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RESHEALIENCE

Rethinking coastal defence and Green-energy Service infrastructures through enHancEd-durAbiLity high-performance cement-based materials

RESHEALIENCE
Ultra High Durability Concrete

D3.1 – Definition and description of the scenarios for WP8 pilots

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Publishable summary and Introduction

This deliverable includes a detailed description of the different scenarios in which the pilots of Reshealience project will be built and operating. This description is addressed from geophysical point of view, taking into account weather conditions in case of outdoor pilots and chemical characteristics of the fluids that will contain, in the case of indoor pilots.

The document also contains the compilation of current practice in the field of each one of the involved partners, together with pathologies that can be associated with those current practices. This represents the starting point to define the requirements and characteristics of the UHDC that is going to be applied in the project, in order to improve durability and avoid the aforementioned problems in the service life of the intended types of structures.

Finally, a pre-design of each one of the pilots of the project including main performance demands is detailed, including a general description, main sections, dimensions and concrete characteristics with manufacturing conditions.

Table of contents

List of acronyms, abbreviations and definitions

Table 0.1 Acronym table

XS	Exposure class with risk of chloride induced corrosion
XA	Exposure class with risk of chemical attack
UHPC	Ultra High Performance Concrete
w/c	Water to cement ratio in concrete
w/b	Water to binder ratio in concrete

Table 0.2 Index of tables

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Table 0.3 Index of figures

1. Scope of the document

The goal of the document is to provide a detailed description of the scenarios in which the pilots of the project are going to be developed in order to establish current practices in those situations related to type of structures, concrete characteristics, evolution of the concrete properties with time and analysis the most common pathologies which can occur along the structure service life.

In order to compose this complex framework, an important collection and analysis of data have been carried out. The compiled data come not only from the literature but also, if not mainly, from industrial partners own experience. The final goal has been to find out which are the key durability factors depending on the application, which is determined at the same time by environment and type of structure.

Moreover, besides current practice data related to marine and industrial environments, several case studies have been analysed in order to investigate the main factors that contribute to pathologies appearance in each situation.

2. Description of environment for pilot scenarios

The specific conditions of marine and industrial scenarios for the pilots of the project are going to be described in this section in order to provide the necessary inputs for the development of WP4 and WP5.

2.1. Marine scenarios

Concrete structures have been demonstrated to be able to support the actions of marine environment for the last hundred years, but the specific conditions of their location play a fundamental role in the durability. For that reason, the marine scenarios for the pilots of Reshealience project are described below in detail.

2.1.1. Mediterranean Sea (offshore floating platform, mussel raft)

The pilots of RDC and CMW are going to be placed offshore but near to Valencia coast. In the case of the UoM pilot the structure to be refurbished is onshore but with evident coast climate conditions of the Malta island. In the following sections the weather conditions for Mediterranean locations are going to be detailed. Data have been obtained from the website "Puertos del Estado" (www.puertos.es/es-es/oceanografia/Paginas/portus.aspx 2018/04/10), institution for the management of all the Port Authorities in Spain.

Sea water salinity

Seawater contains many dissolved salts, some of which affect the durability of concrete. The salts present in sufficient quantities, in most of the seas, are:

- sodium chloride ($NaCl$)
- magnesium chloride ($MgCl_2$)
- magnesium sulphate ($MgSO_4$)
- calcium sulphate ($CaSO_4$)
- potassium chloride (KCl)
- potassium sulphate (K_2SO_4)

The concentrations vary from one sea to another, although the total amount of salt is usually around 35 g/l. An exception is the Baltic sea, which contains only one fifth of this amount of dissolved salts.

It should be mentioned that seawater also contains dissolved oxygen and carbon dioxide. The amount of these gases can vary greatly depending on local conditions. Depending on the dissolved amount of carbon dioxide (CO_2) and hydrogen sulphide (H_2S/ hydrogen sulphide in aqueous solution) the seawater pH can decrease from the normal values of 8.2-8.4 to 7 or less. This "acidification" of sea water reduces the alkalinity and strength of concrete and increases the likeliness of electrochemical corrosion of the steel employed as reinforcement of concrete structures.

Table 2.1 gives the typical salt concentrations (including most common salts, mainly NaCl) in several seas and oceans worldwide.

Table 2.1. Salt concentrations in different seas and oceans

Baltic Sea	3-8
Black Sea	18,3-22,2
White Sea	26-29,7
Atlantic Ocean	33,5-37,4
Pacific Ocean	34,5-36,7
Mediterranean Sea	38,4-41,2
Red Sea	50,8-58,5
Ontario Lake	72
Caspian Sea	126,7-185
Dead Sea	192,2-260
Elton Lake	265

As from Table 2.1 it can be observed that the salt concentration in Mediterranean Sea is slightly higher than the average concentration salt in sea water, set as above equal to 35 g/l.

Specific data of water salinity in Valencia coast are provided, with reference to the latest years. The salinity of water (psu, practical salinity units or g/Kg or g/l) for the period 2013-2017 (monthly average) on Valencia coast is in the range of normal values for Mediterranean Sea, are shown in Figure 2.1

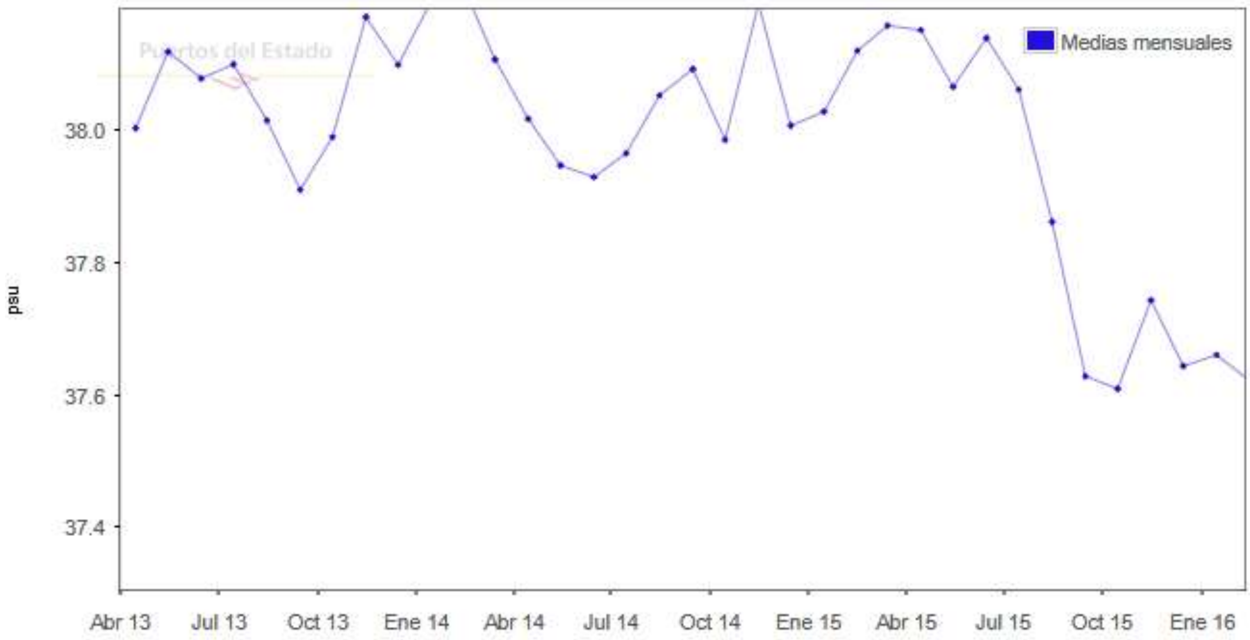


Figure 2.1. Sea water salinity (monthly average) on Valencia coast

Although one of the main causes of corrosion is the chloride penetration, the sulphate content of seawater is also a parameter to be taken into account from the durability point of view.

Average SO_4^{2-} content in sea water is 2650 mg/L. The value is higher in Mediterranean Sea, around 2950 mg/L.

Sea water temperature

The temperature of the sea surface varies from -2°C in very cold areas to 30°C in tropical zones. Seawater temperature varies with depth and at depths between 100 and 1.000 meters the temperature is usually between 2 to 5°C. The temperature of the water is a determining factor of the chemical and electrochemical reactions in the concrete structures. In general, hot or very hot climates accelerate the processes of initiation and progression of the concrete deterioration mechanisms. The temperature of the air to which the marine structure will be exposed must also be taken into account, since the air temperature gradient generating thermal stresses is much higher than the seawater temperature gradient.

It is worth reminding that that in the Persian Gulf 50°C in the shade can be reached while in the Arctic, oppositely, -50°C can occur. These extremely low temperatures may contribute to fragile breakdown of steel under object impact on the surface (iceberg or vessel for example)

This implies that in the tidal zone, part of the marine structure can undergo combined heating and cooling cycles, freezing (ice) and thawing (defrosting) as well as wet and dry cycles. The synergistic effects of these cyclic phenomena must be taken into account when designing the structure from durability point of view.

Historical data of sea water temperature (°C, monthly average) in the Mediterranean Sea, specifically along Valencia coast, for the period 2013-2017 are shown in the following Figure 2.2 to describe the scenario conditions in which the pilots of RDC and CMW are going to be placed.

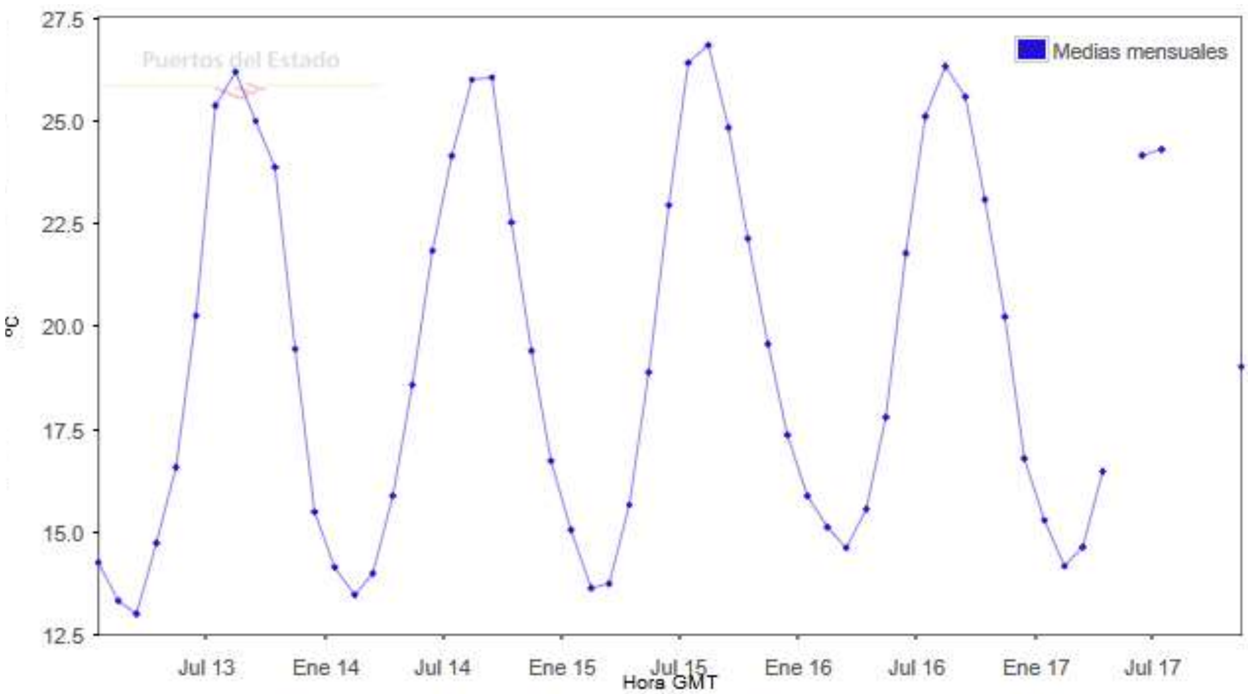


Figure 2.2. Water temperature (monthly average) on Valencia coast

Air temperature
During last year, 2017, the maximum temperature registered in Valencia was 37,1ºC and the minimum 1,4ºC. The accumulated precipitation during 2017 was 2.961 l/m^2 and the maximum precipitation registered in one day was 67,61 l/m^2.

The historical data from 2013 to 2017 of maximum (red) and minimum (blue) temperature are shown in the following Figure 2.3:

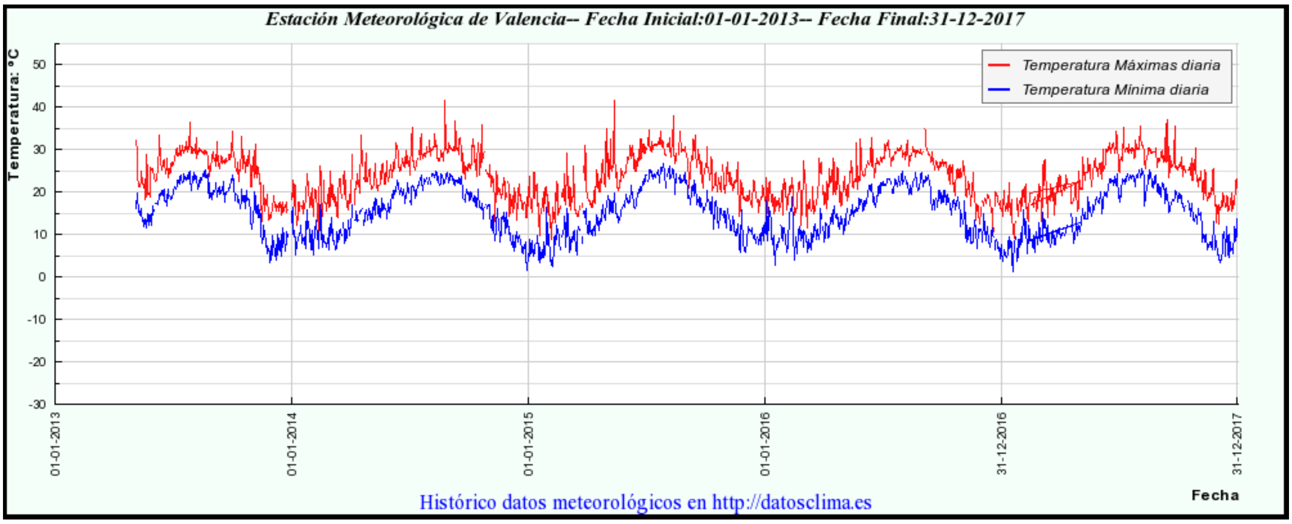


Figure 2.3. Temperature (monthly average for maximum and minimum) on Valencia coast

For the same period the atmospheric pressure (monthly average in mbar) is shown in Figure 2.4.

