The depth of deteriorated concrete cover is estimated up to 20 mm behind the rebars. This means that 100 % of the circumference is corroded. Based on these estimations and figure 6.1, the concrete cover should be removed at least up to 30 mm behind the rebars, in this case 10 mm of depth around the principal beams, 9.4 mm of depth for the bottom surface and 9.6 mm of depth for the short sides. When the depth of the deteriorated zone is found larger during the removal, the new depth of concrete to be removed should be defined by an expert. After the removal, the transverse and the main reinforcement will be revealed in a whole.

8.2.3 Reinforcement

After the removal of the concrete, the rebars are revealed in whole or in part. First, the corroded material should be removed to verify the current section and to ensure a good bonding between the rebar and the repair mortar in a later stage. The different methods of cleaning are described in chapter 6.2.3.2. The method should be applicable for large surfaces of reinforcement bars. According to the technical information sheet of the repair mortar, the surfaces can be prepared using abrasive blast cleaning techniques or high pressure waterblasting techniques with a capacity up to 60 mPa. Where exposed reinforcement is contaminated with chloride or other material which may cause corrosion, the reinforcement shall be cleaned by low pressure with a maximum pressure of 18 mPa. As the construction is located in a marine environment, there should be payed attention to this instruction. In order to fulfil both requirements, all reinforcement bars should be cleaned with the waterblasting technique. In the first stage, all rebars should be cleaned with a maximum pressure of 18 mPa. After inspection by an expert, the decision to clean chloride-free parts with a pressure of 60 mPa could be taken. The minimum standard of the surface quality in function of the repair mortar is SA2½.

Having all rebars cleaned up to the surface of the sound, metal section, an examination of the rebars, whether they are still able to fulfil the targeted function of the design, is required. Depending of the outcome, rebars can be changed or added to the current structure.

8.2.4 Application of the repair mortar

The method of application for Sika®MonoTop-614F is the pouring method. Regarding the angled surfaces, the filling process will consist of different stages. The position of the formwork and the different phases of pouring is described in the following paragraphs.

8.2.4.1 Formwork

The formwork should be positioned in such way that by filling the space of the formwork, the slab returns to its original shape. The formwork should be strong enough to withstand the hydraulic pressure and should be constructed and propped rigidly to retain its shape. The joints should be tight against leakage of the liquid substance. The texture of the surface should be smooth to obtain a finishing surface on the visible parts. Because the hydraulic pressure is relatively low, timber formwork can be used if preferred. Steel forms are recommended as they are more durable and do not shrink or warp.

First, the concrete slab should be turned upside down, in order the facilitate the formwork. Then, the formwork setup around the sides of the structure can be applied for the first stage of the pouring process, as seen in the constructive plan that has been added to appendix 13. Extra fittings should be supplied for the internal pipe to continue its function after repair. For the second stage of pouring, inclined formwork is needed. Support beams can be installed between both inclined surfaces to compensate forces. In addition, it should be anchored to the structure. A constructive plan of the second stage has been added to appendix 14. These plans are made for the slabs of corrosion category (2). Slabs belonging to another category can be repaired in an analogous way with the dimensions of removed concrete layer of the regarding category.

8.2.4.2 Pouring process

Due to the shape of the layer to be applied, the pouring process is split in two stages. In the first stage, the sides and the horizontal layer in the middle of the structure are poured. The repair mortar needs at least three days to ensure full cement hydration and to minimise cracking. The instructions for the mixing and application process can be found in the technical information sheet of the repair mortar. After the period of curation and the installation of the second formwork, the second and last part can be poured. This stage involves, the inclined surfaces and the top layer of the principal beams. Again, a period of minimum three days is required to ensure a sound curation of the repair mortar. The plans for the pouring process are added to appendix 13 and 14.

8.2.4.3 Curing treatment

After 3 days of the last phase of repair mortar, the form work can be removed. In order to ensure a durable repair, a supervised post-treatment is recommended. As the repair mortar is based on hydraulic binders, one should be aware of dehydration. This can be prevented by keeping the surface moistened with wet cloths. However, the technical information sheet of the repair mortar advises not to add additional water during the surface finishing as it will cause discoloration and cracking. Moreover, curing compounds should be chosen carefully as they may adversely affect subsequently applied products and systems. It is recommended to fully cure the slabs up to 28 days before heading back to the marine environment.

8.2.5 Positioning

When the repair mortar is fully cured, the slabs can be transported back to the harbour. During this time, the columns should have been increased as mentioned in chapter 8.1.4. Subsequently, the structures can be brought back in its original position with the required surfacing material.