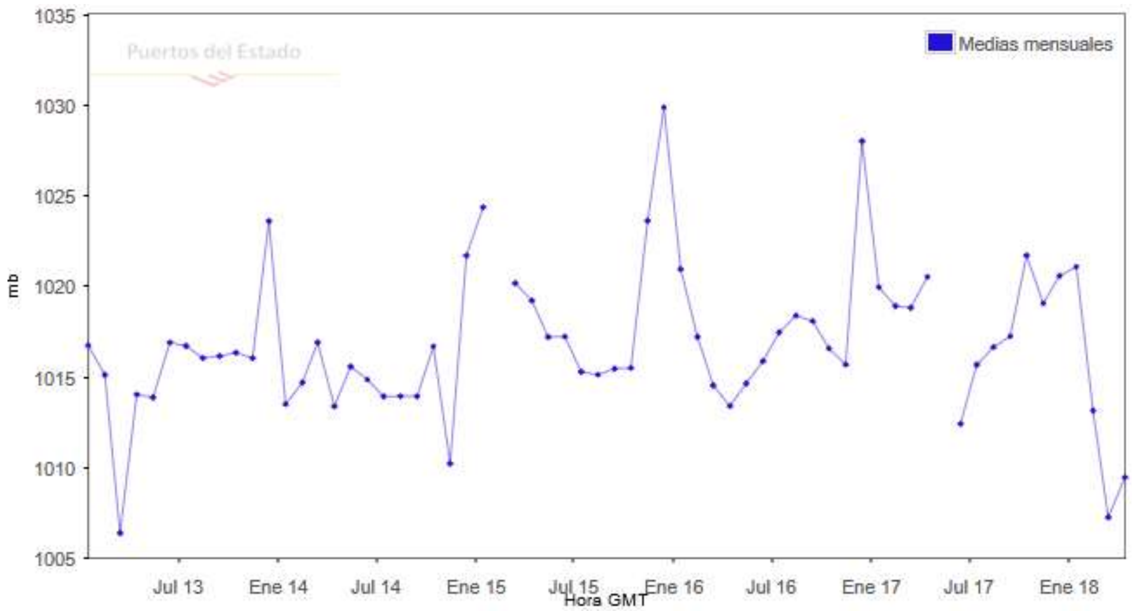


Figure 2.4. Atmospheric pressure (monthly average) on Valencia coast.

Precipitation and Wind

The historical data from 2013 to 2017 of precipitations along the year is shown in Figure 2.5.

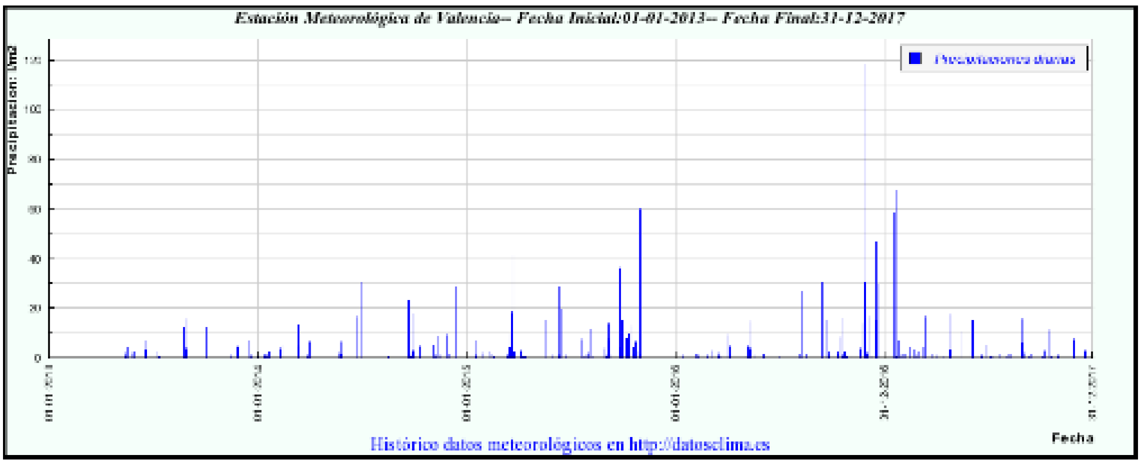


Figure 2.5. Daily precipitation from 2013 to 2017 on Valencia coast (l/m²).

The monthly average of wind velocity in Valencia coast for the period 2013-2017 is shown in Figure 2.6.

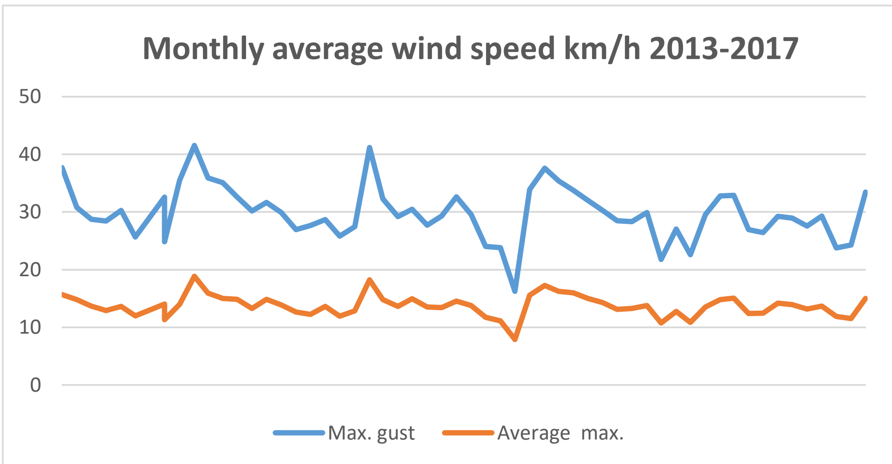


Figure 2.6. Wind velocity (km/h, monthly average) on Valencia coast for the period 2013-2017

Tides and waves

The marine structures are exposed to the action of the astronomical tides generated by the interaction between the Earth and the Moon. This phenomenon causes twice a day an exposure to wet and dry as well as heat-cold cycles and in cold climates, to the action of freezing and thawing as well.

The range of the tide can vary from 0,5 to 15 meters on average in the Atlantic and Pacific Oceans. In deep zones, the range of variation is less than 1 meter and 4-5 meters on the coast.

In addition to the astronomical tide, storms/low pressures can generate significant increases in sea level that in special situations can reach 4-5 meters in The Netherlands (Europe) and 4-7 meters with hurricanes and typhoons.

These situations of deep storms/low pressures are also associated with important storms with large series of waves generated by the friction of the wind with water. The amount of energy that a large storm can transmit to a marine structure is of enormous magnitude.

The pressure of a large wave can reach 300 KN/m^2 on the part of the structure on which it directly impacts. The part of the structure that is subject to the intense action of the waves is called the splash zone and can be attacked by erosion by any solid in suspension carried by the waves such as sand, gravel, ice, wood, etc.

Height of waves (m) registered on Valencia coast for the period 2013-2017 is shown in Figure 2.7 (blue-monthly average, red-monthly maximum, orange-monthly minimum).

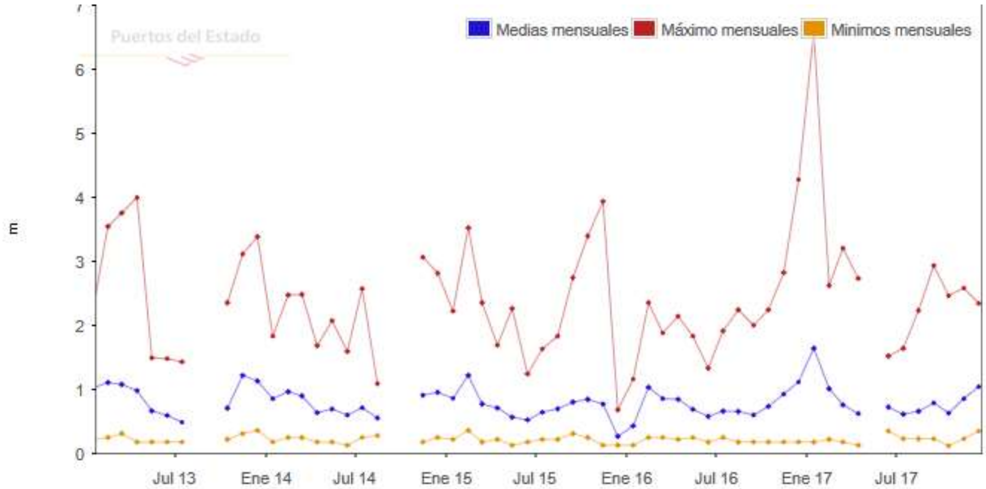


Figure 2.7. Significant wave height (monthly average) on Valencia coast

The tide range or sea level (m) in Valencia coast for the period 2013-2017 is shown in Figure 2.8 (blue-monthly average, red-monthly maximum, light blue-monthly minimum).

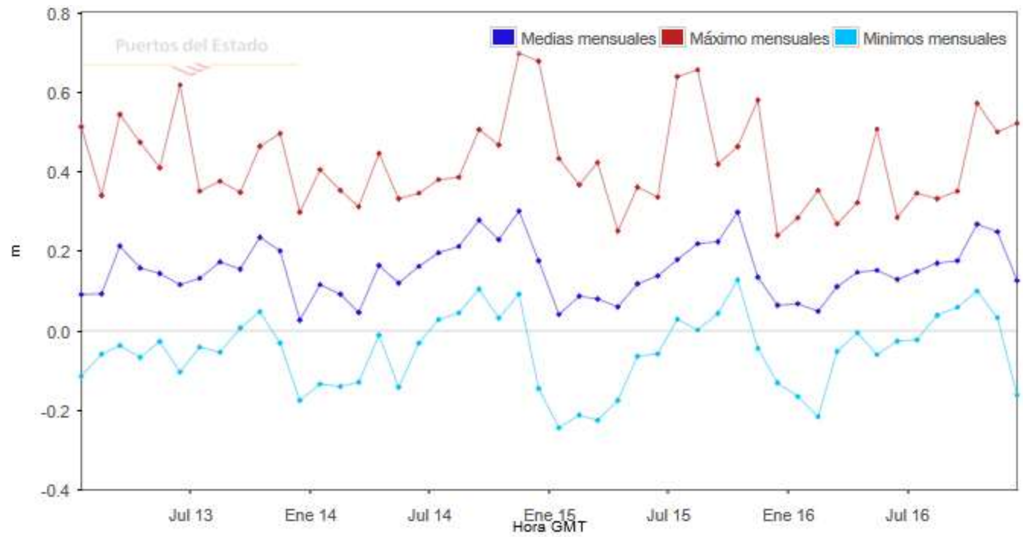
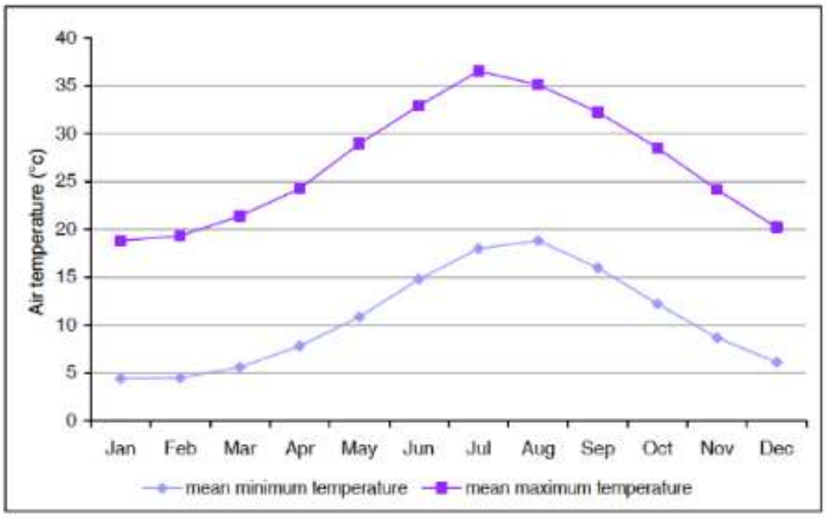


Figure 2.8. Sea level (monthly average) on Valencia coast

2.1.2. Mediterranean Coast (water tower)

Air temperature

Figure 2.9 shows the monthly variation of the average minimum and maximum air temperature in Malta, as recorded at the airport meteorological station. July has the highest mean maximum temperature, while, on average, the lowest winter temperatures occur in February. The overall variation in temperature is to a large extent due to the regional weather patterns in the Central Mediterranean, and the influence from the surrounding sea, which has a warming influence in winter and a cooling influence during the summer period.

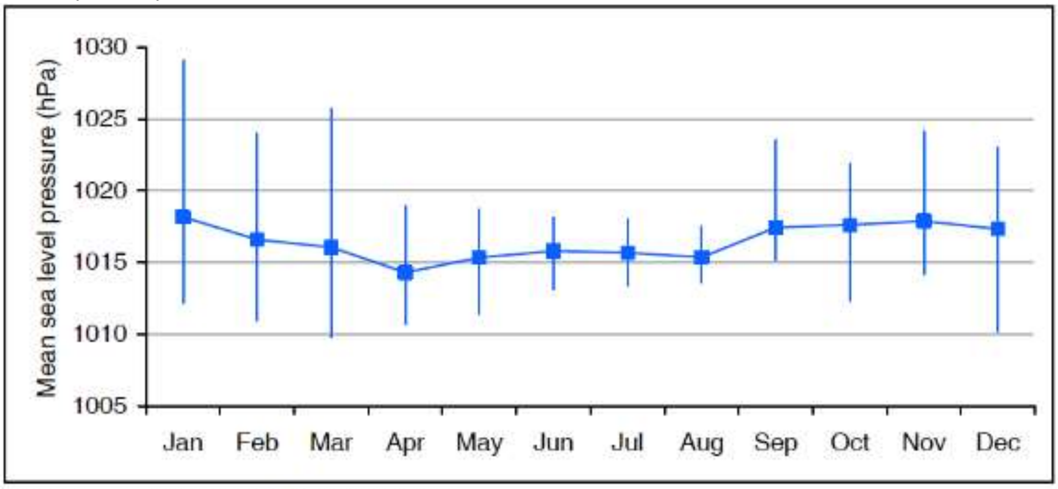


Data collected by the Malta Airport MetOffice

Figure 2.9. Mean min and maxair temperature (based on the 30-year climate period 1961-1990)

(Ref: Galdies, C. (2011). The Climate of Malta: statistics, trends and analysis.)

Atmospheric pressure



Data collected by the Malta Airport MetOffice

Figure ***2.10*** shows the mean monthly sea level pressure and its variability during the period 1961-1990. The yearly mean sea level pressure over the Maltese Islands is 1,016.5 hPa. The lowest air pressure values were observed during March and December, while the highest value of 1,029.1 hPa was reached during January. The highest variation is observed during January. It is interesting to note that June, July and August showed very similar averages and variability in the sea level pressure, which is indicative of the stable weather conditions during the summer months.

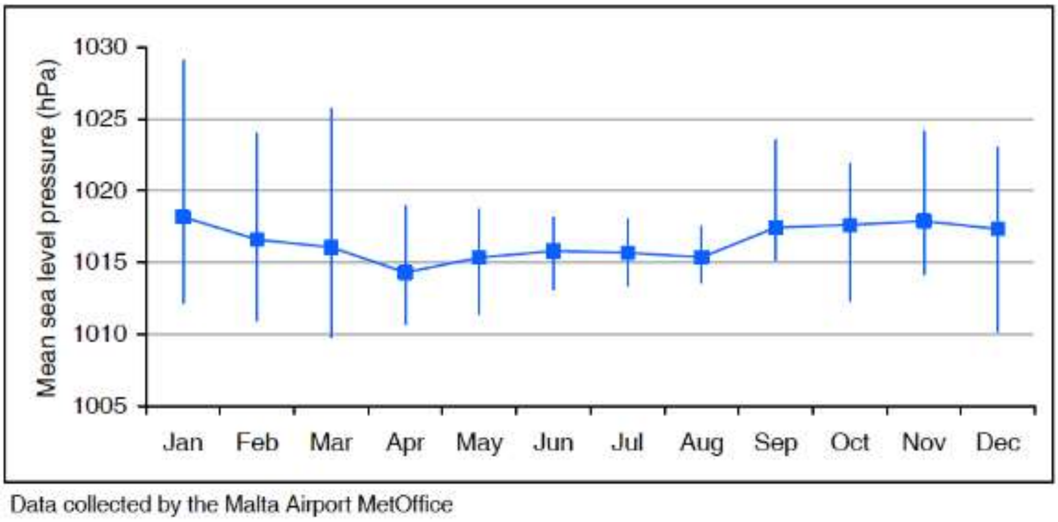


Figure 2.10. Monthly means and variability of the sea level pressure (Based on the 30-year climate period 1961-1990)

Ref: Galdies, C. (2011). The Climate of Malta: statistics, trends and analysis.

Wind Velocity

Figure 2.11 shows the averages of the mean monthly wind speed as measured from Luqa Airport during the period 1961-1990. During this climate period, April gave the highest mean monthly wind speed of 10.3 knots (or 19.1 km/hr). January gave both the highest *extreme* mean monthly wind speed of 14.1 knots, or 26.1 km/hr and the highest variability in the mean monthly wind speed.

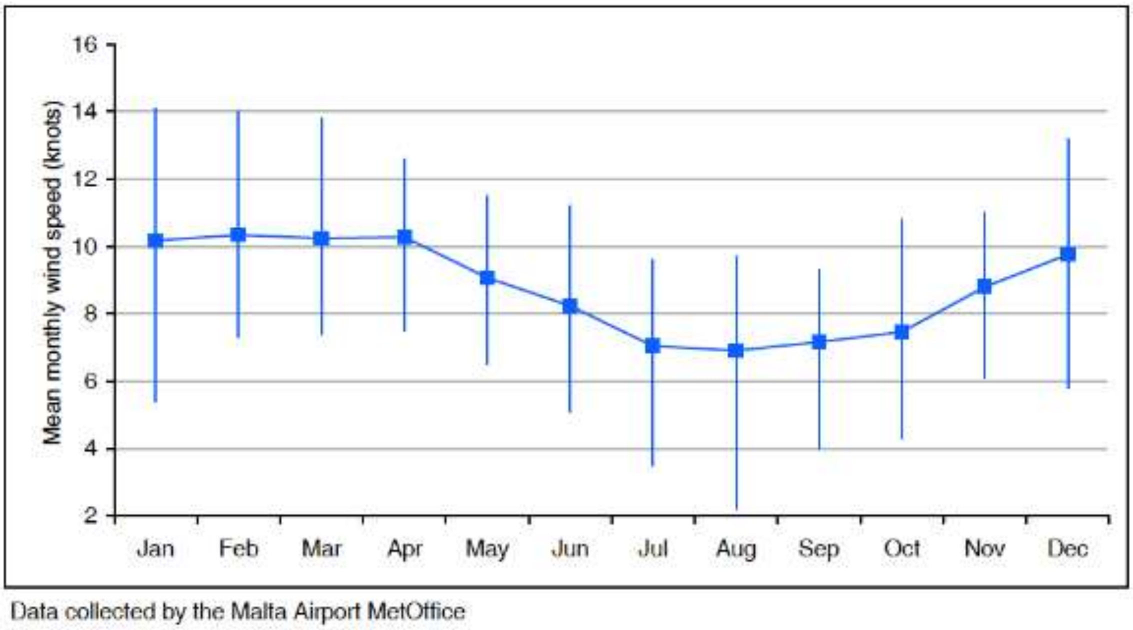
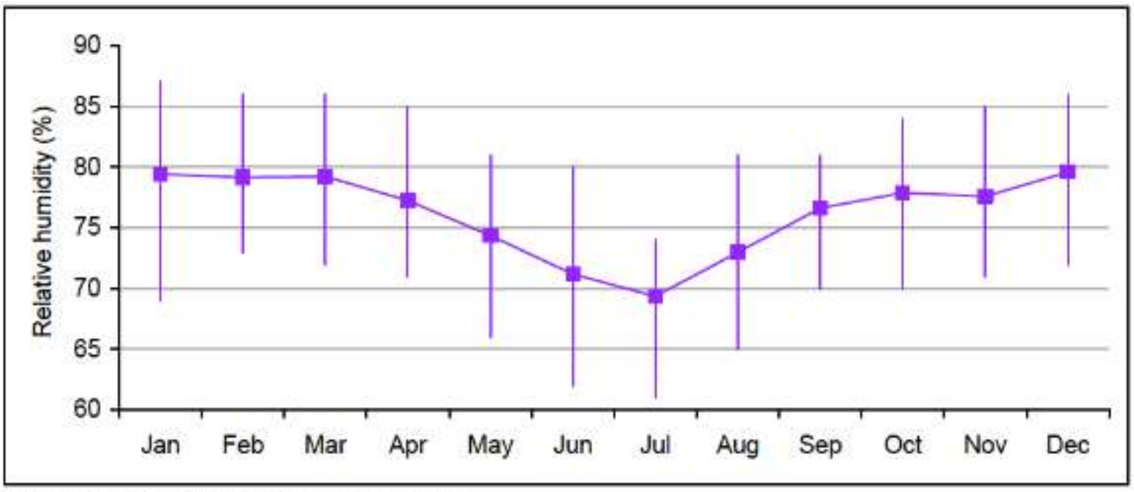


Figure 2.11. Figure Monthly means and variability of the wind speed (Based on the 30-year climate period 1961-1990)

Ref: Galdies, C. (2011). The Climate of Malta: statistics, trends and analysis.

Relative humidity

Figure 2.12 shows the variability of the relative atmospheric humidity over the Maltese Islands during the period 1961-1990. It shows that the mean relative humidity varied from a minimum of 61 per cent in July to a maximum of 87 per cent in January. The transition between July and August shows the highest gradient in both the mean and maximum relative humidity. The highest monthly variability occurred during January and June.



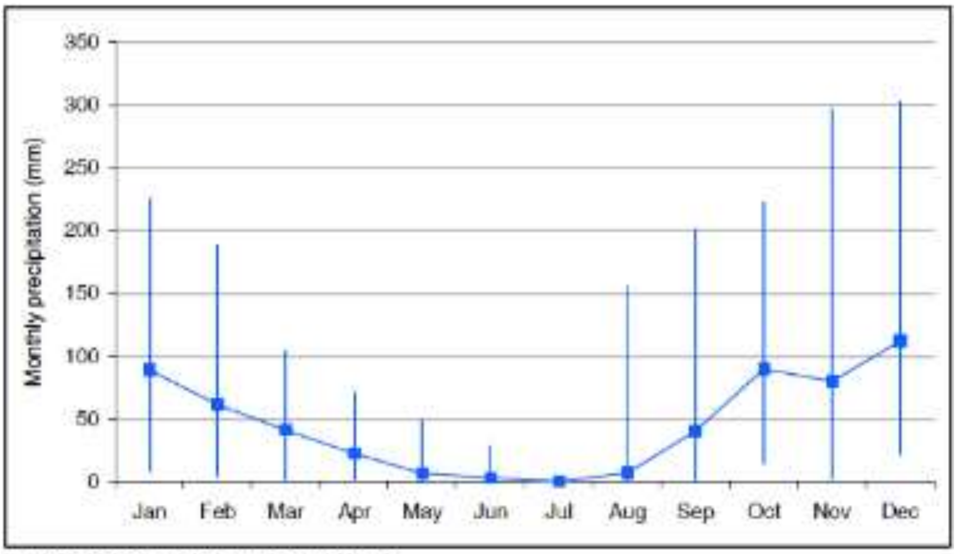
Data collected by the Malta Airport MetOffice

Figure 2.12. Monthly means and variability of the relative humidity (Based on the 30-year climate period 1961-1990)

Ref: Galdies, C. (2011). The Climate of Malta: statistics, trends and analysis.

Daily precipitation

Observations show an annual pattern of rainy winters followed by dry and generally rainless summers (Figure 2.13). Over half the total annual precipitation has been recorded between October and December. December is the month with the highest precipitation, amounting to 20.3% of the total annual precipitation. The summer period between June and August barely comprises 2% of the total rainfall. Meanwhile, November gives the highest precipitation variability throughout the year, ranging from a minimum of 2.6 mm to a maximum of 297.0 mm.



Data collected by the Malta Airport MetOffice

Figure 2.13. Monthly means and variability of the total precipitation (Based on the 30-year climate period 1961-1990)

Ref: Galdies, C. (2011). The Climate of Malta: statistics, trends and analysis.

The total amount of precipitation recorded in 24 hours is a good indicator of the intensity and duration of storms. The climatological trend of the total amount of precipitation recorded in 24 hours is shown in Figure 2.14. Clearly, November shows the greatest variability, which is attributed to convective storms triggered by the movement of the continental air mass from the North African region over cooler areas in the Central Mediterranean.

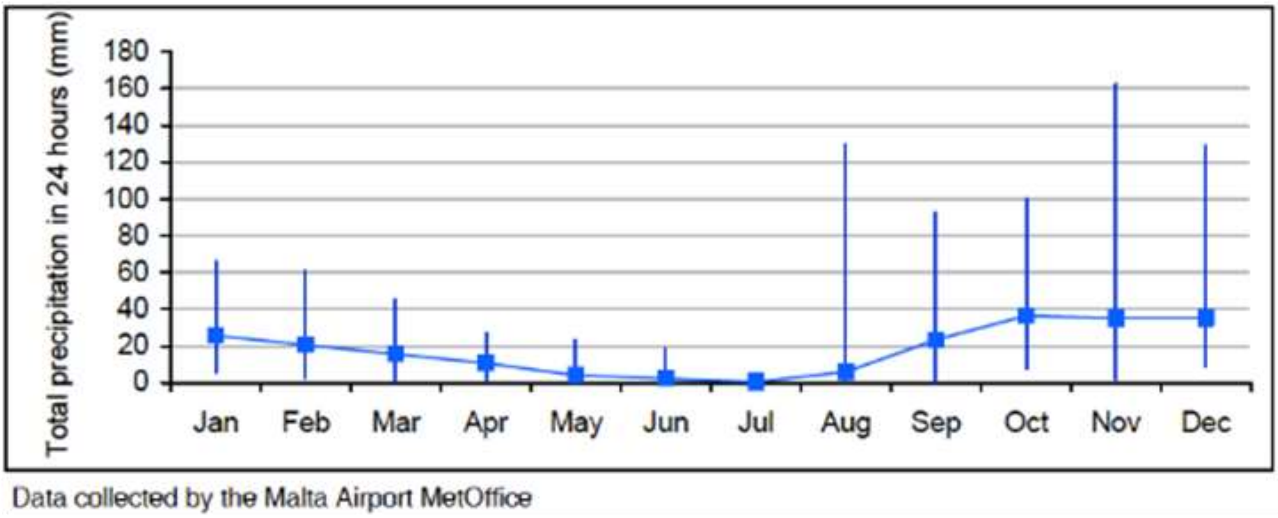


Figure 2.14. Monthly means and variability of the total precipitation recorded in 24 hours (Based on the 30-year climate period 1961-1990)

Ref: Galdies, C. (2011). The Climate of Malta: statistics, trends and analysis.

2.1.3. *Atlantic Coast (floating jetty)*

The pilot of Banagher Precast Concrete Ltd. will be a floating jetty located on the West Coast of Ireland Atlantic Ocean), most likely in Galway, whose specific weather and sea conditions are described in the following sections.

As expected, the conditions along the Ireland west coast are more severe than in the Mediterranean sea, with reference to specific features such as wind, waves and tides, phenomena that cause significant erosion on structures.

Water salinity

Water salinity of Atlantic Ocean is on the average of oceans and seas salinity. In Ireland the general pattern of salinity distribution, derived from long term datasets, shows that in wintertime near shore surface salinities to the west are on average 35 psu or greater. In summertime this drops slightly to a range of 34,5-35,0 psu around the Atlantic and Celtic Sea coasts. Surface salinities increase steadily towards the open ocean, reaching values of 35,4-35,5 PSU at the shelf edge.

In general, salinity increases with water depth. Around Ireland vertical salinity gradients may be seasonally and locally pronounced in near-shore areas and in early summer when warmer stratified water overlies cooler, mixed Atlantic water. However, during the winter months off-shore waters to the west and south of Ireland are generally well mixed from top to bottom with a consequent smaller variation in salinity.

Average SO_4^{2-} content in sea water is 2650 mg/L. The value is lower in Atlantic Ocean where it is around 2360 mg/L.

Water temperature

Average sea surface temperatures to the west and south of Ireland range from 8-10°C in February-March to 14-17°C in August. To the northeast temperatures are, on average, a couple of degrees colder throughout the year. Water temperatures are about 7-8°C warmer around Ireland than the global average at equivalent latitudes. This is due to the North-Atlantic drift which transports warmer waters from the bay of Mexico.

Air temperature

The data available are the monthly average for the period 2013-2017 for minimum, mean and maximum temperature (°C) as shown in the following graph in Figure 2.15. The West Coast reference station that has been chosen is Galway.

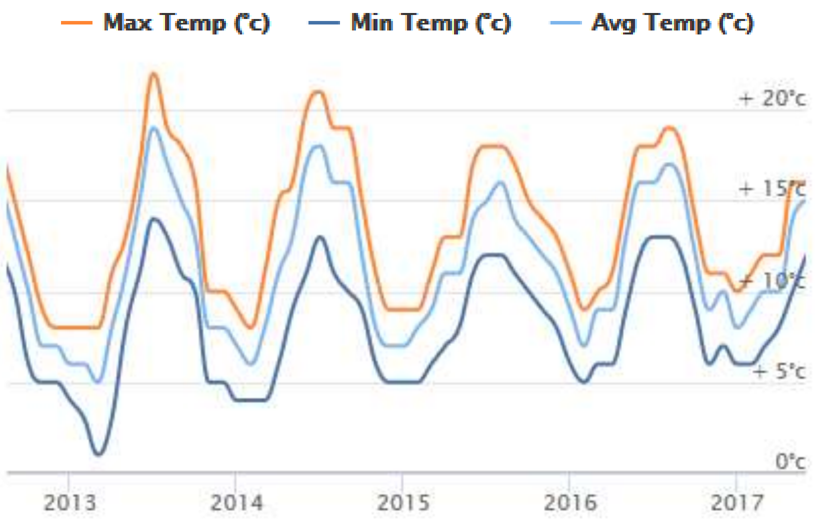


Figure 2.15. Temperature (monthly average) on Irish West Coast

For the same period the atmospheric pressure is shown in Figure 2.16.