8.3 Acceptance of the repair works

When a pontoon is finished, an examination is recommended to evaluate if the repaired construction meets the targeted requirements. After inspection of the safety advisor and the installation of the equipment, the pontoon should be ready for normal operation.

At the end of the project, reports should be provided with future terms, such as:

- The expected lifetime;
- The frequency and tasks of the maintenance;
- The parts that need more attention during maintenance;
- The type of repair works that would be required in case of reoccurrence of deterioration.

9 Economic analysis

This chapter implies an economic analysis of the proposed repair works for the pontoons of the harbour ‘Marina Internacional’. The result should be an estimation of the total budget required to execute the repair project. The prices for materials, equipment and workmen are based on the most recent version of ‘Cuadro de Precios de los Proyectos de Urbanización y de Edificación’, which is provided by the government of Madrid. The following chapters cover a calculation of the quantities, a calculation of the costs, a resume tendering and a global tendering.

9.1 Calculation of quantities

The first part of the economic analysis is the calculation of quantities of material or size. The parameters are deduced of the constructive plans, that have been added to appendix 15, and measured with the software package of AutoCAD. The worksheet with the calculations has been added to appendix 16. Sometimes, the results count for a half slab. This because of the division between the north side and the south side for the corrosion categories. To prevent confusion, this has been indicated clearly in the column of the units and in the explanation of the parameters.

The calculation starts with the standard parameters of a pontoon slab based on the sections A-A’ and B-B’ of the constructive plans. Here, the volume of a whole slab is calculated to estimate the weight of one piece. This is required information for transport facilities. Then, the volume of the deteriorated concrete was calculated for each corrosion category with a result for a half slab and for all the half slabs belonging to the regarding category. The volume of the rebars was neglected. Subsequently, an estimation of the total length of reinforcement bars of a whole slab was made for the cleaning process. The placement of the formwork is independent of the corrosion category and the total number of square meters of formwork needed was calculated. At last, the volume of the repair mortar to be applied was calculated for the total number of half slabs per category. Again, the volume of the rebars was neglected.

9.2 Calculation of costs

This subchapter is based on the information of the calculation of quantities. A worksheet has been added to the annex, see appendix 17, and is divided in five chapters:

01 Demolition
02 Transport

03 Composition of construction materials
04 Application of construction materials
05 Inspection

These chapters contain certain actions or materials of the repair process. Those terms consist of material, equipment and workmen needed to succeed one unit of this action. Each part is provided with the cost for one unit, duplicated with the number of units. In the end, the sum of the amounts was taken. Another term of indirect costs adds 4 % to this value, resulting in the total amount for one unit of the whole action of construction material.

A mayor part of the cost consists of the composition of the repair mortar. One bag of 25 kg mixed with water results in a volume of 11.5 l cured repair mortar, according to the technical information sheet of Sika. These bags are available for 25 €. Prices for large quantities may be reduced, but are only available on request.

9.3 Tendering

The resuming tendering combines the results of the quantities and costs of the two previous subchapters. The total amount required for the action was made deduced by the quantity and the price per unit. For each chapter, the subtotal was taken.

These values are retaken in the global tendering for an overview of the amounts for each chapter. The total amount for the execution of the project is 1 600 084.11 € before taxes. The VAT value adds 21 % to this amount, resulting in the total budget required for execution of the repair project: 1 936 101.77 €. Both working sheets can be consulted in appendix 18 and 19.

10 Conclusion

The objective of the master thesis was to elaborate a project to tackle the corrosion problems of the harbour ‘Marina Internacional’ and ensure a sustainable state for another service of 30 years.

Guided by the literature study, an investigation was conducted to examine the current state of the concrete slabs. The visible damage indicated clearly the corrosion problem and the cause of deterioration, which is the ingress of chlorides. The results of the experiments confirmed the need for renovation. Afterwards, a repair project was elaborated implying the removal of the deteriorated concrete layer, cleaning and evaluation of the reinforcement and application of an appropriate repair mortar. Repaired slabs in combination with an increased level of the pontoons should provide a durable protection for the rebars and reduce the contact with chlorides. The total budget required is estimated on 1 936 101.77 €.

The proposed solution should not be considered as the only possibility. A promising technique is the use of floating pontoons being not susceptible for the rising sea level. Stability and safety are the issues to be dealt with. This was not investigated as it does not allow the reuse of the current pontoons.

Table of images

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Appendix

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