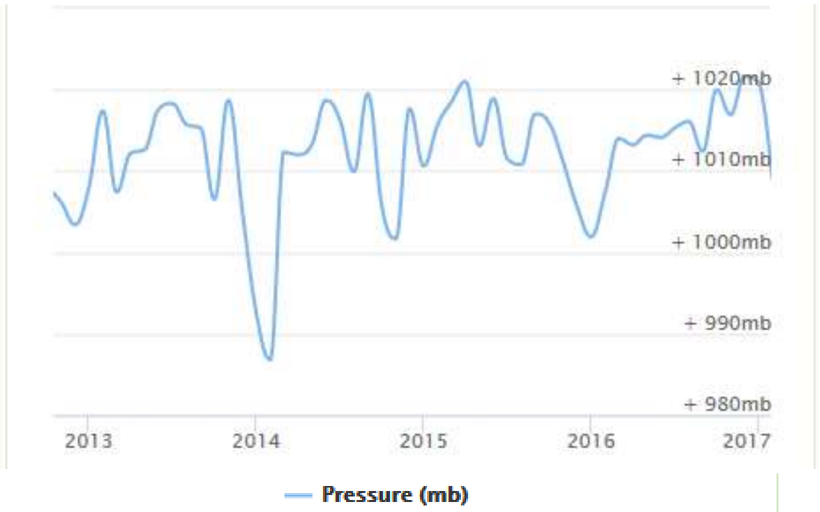


Figure 2.16. Pressure (monthly average) on Irish West Coast

Precipitation and Wind

The rainfall amount in mm and rainy days for the period 2013-2017 in Galway are shown in Figure 2.17

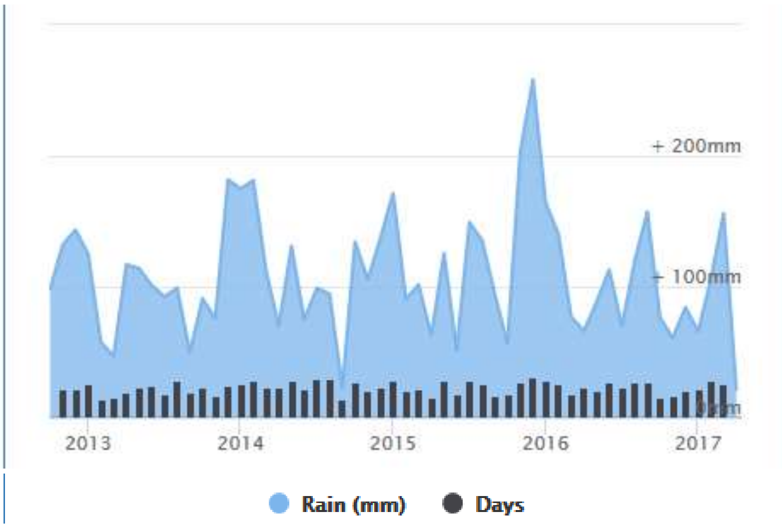


Figure 2.17. Precipitation and rainy days on Irish West Coast

The monthly average and maximum of wind velocity on Irish West Coast (station offshore) for the period 2013-2017 is shown in Figure 2.18 (blue-monthly average, red-monthly maximum).

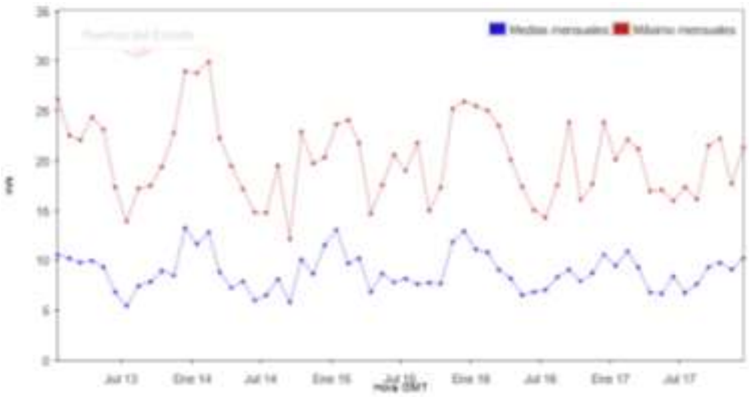


Figure 2.18. Wind speed (monthly average) on Ireland West Coast

Waves and tides

Significant height of waves (m) registered on Irish West coast for the period 2013-2017 is shown in Figure 2.19 (monthly average).

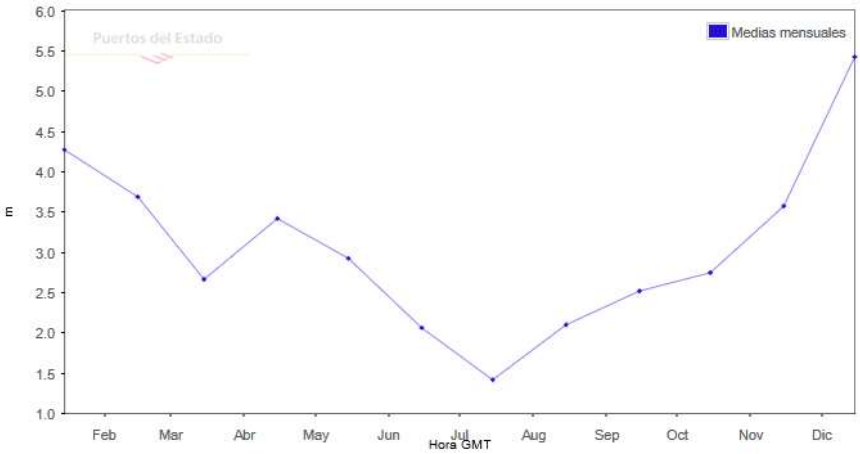


Figure 2.19. Significant wave height (max. monthly average) on Ireland West Coast

The sea level variation on Irish West coast (Galway) for the period January-April 2018 is shown in Figure 2.20. The data have been registered under continuous measurement and it can be highlighted that the variation achieves differences of 8 m while on the Mediterranean coast maximum variation was around 1 m.

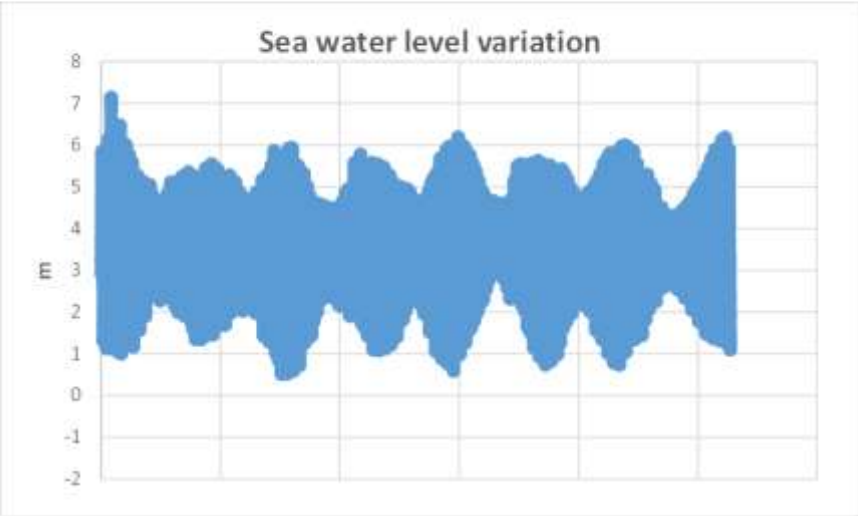


Figure 2.20. Sea level variation on Ireland West Coast

2.2. Industrial scenarios

2.2.1. Industrial environment (water and mud basin)

For the pilot of the project exposed to industrial environment, the main scenario is a cooling tower water collection tank in a geothermal plants belonging to Enel Green Power fleet.

The water inside the tank is not still but in motion, because it is recirculated from the top of the cooling tower to be cooled down, and to feed the plant condenser, where the steam coming from the turbine is collected and condensed. So, even if the speed is not high, the water is not steady. Moreover, another interesting element is that in order to increase the heat exchange, the water falls from the upper part of the tower (10-12m) into the basin with a non-negligible mechanical effect on the basin walls and bottom.

Table 2.2. Exposure conditions for EGC pilot in XA environment

Plant	Location	Date	Typology (Kind of water)	T	pH	Cond	Alk	Alk rit	O_2 dissolved	H_2S	SO_4^{2-}	SO_3^{2-}	$S_2O_3^{2-}$	Cl^-	Na^+
				°C	-log [H^+]	µS/cm	meq/L HCl	meq/L HCl	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
Nuova San Martino	LAGO	06-mar-18	Blown down water	n.d.	7,07	4.430	3	1,8	0,1	0,4	1.223	77	26	213	n.d.
Le Prata	LAGO	12-gen-17	column C2 water inlet	19,3	7,08	6.460	11	n.d.	n.d.	0,2	2.292	980	229	1,1	1.352
Valle Secolo Gr.1	LARDERELLO	27-mar-17	column C2 water inlet	16	7,99	1.060	4,9	n.d.	n.d.	u.d.l.	202	u.d.l.	u.d.l.	2,6	7
Bagnore 4	PIANCASTAGNAIO	26-gen-17	Injection water	19,9	6,49	24.200	3,1	n.d.	1,1	0,3	10.020	505	11,2	2,8	0,9
Chiusdino	RADICONDOLI	07-lug-17	cooling tower water inlet	n.d.	6,9	7.870	n.d.	n.d.	n.d.	1,5	5.119	325	46	1	n.d.

There are different kinds of water in the different parts of the plant: as a matter of fact the water is the geothermal fluid itself, which is extracted from the production wells, works in the geothermal plant and then is reinjected, after being cooled down in the cooling towers, in the reinjection wells. The most useful for the scope is the water inside the basin, of typology blown down water, injection water or cooling tower water inlet. Although the pH values are around 7, the content of different salts is very high as well as the conductivity.

The fluid composition is very much site dependent, coming from different geothermal reservoirs.

The typical dimensions of a cooling tower water collection tank are about 35,0-40,0m x 14,0-18,0m in plan, while the walls are usually 2,0-2,5m high (from the bottom of the basin) and 200 mm thick; so the volume is about 700-1.500 m^3, considering that on each basin 3 cooling tower cells are usually installed (3 fans, being forced draft cooling towers).

3. Current practice in concrete applications

3.1. Current practice in marine environments

In this section of the document the current practice related to concrete structures in marine environment are analysed as well as the evolution of the concrete along time and the appearance of different durability pathologies and possible reasons.

The exposure to marine environment implies the possibility of corrosion of the reinforcement induced by chlorides from sea water.

Structures located in water can be subjected to rapid deterioration of their in-water elements such as piles and docks. Freezing and thawing effects, sulphate attack, and corrosion of steel reinforcement (particularly in salt water) are representative of marine deterioration types whose extent and rate of progress are affected by local environmental factors. The local combined effects of wind, temperature, and water generally control the deterioration rate of any given structure. Good design, construction and maintenance can mitigate the extent and rate of deterioration.

The most rapid deterioration in structures exposed to marine environment occurs in the splash zone due to the ready availability of moisture and oxygen. The tidal zone, or zones of seasonal water level variation, also experience corrosion rates higher than the submerged zones or those permanently in contact only with air. The tidal zones are often critical for concrete and steel construction. Areas fully submerged generally show reduced corrosion due to the lower availability of oxygen.

3.1.1. General Maritime Works

In this section a review about current construction practice in typical marine environments is described, according to the most representative elements or structures related to partners normal activities and field of work.

3.1.1.1. Concrete caissons

Employed materials:

- *Reinforcement Steel*
  Reinforcing steel shall comply with ISO 6935, Parts 2 and 3 or relevant international standards for reinforcing steel. The type of steel reinforcement used will be RB 500 or similar, with the next minimum strength properties:
    - Yield strength: 500 MPa
    - Ultimate strength: 550 MPa

- *Concrete*
  Due to durability considerations, a minimum concrete strength is imposed. According to DNV-OS-C502, Sec. 4. C210 for concrete exposed to sea water (XS2) the minimum grade is C35. According to DNV-OS-J101, Sec. 66. C307, for concrete exposed to seawater the minimum grade is C40. In a conservative way, C40 grade is adopted as a minimum. The strength properties for normal concrete grades according to DNV-OS-C502 are shown in Table 3.1.

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Table 3.1. Normal concrete grades in standard DNV-OS-C502

Concrete grade	C25	C30	C35	C40	C45	C50	C55	C60	C70	C80
f_{ckk} (MPa)	25	30	35	40	45	50	55	60	70	80
f_{cn} (MPa)	24.0	28.5	33.0	37.3	41.6	45.8	50.0	54.0	61.8	69.3
f_{tk} (MPa)	2.4	2.63	2.84	3.04	3.22	3.39	3.56	3.72	4.02	4.29
f_{tn} (MPa)	1.81	1.95	2.07	2.18	2.28	2.37	2.45	2.53	2.68	2.80

3.1.1.2. Offshore concrete platforms (general review about concrete offshore platforms)

General considerations

The majority of concrete offshore platforms have employed high-strength concrete highly reinforced and pre-stressed. Concrete Classes of Resistance from C40 to C85 are usually specified in these complex structures. These types of structures have pushed the technology of concrete far beyond the limits of standard design practices. The necessary provision for dealing with confinement of high-strength concrete is not always present in codes of practice. Some of these codes tend to be more conservative than necessary and others overestimate the strength capacity of key structural members. For this reason, some research is still needed in this area despite the considerable number of proposed models to deal with confinement and ductility enhancement.

Employed materials

The high-strength required in *offshore* concrete structures depends on a certain number of factors such as the use of high-strength aggregates. The use of additional binder materials like active silica, fly-ash and slag together with traditional Portland cement is very important to improve the concrete strength and the durability. These additions allow a reduction of porosity and also an improvement in strength due to a reduction of water consumption. The reduction of porosity is essential to guarantee the durability of the structure. In offshore applications the consumption of binder materials varies from 380 kg/m^3 to 500 kg/m^3, while the water/binder material ratio can be reduced to 0.30.

The use of high-strength concrete brings some difficulties to the construction of *offshore* structures. The need of high reinforcement ratios demands criteria for detailing the reinforcement bars and pre-stressing cables. The reduction of space between the reinforcing bars normally implies the use of reduced maximum aggregate size and the need for enhanced workability. The usual maximum aggregate size is 10 to 14 mm with a slump of 180 to 220mm.

The large amount of binder materials is responsible for high levels of heat associated with the hydration process. Precaution is recommended to avoid early damage to the structure due to the temperature rise (usually reaching 70^{o}C). Sometimes the solution of this problem is the production of concrete at low temperatures (5-7^{o}C). This is more difficult and expensive in regions with warmer climates.

In the last 15 years, a lot of research has been carried out on structural light weight aggregate (LWA) concrete for offshore platforms. The goal has been to produce a concrete with the highest possible strength and minimum weight for offshore platforms. Such properties possess a large potential in optimizing design and improving competitiveness of concrete structures. Companies such as Franken Leichtbaustoffe GmbH (Germany), A/S Norsk Leca (Norway) Texas Industry Inc. (USA) Solite Corporation Inc. (USA) or Boral Lytag (Great Britain) provide LWA validated for offshore concrete works. Currently, LWA-concrete with high strength and densities of 1900-2000 kg/m^3 may be produced in large scale on site (compared to Normal Density concretes C75-C85 with density of 2450 kg/m^3). The reduced weight is achieved by replacing the natural coarse aggregate with lightweight materials made of expanded shale or clay.

Currently, LWA-concrete with high strength and densities of 1.900-2.000 kg/m^3 may be produced in large scale on site (compared to Normal Density concretes C75-C85 with density of 2.450 kg/m^3). The reduced weight is achieved by replacing the natural coarse aggregate with lightweight materials made of expanded shale or clay. For offshore projects, a limited number of LWA sources (suppliers) have been tested and evaluated so far. The *offshore* concrete platform industry has been a driving force in the development of high quality concretes and modern construction methods, as also witnessed by the following facts:

- Off-shore concrete platform structures have shown an excellent performance in operation. No significant evidences of material related deficiencies have been observed.
- The technological development of *offshore* concrete platforms will certainly have a future application for demanding *offshore* conditions.
- Concrete structures for offshore applications can be built almost anywhere, as long as ships and barges can access the site. In some cases, however, the design will depend on water-depth limitations.
- In order to achieve the concrete high strengths necessary for *offshore* applications, the coarse aggregate must be appropriately selected. Usually basalt and granite aggregates should be employed. Certain types of gneisses are also possible, but limestone and dolomites are usually avoided. Light aggregates, like expanded clay, are also frequently employed for *offshore* applications. The quality of cement is also crucial to achieve the designed characteristics of strength and durability.

Main deterioration in offshore concrete structures.

Table 3.2. Deterioration in offshore concrete structures shows the correlation between the main parts of a concrete offshore structure and the primary degradation mechanisms applicable to these areas. The information comes from a study of petrol platforms in Norway.

Chemical degradation and steel reinforcement corrosion are the factors affecting most parts of the structures.

Table 3.2. Deterioration in offshore concrete structures

Deterioration mechanism	Part of structure						
	Legs / Towers / shafts - general	Splash zone	Topsides	Steel concrete transition	Shaft / base junction	Storage cells	Foundations
Chemical deterioration	√	√			√	√	
Corrosion of steel reinforcement	√	√			√	√	
Corrosion of prestressing tendons	√	√			√	√	
Fatigue			√	√	√		
Ship impact	√	√					
Dropped objects						√	
Bacterial degradation	√	√			√	√	
Thermal effects					√	√	
Loss of pressure control					√	√	
Loss of air gap			√	√			
Scour & Settlement							√

3.1.1.3. Offshore floating caisson (CMW specific project)

Structure concept
Floating offshore concrete caisson. Data described in this section are included in a non executed project of CMW.

Employed materials
Concrete C40 (DNV)

Characteristics:
- Elastic modulus Ecm:33.315 MPa
- Cylinder compressive strength fck: 40 MPa
- mean tensile strength f_{ctk}: 3.04 MPa
- In-situ tensile strength ftn= 2.18 MPa
- w/c = 0.45 (maximum)
- Minimum cover: 40 mm
- Minimum cement content: 360 kg/m^3
- Maximum size aggregates: 22 mm
- Consistency: 18$\pm$3 cm
- Permeability coefficient: 10^{-12} to 10^{-8} m/s

The total amount of chlorides in the fresh concrete, calculated as free calcium chloride, shall not exceed 0.3% of the weight of cement.

Operational issues.
In this project it was established that chloride profiles should be measured in order to establish the rate of chloride ingress through the concrete cover. Either total chloride ion content or water soluble chloride content should be measured. However, the method chosen should be consistent throughout the life of the structure. These profiles can be used for estimating the time to initiation of reinforcement corrosion attack in the structure. Periodic examination with measurements shall be carried out to verify that the cathodic protection system is functioning within its design parameters and to establish the extent of material depletion.

As far as catjodic protection is utilized for the protection of steel reinforcement crucial to the structural integrity of the concrete, the sustained adequate potential shall be monitored. Examination shall be concentrated in areas with high or cyclic stress regimes, which need to be monitored and checked against the design basis. Heavy unexpected usage of anodes should be investigated.

3.1.2. Precast elements for coastal maritime and offshore structures

The current practice related to mix design of Banagher in the manufacturing of precast elements is summarized in the following tables 3.3 and 3.4 for pre-stressed and not pre-stressed reinforcement.

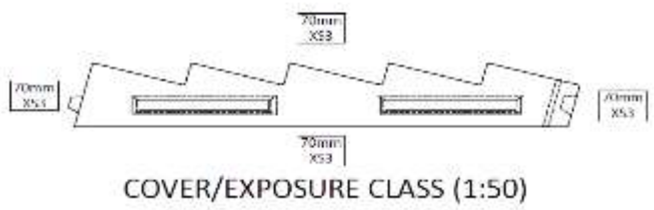
Table 3.3. Precast stressed beams characteristics

C50/60 - 250 / Cem IIIA – 36% GGBS - Stressed MY Beams				
MATERIAL DESCRIPTION	**MASS PER m^3**		**UNITS**	**MATERIAL SUPPLIER / COMMENTS**
ement Type – Rapid Hardening Portland Cement CEM I	288	CEM IIIA	Kg's	Irish Cement.
GGBS	162		Kg's	Irish Cement.
Betocarb Lime	100		Kg's	Omya Ltd.
20mm Aggregate	0		Kg's	Banagher Precast Concrete Ltd.
14mm Aggregate	710		Kg's	Banagher Precast Concrete Ltd.
8mm Aggregate	0		Kg's	Banagher Precast Concrete Ltd.
Sand	920		Kg's	Banagher Precast Concrete Ltd.
Crushed Limestone Fines	0		Kg's	Banagher Precast Concrete Ltd.
Max Free Water	175		Ltrs.	
Water/Cement Ratio	0.39		Ratio	
ADMIXTURE DESCRIPTION				
Sika Viscocrete Premier	2.5		Ltrs.	Sika
Sika Rapid 2	-		Ltrs.	Sika
Mastermatrix 233	-		Ltrs.	B.A.S.F
Excel 650	5		Ltrs.	Chryso
Premia 360	-		Ltrs.	Chryso

Table 3.4. Precast element characteristics

C40/50 - 2002 / Cem IIIA – 36% GGBS				
MATERIAL DESCRIPTION	**WEIGHT/ m^3**		**UNITS**	**MATERIAL SUPPLIER / COMMENTS**
ment Type – Rapid Hardening Portland Cement CEM I	275	CEM IIIA	Kg's	Irish Cement.
GGBS	150		Kg's	Irish Cement.
Betocarb Lime	140		Kg's	Omya Ltd.
20mm Aggregate	0		Kg's	Banagher Precast Concrete Ltd.
14mm Aggregate	440		Kg's	Banagher Precast Concrete Ltd.
8mm Aggregate	440		Kg's	Banagher Precast Concrete Ltd.
Sand	750		Kg's	Banagher Precast Concrete Ltd.
Crushed Limestone Fines	0		Kg's	Banagher Precast Concrete Ltd.
Max Free Water	170		Ltrs.	
Water/Cement Ratio	0.4		Ratio	
ADMIXTURE DESCRIPTION				
Sika Viscocrete Premier	-		Ltrs.	Sika
Sika Rapid 2	-		Ltrs.	Sika
Mastermatrix 233	2.7		Ltrs.	B.A.S.F
Excel 650	-		Ltrs.	Chryso
Premia 360	-		Ltrs.	Chryso

For these precast elements the cover is usually 70 mm for exposure class XS3 as shown in Figure 3,1. The minimum binder content is 380 kg/m^3, maximum size of aggregates 20 mm and maximum w/b ratio 0.4.



STRENGTH 28 DAYS	**C40/50**
GGBS CONTENT	**0%**
COVER	**70mm**
WEIGHT	**20.75 T**
VOLUME	**8.37 m^3**

General Notes:
[1] Concrete Grade shall be C40/50 exposure class shall be XS3 to BS8500-1.
[2] Maximum water-cement ratio shall be 0.4, the minimum cement or combination content should be 380kg/m^3 and the maximum size aggregate 20mm.
[3] Nominal cover to be 70mm.
[4] All Steel Reinforcement in the Proposed Precast Concrete Units shall be deformed Stainless Steel Grade 316 to BS 6744 cut and bent to BS 8666.
[5] Concrete Finished are indicated by this Drawing. Exposed formed upper surfaces to have an F3 finish, all unexposed formed surfaces to have an F1 finish. All unformed surfaces shall have a U1 finish.

Figure 3.1. Current practice concrete design for precast elements

3.1.3. Precast UHPC

In this section the current practice of RDC related to UHPC elements is described.

3.1.3.1. Floating platform

Type of element: Floating platform (5 of them, the first one floated in October 2016). Produced by IDIFOR.

Figure 3.2. RDC floating platform

Current structure concept

Floating structure consisting of UHPC beams or frames. The flotation is obtained through six steel floaters.

Current dimensions and details (cover, reinforcement, protection, etc)

Total dimension of 20x27 m. The primary beams are pre-stressed and have a width of 350 mm and a depth of 230 mm; they are lightened, and the wall thickness is equal to 60 mm. The geometric cover is 20 mm. The amount of prestressing steel is 180 kg/m^3 for the primary beams and 256 kg/m^3 for the secondary beams. There is no protection system applied, but in the last platforms a dark red layer to homogenize the surface aspect is going to be included.

Current employed materials

The composition of concrete is the following:

- Cement 42.5 R/SR: 800 kg/m^3
- Silica fume Elkem 940D: 175 kg/m^3
- Quartz flour Sikron US-500: 225 kg/m^3
- Quartz sand 0-1.6 mm: 867 kg/m^3
- Water: 180 liters
- w/b = 0,18
- Admixture Sika Viscocrete 20HE: 30 litros
- Fiber 13/0.2 mm: 160 kg/m^3

Characterization of current employed materials
Self-compacting concrete is employed with an average slump flow diameter equal to 730 mm and average 28-days compressive strength equal to 150 MPa. Porosity to water between 4% and 5%.

The durability parameters are measured according to the tests indicated in the French UHPC Standard. The diffusion coefficient of chloride ions obtained ranges between 0.25 to 0.45 x 10^{-12} m^2/s. The measurement of the oxygen permeability was not possible, as the measuring range of the test is higher than the value of the material (lower than 10^{-18} m^2).

Current application technology and employed casting requirement.
Precast concrete in the precast plant with high level of control. CE marking.

Current experienced operational issues
At this moment there is no pathology observed. The sensors installed by the IDM in three of the rafts indicate a very low risk of corrosion and that the corrosion phenomena has not started.

3.1.3.2. Pedestrian bridge

Type of element
Footbridge in Alicante. Designed by the current members of RDC, produced in PREVALESA.

Figure 3.3. RDC pedestrian bridge

Current structure concept
Pre-stressed UHPC footbridge, 43.5 m of span, 3 meters of useful width.

Current dimensions and details (cover, reinforcement, protection, etc)
The top chord is not pre-stressed, while each bottom chord has 18 strands type Y1860S7.

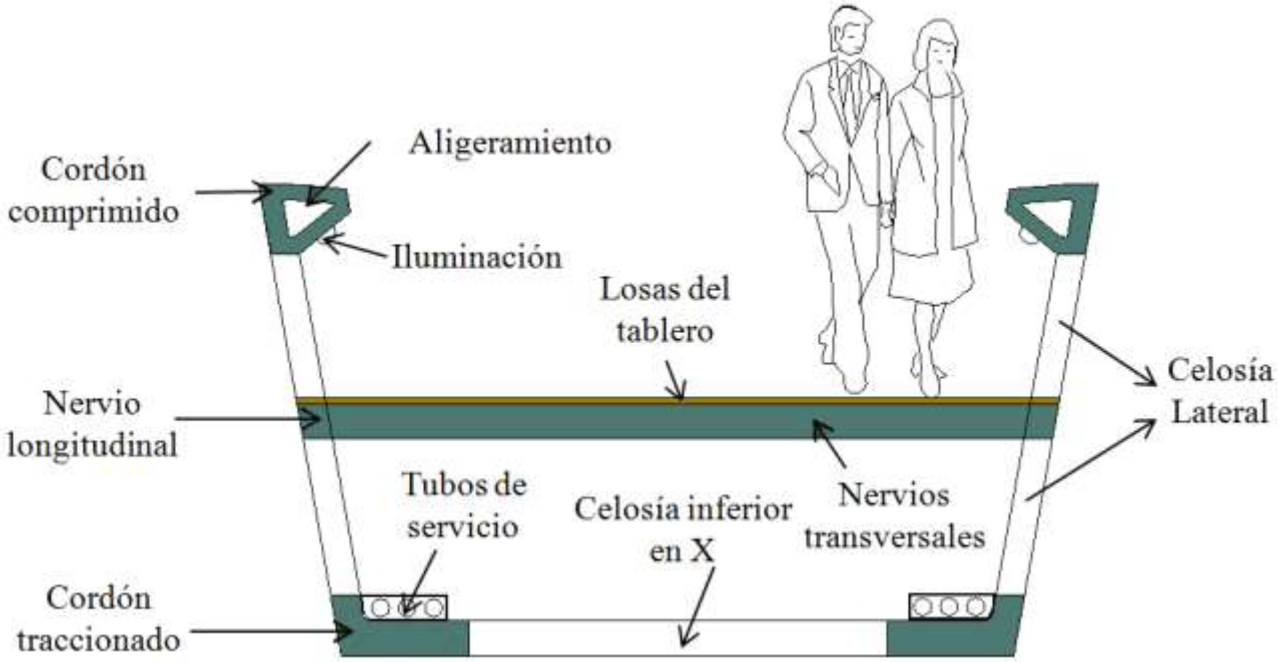


Figure 3.4. RDC pedestrian bridge section

Current employed materials

The composition of concrete is the following:

- Cement 52.5 R/SR: 1000 kg/m^3
- Silica fume Elkem 940D: 175 kg/m^3
- Quartz sand 0-1.6 mm: 892 kg/m^3
- Water: 190 liters
- w/b = 0,162
- Admixture Sika Viscocrete 20HE: 30 l.
- Fiber 13/0.2 mm: 80 kg/m^3
- Fiber 80/30 Dramix, high strength limit: 80 kg/m^3

Characterization of current employed materials

Self-compacting concrete was employed with an average slump flow diameter equal to 730 mm and average compressive strength after 1, 28 and 90 days equal to 80, 135 and 150 MPa respectively.
No durability parameters measured.

Current application technology and employed casting requirement.

Precast concrete in the precast plant with high level of control.

3.2. Current practice in aggressive industrial environment.

Typical concrete pathologies in structures under industrial aggressive environments include cracking produced by expansion and efflorescence. In the case of acid attacks, the dissolution of cement paste and non-stable calcareous aggregates can occur.

3.2.1. Concrete basins for mud, water, debris (EGP)

*Type of element***:** Concrete basin made of cast in situ concrete, to contain fluids used for drilling of geothermal wells.

Current structure concept
Inside a well pad construction there are some basins made of concrete for the storage of water needed for drilling activities and for the collection of debris and fluids (such as mud) produced during the drilling phase.

Current dimensions and details
In older plants, there were three different types of concrete basins:

- Debris basin: to collect debris produced during the drilling phase.
- Mud basin, to contain exhausted drilling muds and cement used for the well casing;
- Water basin: for water storage, necessary for drilling activities.

Nowadays, for the design of the new well pads, the mud basin and the water basin are merged into a single tank with an internal wall to divide the geothermal water from the exhausted mud. The basins are used for a period of several months, at the end of which they are cleaned and remain unused for several years, until a new drilling.

According to old design standards, the basin has been made with the following dimensions and materials (Table 3.5).

- **Debris basin**:

Table 3.5.EGP basins characteristics

Basin	Plan dimensions	Wall thickness	Concrete resistance class	Lean concrete	Minimum concrete cover	Steel Reinforce-ment	Other
Debris	5.0 m x 18.0 m, maximum depth 3.0 m, bottom slope about 1,5%.	0.40m; minimum thickness of slab foundation 0.40m	C25/30	C10/15 thickness 10 cm	4 cm	B450C	two internal walls located at about 2.4 m and 7.4 m from the edge
Mud	5.0 m x 18.0 m, maximum depth 3.0 m, bottom slope about 1,5%.	0.40m; minimum thickness of slab foundation 0.0m	C25/30	C10/15 thickness 10 cm	4 cm	B450C	one internal wall 25cm thickness located at about 3.0m from the edge
Water	16.8 m x 16,8 m maximum depth 4 m, bottom slope about 1,5%	0.4 m; minimum thickness of slab foundation 50 cm	C25/30	C10/15 thickness 10 cm	4 cm	B450C	

The new design standards, in order to ensure a longer service life for the basin, require the use of concrete require with higher performance, as reported in Table 3.6.

Table 3.6.EGP basins characteristics

Basin	Plan dimensions	Wall thickness	Concrete resistance class	Lean concrete	Minimum concrete cover	Steel Reinforce-ment	Other
Debris	5 m x 18 m, maximum depth 3 m, bottom slope about 3%	0.4 m; minimum thickness of slab foundation 40 cm	C35/45	C16/20 thickness 10 cm	4 cm	B450C	
Mud	12,6 m x 26,6 m, maximum depth 3,85 m, bottom slope about 4%	0.3 m; minimum thickness of slab foundation 40 cm	C35/45	C12/15 thickness 10 cm	4 cm	B450C	Slump class S3 (foundation slab) S5 (walls) w/c, 0,45 Minimum cement content 360 kg/ m^3 Maximum diameter of the aggregates 20 mm Non-frosty type aggregates (according to EN 12620:2000)
Water	12,6 m x 26,6 m, maximum depth 3,16 m, bottom slope about 4%	0.3 m; minimum thickness of slab foundation 40 cm	C35/45	C12/15 thickness 10 cm	4 cm	B450C	Slump class S3 (foundation slab) S5 (walls) w/c, 0,45 Minimum cement content 360 kg/m^3 Maximum diameter of the aggregates 20 mm Non-frosty type aggregates (according to EN 12620:2000)

Current application technology and employed casting requirement.
Reinforced concrete cast in situ; material qualification and acceptance through laboratory tests; in-situ tests to check for appropriate execution.
During the execution of the concrete works the slump of the concrete is checked and laboratory tests are executed to evaluate the compressive strength of concrete, according to Italian code (NTC 2018).

Current experienced operational issues.
For older basins which are in operation for several years, problems about cracking and loss of waterproofing have been found.

3.2.2. Concrete basins for water (EGP)

Type of element
Concrete basin made of cast in situ concrete, to contain the water coming from the cooling tower.

Exposure environment: XA

Current structure concept
To condense the steam coming out of the turbine, the steam comes into contact with cold water produced by the cooling tower. The tower consists of a wooden or fiberglass super-structure and a concrete basin at the base for water collection. The flows involved are of the order of 3500m3/ h.

Current dimensions and details
The size of the tank depends on the type of plant but in general they are about 35,0/40,0m x 14,0/18,0m in plan while the walls are usually 2,0/2,5m height (from the bottom of the basin).

According to old design standards, the basin have been made with the following dimensions and materials (Table 3.7).

Table 3.7.EGP characteristics of a water basin in a cooling tower

Basin	Plan dimensions	Wall Thickness	Concrete resistance class	Lean concrete	Minimum concrete cover	Steel Reinforce-ment	Other
Water basin for cooling tower	35,0/40,0 m x 14,0/18,0m	40 cm slab foundation: usually 50 cm	C25/30	C12/15 thickness 10 cm	4 cm	FeB44K	Wall height: 2,0/2,5m height (from the bottom of the basin) anti-shrinkage additives

Application technology
Reinforced concrete cast in situ; material qualification and acceptance through laboratory tests; in-situ tests to check for appropriate execution.
During the execution of the concrete works the slump of the concrete is checked and laboratory tests are executed to evaluate the compressive strength of concrete, according to Italian code (NTC 2018).

Operational issues
For older basins which are in operation for several years, problems about cracking and loss of waterproofing have been found.

3.3. Current practice for crystalline admixture

The self-healing capacity of the crystalline admixture supplied by Penetron has been proved with reference to several types of concretes and curing environments, including rich Cl- environment.

The admixture works properly, with capacity for healing cracks in normal concrete and UHPC, whose compositions are reportedin Table 3.8According to these data, it is expected that the admixture will be effective in the pilots of the Project, since it has been validated in high performance cocnretes.

Table 3.8. Concretes in which crystalline admixture has been validated

	Abrams mm	Cement kg/m³	Addition kg/m³	Admixture kg/m³	w/b	Water penetration mm (pressure)	Steel Fibres kg/m³	Rc Mpa (28 days)
NSC	210	CEM III 360	-	3,6 (1%)	0,44	3	-	40
NSC (Singapur airport)	100±25	CEM I 398	-	3,18	0,46	9,6 16 (CC)	-	47,5
HPFRCC	-	CEM I 52,5 600	Slag 500	3	0,18	-	100	-

3.4. Current practice summary

Table 3.9 summarizes the main facts, as above discussed in detail, about concrete characteristics from current practice examples in maritime works, in which cast-in-situ and precast elements are exposed to XS environment, together with industrial elements exposed to XA environment.

Table 3.9. Current practice summary

	Marine Environment					Industrial Environment
Parameter	**Offshore platforms (HPC general)**	**Offshore floating caisson**	**Docks (bibliography and CMW experience)**	**Precast element**	**UHPC Precast element**	**Basins**
Rc MPa	C40-C85	C40	CA25-30	C40-C60	C140-150	C25-45
w/c or w/b	<0.3	<0.45	0.5-0.65	0,4	0,16-0,18	0,45
Cover mm		>40	40-50	70	20-30	40
Cement or binder kg/m³	380-500	360	300-350	425-450	975-1175	360
Max. Size agg mm	10-14 (basalt, granite)	22		14		
Steel fiber kg/m³					80-160	
Consistency	18-22	18			SCC	
Permeabitlity	-	$10^{-12} – 10^{-8}$				
Water penetration			30-50 mm			
Cb %		<0.3				
Type of cement + addition	SF, BFS, FA			CEM I + GGBS	CEM I42.5 R/SR + SF	

CT = total chloride content, Cf = free chlorides, Cb = bound chlorides

4. *Current pathologies in aggressive environments.*

In this section the most common pathologies associated with current practice are described for each environment. The information has been collected from partners own experience and from the literature. Main conclusions about the most important problems in each environment are shown in order to take them into account for the approach of Durability Assessment Based Design.

4.1. Case studies in marine environment.

Complete information about these case studies is included in Annex I.

4.1.1. Durability of marine concrete: docks case study.

A study about estuaries and harbours of the South western part of The Netherlands has been analysed.

Six structures were chosen for field investigations that were thought representative of a larger group of structures.

Properties such as strength, cover thickness and resistivity of concretes were studied as well as chloride concentration profile along cover thickness.

One of the most important conclusions of this case study is that, neglecting minor differences of w/c and age**, Portland cement concrete in the splash zone has a much lower effective diffusion coefficient than in submerged zone**.

4.1.2. Concrete reinforcement corrosion in marine (Mediterranean) environment: submerged zone and intertidal zone

This study is specially focused on pathologies of concrete caissons in 7 ports of Spain. Samples from them were characterized from corrosion point of view. All these caissons had been executed properly.

In Spain, the caissons are built with pontoons or floating dams, so they are in contact with sea water from a very early age. A caisson equivalent to the structure of an 8 story-building of 8can be built in a week, by progressive sinking. In all the docks, the caissons were manufactured in situ; due to the construction procedure, the curing was carried out with seawater.

In submerged concrete there has been no sign of corrosion until 30 years. In the tidal zone there is corrosion but not deterioration due to low coating or high penetration. In the three environments (air, tides and submerged) concrete with blended cement performs better (lower water penetration and lower diffusion coefficient). According to data obtained from sample characterization a qualitative assessment is made in order to classify the concrete depending on several parameters. The point here is to highlight that the quality for the same concrete can be different depending on which parameter is chosen for the study (see Table 4.1). Quality classification for each parameter was made taking into account concrete properties from CEB-FIP Bulletin 243.

It is observed that the classification criterion is not uniform for the different properties. Therefore, the porosity criterion establishes an intermediate quality of concrete with a fairly narrow range, while those of capillarity and absorption are excessively wide and even more the criterion of water permeability.

Table 4.1. Qualitative assessment of the concrete

Dock	Age (years)	Caisson	Rc MPa	UltraSound	Porosity	Permeability	Absorption	Capilarity
A	4,5	A1-A2	44,3	Good-excellent	Bad	Medium	Medium	Medium
		A3-A4	48,7	Excellent	Bad	Medium	Medium-bad	Medium
		A5-A6	44,6	Excellent	Bad-medium	Medium	Medium	Medium
B	6,5	B1-B2	44,5	Good-excellent	Medium	-	Medium	Medium
		B4-B5	42,8	Good-excellent	Bad	Medium	Bad	Medium-bad
C	7,5	C1	40,0	Good-excellent	Good	-	Medium	Medium-bad
		C2	39,4	Excellent	Good	Medium-good	Medium	-
		C3	38,3	Excellent	Good	-	Medium	Medium-bad
D	5	D1	36,4	Good	Bad	Medium	Medium	Bad
		D2	-	Good	Bad	Medium	Medium	Bad
		D3	33,4	Good	Medium	Medium	Medium	-
		D4	24,0	Good-acceptable	Bad	Medium	Medium	-
E	2	E1	38,1	Excellent	Bad	Good	Medium	Medium
		E2	47,2	Excellent	Bad	Good	Good	Good
		E3	54,5	Excellent	Bad	Medium-good	Medium	Medium
		E4	31,8	Excellent	Bad	Medium-good	Medium	Medium-good
F	2	F1	34,6	Excellent	Bad	Medium	Medium	Medium
		F2	50,5	Excellent	Medium	Medium-good	Medium	Medium
		F3	29,3	Excellent	Medium	Medium	Medium	-
		F4	-	Excellent	Medium	Medium	Medium	Medium-good
G	31	G1	-	Excellent	Bad	Medium	Bad	-
		G2	31,9	Excellent	Bad	-	Bad	Medium
		G3	-	Excellent	Bad	-	Medium	Bad
		G4	28,1	Excellent	Bad	Medium	Bad	Medium
		G5	-	Good-excellent	-	-	Bad	-

Depending on the environment in which the concrete is located (tidal or submerged), the relative quality limits associated with each of the tests will be different, since different transport mechanisms prevail in different environments. Besides, the addition of mineral blenders also affects these limits indicated by CEB-FIP Model Code, since they have a different influence on some or other physical properties of the concrete.

Conclusions of this study

- **There is a necessity to apply sulphate resistant cements in marine environments. Execution of works is very important to avoid permeable concretes.**

- **The control of the strength does not guarantee the durability**: In almost all the caissons studied, the resistance to compression of the extracted specimens was above 30 N/mm^2, and in some cases it was even higher than 50 N/mm^2. However, despite these good strength values, the tests carried out to evaluate the impermeability of the concrete denote in general a medium or even bad quality.

- **The favourable effect of the additions on the durability has been very important**. Two of the docks built with blended cement (E and F docks) are in tidal zone, although the concrete dosage used did not comply the requirements of minimum cement content and maximum water/cement ratio required in this environment. However, these concretes did meet the specifications of water penetration in both cases, due to the beneficial effect of the additions on the permeability. The presence of C_3A is less determinant than the compactness of the concrete

- There is a good correlation between penetration and chloride diffusion coefficient. **Chloride content influence (surface and critical) on corrosion is not clear and it is different for aerial, tidal or submerged zone.**

- The critical content of chloride in submerged concrete could be higher than that indicated by the Spanish standard (0.4%) because oxygen does not penetrate inside, so corrosion does not start even with higher concentrations of Cl^-.

The critical contents of chlorides collected in the literature vary between 0.17 and 2.2%, expressed as % Cl^- to total weight of cement. The following Table 4.2 shows the critical content of chlorides (by weight of cement) that causes the onset of corrosion collected by different references:

Table 4.2. Critical chloride content from literature

	Soluble in water	Soluble in Acid
ACI 201	0.10 a 0.15*	-
ACI 222	-	0.20
ACI 318	0.15 a 0.30	0.20
BS 8110	-	0.40
Australian Standards	-	0.60
RILEM	-	0.40
Norway Standards	-	0.60
Hope and Ip	-	0.10 a 0.20
Everett and Treadaway	-	0.40
Thomas	-	0.50
Hussain et al. Rasheeduzzafar¥	-	0.18 a 1.2
Page and Havdahl	0.54	1.00
Stratfull	-	0.15

4.1.3. Concrete reinforcement corrosion in ocean environment worldwide: submerged zone and intertidal zone

The document describes corrosion examples in concrete exposed to marine environment all around the world compiled from the literature. It includes several kinds of elements and environments but all exposed to XS environment.

The study compiles data from different structures exposed to marine environment in different countries, providing data about the structure, the original concrete that was applied and the pathologies and concrete characterization after a period of time.

The following table show a comparison between the analysed case studies.

Conclusions

The chloride content on the surface of the concrete, which is a fundamental data to be able to determine the diffusion coefficient of chloride, is not clearly defined, since very different values are found in the surveyed literature.

The same happens with the critical content of chlorides that it is supposed to start the corrosion. The international standards are not unanimous with respect to the requirements adopted for the dosage of concrete located in the marine environment; the water/cement ratio, the cement content, the coating and the maximum allowable chloride content vary from one to the other.

Table 4.3. Comparison between data from different international case studies related to pathologies in marine environments

Structure/element		Resist. (MPa)	Age (years)	Cover (mm)	w/c ratio	Quality execution	Resistivity (kficm)	Permeab. ($m/s \times 10^{-11}$)	Porosity (%)	Corrosion	D1. ($\times 10^{-8}$ cm^2/s)	Environment	Carbonat. (mm)	Cl- content		
														Surface	Bars	Bars*
Dam 20 Portugal	Walls	17.6	5	40	0.7	Bad	4 - 8	14	17.8	Very serious[f]	70.9 - 31.8	2 cycles/year	10 - 25	0.2 - 0.7 [a]	0.14 - 0.40 [a]	1.07 - 3.06
	Inferior slab	20.6	5	60	0.7	Bad	-	0.6	16.4	Serious[f]	-	2 cycles/year	10 - 25	-	-	-
Dam21 Portugal	Walls	17.6	5	40	0.7	Bad	4 - 8	14	17.8	Very serious[f]	248 - 378	2 cycles/month	10 - 25	0.18-0.22[a]	0.15 - 0.19 [a]	1.15 - 1.45
	Inferior slab	20.6	5	60	0.7	Bad	< 1.5	0.6	16.4	Serious[f]	-	2 cycles/month	10 - 25	-	-	-
Dam 22 Portugal	Walls	17.6	5	40	0.7	Bad	1.5 - 3	14	17.8	Very serious[f]	-	2 cycles/month	10 - 25	-	-	-
	Inferior slab	20.6	5	60	0.7	Bad	< 1.5	0.6	16.4	Serious[f]	-	2 cycles/month	10 - 25	-	-	-
Dock Portugal	Piles	-	24	-	-	-	< 5	-	-	-	-	Tidal-submerged	-	0.24 [a]	0.18 - 0.13 [a]	1.38 - 0.99
	Beams	35	24	30-50	-	-	5-15 < 5	-	-	Very serious[f] - Serious[h]	34.3- 1.695	Splash – tidal	-	0.18 [a]	0.14 - 0.17 [a]	1.07 - 1.30
	Slab	30	24	30-50	-	-	5 - 15	-	-	Very serious[f.g]	18.1 - 15.7	Splash	-	0.10 [a]	0.07 - 0.05 [a]	0.54 -0.38
Bridge Portugal		50	35	< 20	0.32	Bad	> 40	10	-	Media[f]	0.7 - 1.2	Air	-	0.16-0.34[a]	0.02 -0.09[a'c]	0.15 - 0.65
	Land side	52.8	40	49.6	0.53	-	-	-	10.6	replaced[1]	62.8	Air	-	5.26 [b]	4 [b]	1.33

Viaduct Rocky Point USA	Ocean side	52.8	40	49.6	0.53	-	-	-	10.6	replaced [1]	48.8	Air	-	9.58 [b]	7 [b]	2,33
Bridge Brush Creek USA		-	30	25.5	-	-	-	-	-	replaced [1]	22.5 - 57.0	Air	-	4.5 - 7 [b]	3.5 - 6 [b]	1,17 - 2
Dam Muroran (Japan)		34	66	75-175	0.73	-	-	-	16	Minimum[i]	12.2 - 64.2	Tidal-submerged	0	3.8 [b]	1.4 [b]	0.75
Sidney Dock (Australia)		40	20	40	0.53	Good	2.4	-	-	Medium[f]	-	Splash	5 - 15	-	0.10 -0.50[d,e]	0.1 - 0.5
Bridge Tay Road (United Kingdom, Scotland)		22-68	25	75	0.44-0.66	Bad	-	-	-	Serious[f]	10.9-33.3	Splash	-	0.6-2.5[e]	0.15-0.89 [e]	0.15-0.89
Reinforced concrete Dam (Japan)		18-36	20	40	0.6-0.7	Bad	1.5-3	-	17-20	Serious[f]	42.9-20.1	1 Cycle/week	10	0.38-0.20 [a]	0.30-0.14 [a]	2.3-1.1

a = % Cl-/weight concrete
b = kg Cl-/m^3 concrete
c = at 25 mm
d = at 50 - 70 mm
e = % Cl-/weight cement
f = cracking and covering detachment. Section loss in bars is not very high
g = coorrosion macrocells, high section loss.
h = black or green corrosion
i = undamaged concrete, weight loss in bars up to 1.11%
* = % Cl-/weight cement, estimated 300 kg of cement in 2300 kg of concrete, Muroran Port 186 kg.

4.1.4. *Visual inspection in different harbour docks (CMW internal reports)*

- **Casablanca (Morocco)**

Several pathologies in the docks of Casablanc Port (Figure 4.1 to Figure 4.3) have been highlighted and studied related to concrete - steel degradation produced by several effects: mechanical actions, concrete wear, reinforcement corrosion, cracking, insufficient coating, etc. These pathologies require repair to avoid aggravation. The situation of unusable or inefficient defences, which can cause damage to concrete in an indirect way because of failure to provide the necessary protection is also shown.

Figure 4.1. Degradation of the concrete slab

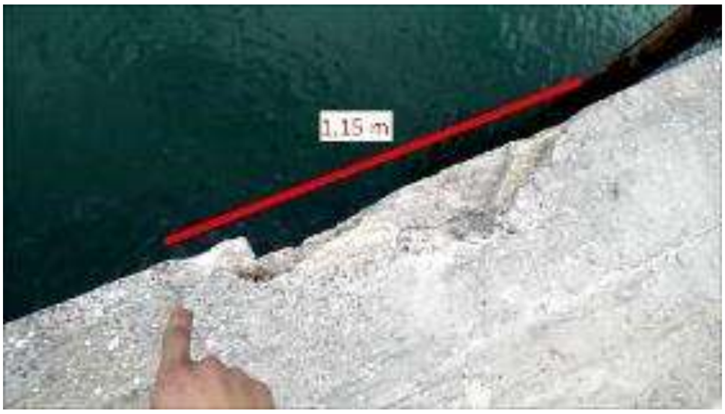


Figure 4.2. Spalling

Figure 4.3. Bollard of damaged concrete

- **Laayoune (Morocco)**
  Special attention must be paid on coronation beams (Figure 4.4Figure 4.5), which are subjected to mechanical actions, which can cause deterioration of the concrete and subsequently steel.

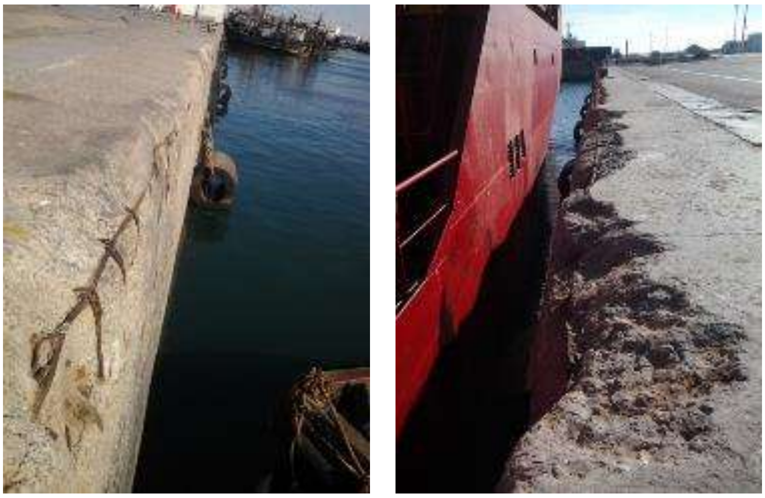


Figure 4.4. Vertical paraments

- **Agadir (Morocco)**
  Most of the forehead of docks is severely damaged (Figure 4.5). The majority of the surface of the front side of the slab is affected by carbonation of the concrete, exposing the reinforcement. The second most important degradation consists of breakdowns, failure or lack of adequate defences to the stressed induced by the berthing of ships. This degradation will lead to mechanical breakdowns of in the concrete.

Figure 4.5. Carbonation and corrosion

- **Valencia (Spain)**
  The different elements of docks are studied periodically by visual inspections: slabs, piles, caissons. Ordinary inspections reveal that, although on the top side of the slab the state of concrete is good, on the lower face the situation is not so good (Figure 4.6).

  The possible cause of damage in the lower side of the slab can be the following: the air under the slab is not easily renewed since the tidal range is very short in Valencia. Water condensation is boosted by temperature variations between night and day, creating an aggressive airborne environment. In such a situation ventilation is important in order to avoid corrosion since it will help to reduce the water condensation.

Figure 4.6. Block of cantile beam

4.2. Case studies in industrial aggressive environments

4.2.1. *Basin for cooling tower: Pianacce Power Plant*

Source: Own construction
Kind of element: Concrete basin made of cast in situ concrete, to contain fluids of cooling tower.

Exposure environment:
Very aggressive chemical environment

Type of structure.
Year of construction 1985.
14.40m x 38.14m, concrete class 25/30, concrete cover 40 mm, ordinary reinforced concrete.

Pathology description.
For older basins which have been in operation for several years, problems concerning cracking and loss of waterproofing have been found (Figure 4.7). The main cause of degradation is the ageing of waterproofing products applied to the walls and to bottom of the basins, also due to the action of geothermal water.

Maintenance.
The following treatments are carried out to restore the water-resistance of the basin (internal walls and bottom of the basin).

The works concern the basin of the Unit 3 of the Pianacce cooling tower and it consisted of the remaking of the bituminous coating, after localized restoration of the concrete and sealing of construction joints between foundation and elevation.

Figure 4.7. Pianacce Power Plant pathology details.

4.2.2. *Basin for cooling tower: Farinello Power Plant*

Source: Own construction

Type of element: Concrete basin made of cast in situ concrete, to contain fluids of cooling tower.

Exposure environment
Very aggressive chemical environment

Type of structure.
Year of construction 1985.
16.90m x 16.70m, concrete class C 25/30, concrete cover 40mm, ordinary reinforced concrete

- Wall height: 1.80m height (from the bottom of the basin)
- Thickness of slab foundation: 400 mm
- Wall thickness usually 450 mm
- Concrete resistance class C25/30
- Exposure class XA3
- Lean concrete C12/15 thickness 50 mm
- Minimum concrete cover 40 mm

Pathology description.
For older basin which have been in operation for several years, problems concerning cracking and loss of waterproofing have been found. The main cause of degradation is the ageing of waterproofing products applied to the walls and to bottom of the basins, due also to action of geothermal water.

Maintenance.
The main works concerned the structural restoration of 2 central columns of the tower that were strongly deteriorated (Figure 4.8); it was necessary to create a special metal support structure, before restoring the reinforcement bars and increasing columns cross section by means of special grout.

After that, additional works have been done to restore degraded concrete of the walls and beams, by means of anti-shrinkage and fiber-reinforced mortars. Similar operation was also performed in unit 4.

Figure 4.8. Farinello Power Plant pathology details

4.2.3. *Basin for cooling tower: Chiusdino Power Plant*

Source: Own construction

Type of element: Concrete basin made of cast in situ concrete, to contain fluids of cooling tower.

Exposure environment : Very aggressive chemical environment

Type of structure

Year of construction 2009, 38.20m x 14,80m, concrete class 28/35, concrete cover 40 mm, ordinary reinforced concrete

- Wall height: 1,95m height (from the bottom of the basin)
- Thickness of slab foundation: 500 mm
- Wall thickness usually 400 mm
- Concrete resistance class C28/35
- Exposure class XA3
- Lean concrete C12/15 thickness 100 mm
- Minimum concrete cover 40 mm
- Steel reinforcement B450C

Pathology description

The mechanical action of the water falling down from the top of the cooling tower and the chemical aggressiveness of the environment have damaged the upper part of concrete internal walls (Figure 4.10).

Figure 4.9. EGP Cooling Tower Interior

Maintenance

The main works concerned the structural restoration of 2 central walls in the following way:

- cleaning and removal of damaged detaching parts
- application of anti-corrosive mortar on reinforcing bars
- restoration of concrete with fiber-reinforced mortar with compensated shrinkage;

Finally, stainless steel flashings were placed on the head of the walls to protect the concrete from the mechanical action of the water. Considering the very limited available time it was not possible to restore the protective coating system inside the basin.

Figure 4.10. Chiusdino Power Plant pathology details

5. Pilot pre-design: performance demands

In addition to current practice review and description of scenarios it was considered necessary to include a pre-design of pilots in order to provide more information to WP4 and WP5 about the concrete to be tested and validated and design a suitable number of Ultra High Durable Concrete mixes capable to meet the requirements of all the pilots.

5.1. OFFSHORE FLOATING PLATFORM

Description
The pilot of CMW will consist of a floating platformfor offshore wind turbines, full scale size of 10 m diameter, 30 m height, and 50 m of arm length.

A full scale model has been pre-designed and a preliminary assessment of main loads has been made. According to the chosen model of wind turbine (NREL 5 MW) and its loads, the following maximum values can be established (Figure 5.1):

$$F_{t,max} = 750\ kN \text{ (maximum thrust force)}$$
$$Mx,_{max} = 43\ kN \cdot m \text{ (maximum rotor torque)}$$
$$F_{z,wt} = W_{nacelle} + W_{rotor} + W_{tower} = 6.835\ MN\ (weight\ of\ the\ wind\ turbine)$$

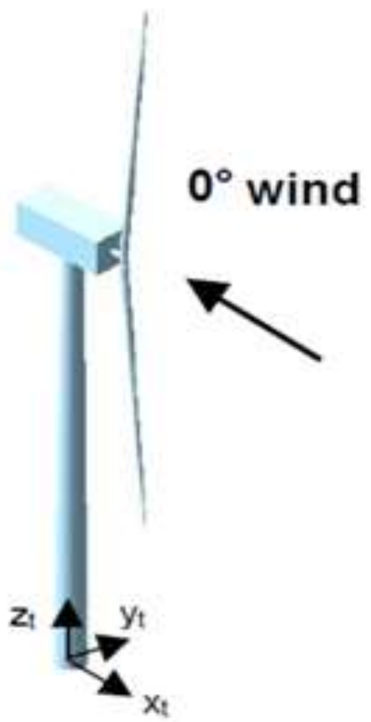


Figure 5.1. Origin of coordinate system

This is the floating platform that has been chosen for the full scale design. Prototype will consist of the central floater that will support the turbine and one of the platform arms.

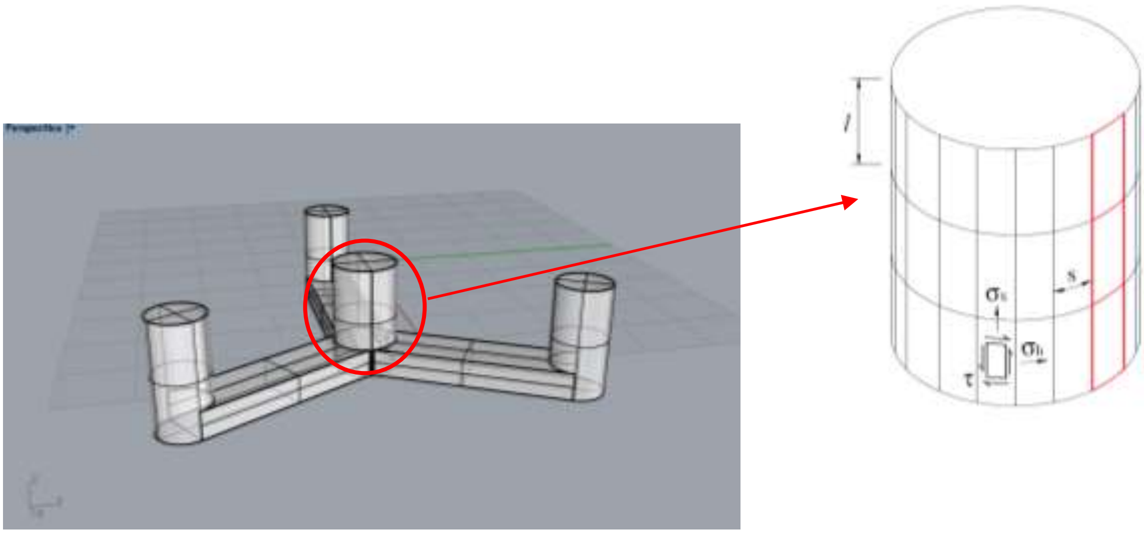


Figure 5.2. CMW prototype scheme

Different constructive solutions have been assessed for cylindrical bodies made in steel (Figure 5.3). These techniques are based on the **DNV-RP-C202** guidelines. The objective of reviewing these techniques is to find an equivalent one made of concrete instead. For this purpose, reference the Bureau Veritas standard **NI 594: Design and Construction of Offshore Concrete Structures** will be taken into account**.**

Dimension ***l*** has been chosen as **3 m** for adequacy. Ring frame has an external and internal diameter of **9** and **8 m** respectively. Longitudinal stiffener will be placed between the shell and the internal ring frame diameter. External column section will be divided in **12 sections** as shown in Figure 5.3. For the calculation it will be considered that all the steel volume of rings, stiffeners and shell is the shell with an equivalent thickness.

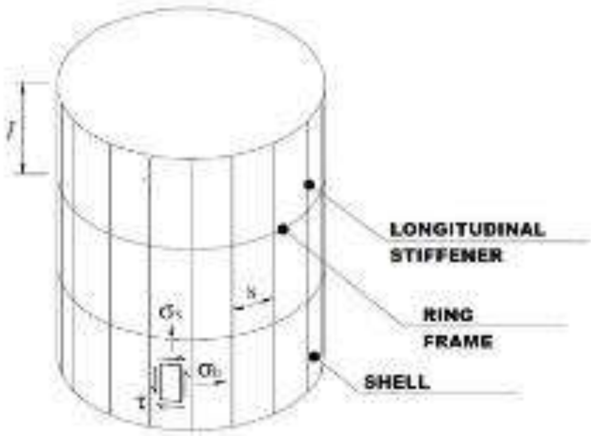


Figure 5.3. CMW prototype scheme

Construction material for this prototype is Naval steel A.

Figure 5.4. Material requirements for CMW pilot

Material	Type	Yield strength	Safety factor used	Max design stress
Steel	Naval steel A	235 MPa	2	117,5 MPa

For the simplified shell preliminary design, two hypothese's were considered: operational and extreme conditions. The results obtained in this section will be used in the calculation of a concrete shell with the same dimensions. The calculation is done in the worse situation that can be found. $M_{x,max}$ is considered to be negligible since F_z and $M_{o,y}$ are far greater than it.

Operational conditions:
Under this condition, it is assumed that the platform is **5 degrees pitched**. $M_{o,y}$ (bending moment at the centre of the column base on y axis) is calculated as follows:

$$\begin{aligned}|M_{o,y}| &= M_{y,weight} + M_{y,thrust} + M_{y,inertial}\\ &= F_{z,global} \cdot [x_{cm} + y_{cm} \cdot \sin(\alpha_{max})] + F_{t,max} \cdot (H_{hub} + H_{column}) + y_{cm} \cdot m_{WT} \cdot a_{max}\\ &= 303.98\ MN \cdot m\end{aligned}$$

Extreme conditions:
Under this condition, it is assumed that the platform is **17 degrees pitched**. This may occur, for example, in a storm situation. It is permissible to ignore $F_{t,max}$ term in this conditions as the turbine will be in safety position. Under this hypothesis, we recalculate $M_{o,y}$ as follow:

$$\begin{aligned}|M_{o,y}| &= M_{y,weight} + M_{y,inertial} = F_{z,global} \cdot [x_{cm} + y_{cm} \cdot \sin(\alpha_{max})] + y_{cm} \cdot m_{WT} \cdot a_{max}\\ &= 457.48\ MN \cdot m\end{aligned}$$

$F_{z,global}$ remains the same in both conditions and has been calculated as:

$$F_{z,global} = F_{z,wt} + F_{z,column} = 15.87\ MN$$

To calculate the stress resultants governing the stresses in a cylindrical shell both compression and tension will be checked separately.

Compression stress: The stress distribution using the Navier's equation is found as:

$$|\sigma_{c,max}| = \frac{F_{O,z}}{S} + \frac{M_{O,y}}{I_y} \cdot y_{max}$$

For a cylinder shell:

$$S = \frac{\pi}{4} \cdot (\phi_{ext}^2 - \phi_{int}^2)$$

$$I_y = \frac{1}{64} \cdot \pi \cdot (\phi_{ext}^4 - \phi_{int}^4)$$

$$y_{max} = \frac{\phi_{ext}}{2}$$

$$\phi_{int} = \phi_{ext} - 2 \cdot t_{eq}$$

The maximum design stress has been defined inFigure 5.4:

$$\sigma_{max,c} = 177.5\ MPa$$

Tensile stress: In the same way, maximum tensile stress stress is found as:

$$|\sigma_{t,max}| = -\frac{F_{O,z}}{S} + \frac{M_{O,y}}{I_y} \cdot y_{max}$$

In this case, the stress is a bit smaller than calculated in compression:

$$\sigma_{max,t} = 144.17\ MPa$$

Since no stiffeners were considered for this calculation, the equivalent thickness in the worse situation (maximum compression) has been found as:

$$t_{eq} = 26\ mm$$

This equivalent thickness has been calculated by imposing the maximum design stress and solving the Navier's equation for t_{eq}.
Finally, starting from the values of strees, compression and tensile forces can be calculated.

Compression. Axial force acting over each steel shell can be calculated by dividing the stress obtained above by the transversal section of the panel:

$$F_{max,c} = 14.12\ MN$$

Tension. In the same way, axial force can be calculated as follow:

$$F_{max,t} = 11.47\ MN$$

Complete full scale model is under development to complete this first approach. After that, the prototype model with equivalent loads will be defined. It is estimated that the prototype will be manufactured at scale from 1:5 to 1:7.5. Figure bellow shows a proposal of the 1:5 scale prototype:

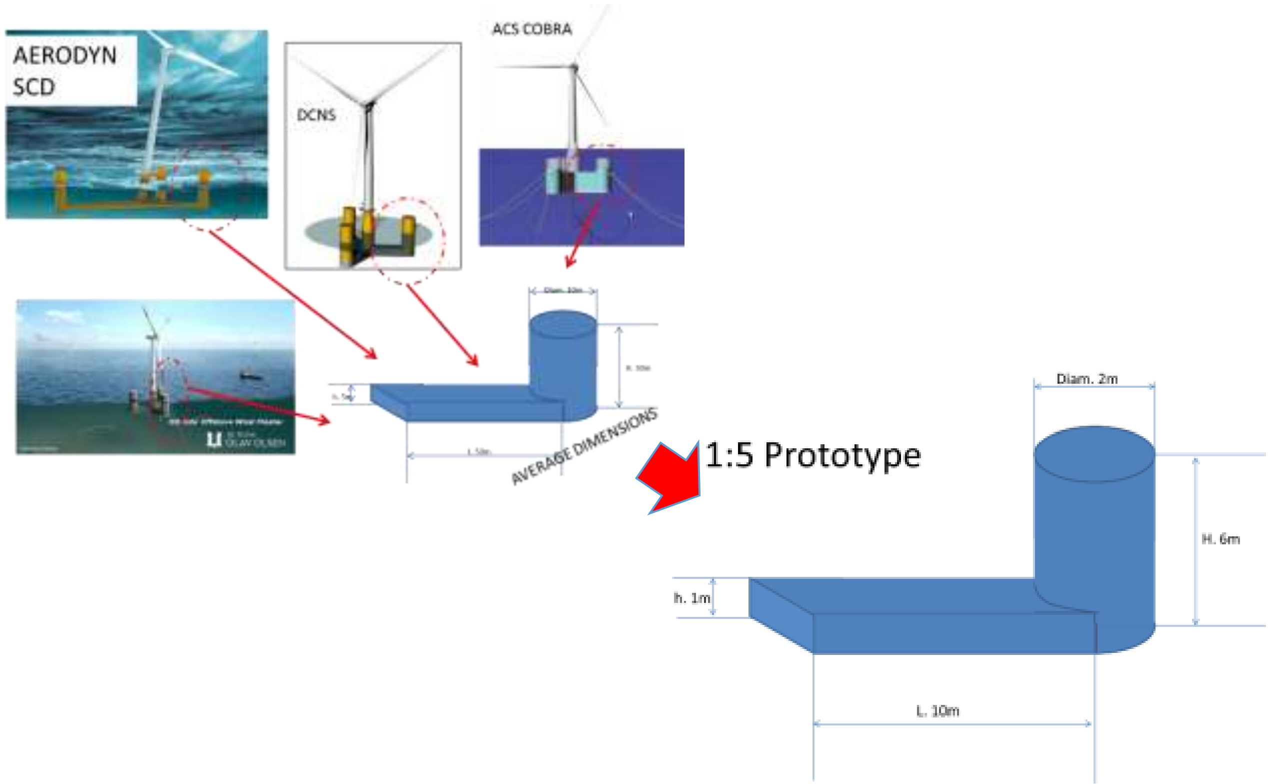


Figure 5.5. CMW prototype scheme

Concrete characteristics

Roughly, it has been estimated that an equivalent prototype in reinforced concrete would require a thickness about three times higher (10-12 cm). The assembly will be calculated according to requirements of the element, when the detailed design phase will be developed in WP8. The required concrete will be a UHPC C100-C120 with steel fibers and a cement amount around 1.000 kg/m^3.

Expected pouring conditions: Plane, with formwork, self-compacting.

5.2. AQUACULTURE MUSSEL RAFT

Description

Floating structure with a size of 20 m x 27 m, composed of beams with a section of 23 x 23 cm and others with a section of 35 cm (width) x 23 cm (depth).

Pre-stressed.

Environment XS3.

There is abrasion with the impact of the waves along the lateral surface of some beams.

Loads: The dynamics of the sea (currents and waves) and the weight of the harvest.

The pilot will be similar to the one in the picture (subjected to changes).

Figure 5.6. Floating structure for RDC pilot

Section of 230 x 230 mm and others with a section of 350 mm (width) x 230 mm (depth).

Total dimension of 20x27 m.

The primary beams are pre-stressed and have a width of 350 mm and a depth of 230 mm, they are lightened, and the wall thickness is of 60 mm.

The geometric cover is 20 mm.

The amount of pre-stressing steel is 180 kg/m^3 for the primary beams and 256 kg/m^3 for the secondary beams.

Concrete characteristics

Different concretes will be tested in the different beams:

- One, with the most risky design, would be done with less pre-stressing than the current and with a concrete with approximately 110 MPa strength (average compressive strength on 100 mm side cubes).
- Other beams will be designed with a concrete with approximately 140 MPa strength (average compressive strength on 100 mm side cubes) and higher pre-stressing in the beam. All data are referred to air curing 20ºC, 28 days.

The employed UHDC will be required to exhibit a minimum strain capacity equal 0.4% before the onset of the strain softening branch and a minimum tensile strength equal to 5 MPa.

Design in ULS considering that the most tensioned pre-stressing steel will have a strain of 1%. Pre-stressing is done with a strain of approximately, 0.6%.

In at least one of the beams, RDC would likely be able to be in service accepting any level of strain as far as we are in strain-hardening, without opening of macrocracks.
In other/other beams design will be made under frequent loads without micro-crack opening (as generally done with pres-tressing), but being conscious that during the service life some cracks may appear and can heal, exploiting the self-healing functionality incorporated through tailored constituents peculiar to the project.

Pouring conditions: Plane, with formwork, self-compacting.

5.3. FLOATING JETTY

Description
Precast breakwater elements along the British Isles coast. The proposal is to examine concrete behaviour based on the exposure classes set out in EN206 and BS8500 for marine exposure XS3.
The pilot will consist 3 floating pontoons 3x1,2x1,2 m with a wall thickness of 120 mm. It will have an upper piece made of metallic galvanised frame and a deck of (glass?) fibre reinforced concrete.

Concrete characteristics
The initial demanded characteristics of the concrete are the following: C90 and cover 50 mm
The pilot will be located in Galway Bay, on the west coast of Ireland.
It will be floating in the Pier and immersed in the Atlantic sea.
Although these demands are below a UHDC characteristics, Banagher is working in a continuous feedback with partners in WP4 to redesign the pilot with an UHDC.

Figure 5.7. Banagher breakwaters in Irish coast.

Figure 5.8. Banagher breakwaters in Irish coast

Nominal cover[a)] mm	Compressive strength class[f)], maximum w/c ratio and minimum cement or combination content for normal-weight concrete[m)] with 20 mm maximum aggregate size[n)]											Cement/combination types
	30 + Δc	35 + Δc	40 + Δc	45 + Δc	50 + Δc	55 + Δc	60 + Δc	65 + Δc	70 + Δc	75 + Δc	80 + Δc	
Corrosion induced by chlorides from sea water (XS exposure classes) adequate for any associated carbonation induced corrosion (XC)												
XS1	—	—	—	—	—	—	—	C45/55[f)] 0.35[g)] 380	C40/50[f)] 0.40 380	C35/45[f)] 0.45 360	C32/40[f)] 0.50 340	CEM I, IIA, IIB-S
	—	C40/50[f)] 0.35[g)] 380	C40/50[f)] 0.35[g)] 380	C35/45[f)] 0.40 380	C32/40[f)] 0.45 360	C28/35 0.50 340	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	IIB-V, IIIA
	—	—	C35/45[f)] 0.40 380	C32/40[f)] 0.45 360	C28/35 0.50 340	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	IIB V ≥25% fly ash, IIIA ≥46% ggbs
	—	—	C32/40[f)] 0.45 380	C28/35 0.45 360	C25/30 0.50 340	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	IIIB
	—	—	C35/45[f)] 0.40 380	C30/37 0.45 360	C28/35 0.50 340	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	IVB-V
XS2	—	—	—	—	—	—	—	—	—	—	—	CEM I, IIA, IIB-S
	—	—	—	—	C40/50[f)] 0.35[g)] 380	C35/45[f)] 0.40 380	C32/40[f)] 0.45 360	C28/35 0.50 340	C28/35 0.50 340	C25/30 0.55 320	C25/30 0.55 320	IIB-V, IIIA
	—	—	—	C40/50[f)] 0.35[g)] 380	C35/45[f)] 0.40 380	C32/40[f)] 0.45 360	C28/35 0.50 340	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	C25/30 0.55 320	IIB V ≥25% fly ash, IIIA ≥46% ggbs
				C35/45[f)] 0.35[g)] 380	C32/40[f)] 0.40 380	C28/35 0.45 360	C25/30 0.50 340	C20/25 0.55 320	C20/25 0.55 320	C20/25 0.55 320	C20/25 0.55 320	IVB-V, IIIB
XS3	—	—	—	—	—	—	—	—	—	—	—	CEM I, IIA, IIB-S
	—	—	—	—	—	—	C40/50[f)] 0.35[g)] 380	C40/50[f)] 0.35[g)] 380	C35/45[f)] 0.40 380	C32/40[f)] 0.45 360	C28/35 0.50 340	IIB-V, IIIA
	—	—	—	—	—	—	C40/50[f)] 0.35[g)] 380	C35/45[f)] 0.40 380	C32/40[f)] 0.45 360	C25/30 0.55 320	C25/30 0.55 320	IIB V ≥25% fly ash, IIIA ≥46% ggbs
	—	—	—	—	—	—	C35/45[f)] 0.35[g)] 380	C32/40[f)] 0.40 380	C28/35 0.45 360	C25/30 0.55 320	C25/30 0.55 320	IVB-V, IIIB

Figure 5.9.Durability recommendations for reinforced or pre-stressed elements with an intended working life of at least 100 years.

This table is very important and is the basis for all Banagher design. The aim is to increase the concretes resistance and reduce the cover requirements.

5.4. WATER BASIN IN A GEOTHERMAL PLANT

Description

Concrete basin with a size of 7.0 m x 20.0 m close to an operating basin of a cooling tower, composed of slab foundation about 500 mm thickness, edge and internal walls with a section of 400 mm thickness.

Ordinary concrete.

Environment XA.

There is the risk of chemical attack due to the presence of geothermal water inside the basin. The pilot is similar to the one in the picture (small-scale) (Figure 5.10).

Figure 5.10. EGP concrete basins

The content of ordinary steel is about 90-100 kg/m^3 both for the foundation slab and the walls.

Concrete characteristics

Although the demanded mechanical resistance for the pilot falls within the usual range of an ordinary concrete, high performance concrete is proposed in order to reduce crack width, cover and thickness of walls and more detailed characteristics will be provided in next deliverables. As the basin is 20 m in length, the tank will be divided into 3 sectors by means of internal reinforced walls in order to test different concretes (see Figure 5.11)

1) The first section will be built as for current state

2) The second section will be built with conventional concrete retrofitted with UHDC layer (sprayed or cast in situ)
3) The third section will be realized reducing the thickness of structural elements (walls and slab), using UHDC with higher tensile strength, strain hardening capacity, and self-healing properties

- High-performance concrete to verify if (under the typical actions anticipated for this kind of structure) a non-cracked concrete service condition (SLS of tensile crack formation) can obtained considering the same thickness of the structural elements. So for this scope it would be interesting a concrete with high tensile strength (5-6 MPa) and a moderate strain hardening capacity.
- Reduction of the thickness of the structural elements, designing for the limit opening state of the cracks, with maximum width equal to w1 = 0.2 mm testing the impermeability of the basin considering the self-sealing properties of the concrete in the presence of geothermal water.

Tensile (stress-strain curve): Tensile strength of a minimum of 5-6 MPa.

Pouring conditions: plane, with formwork.

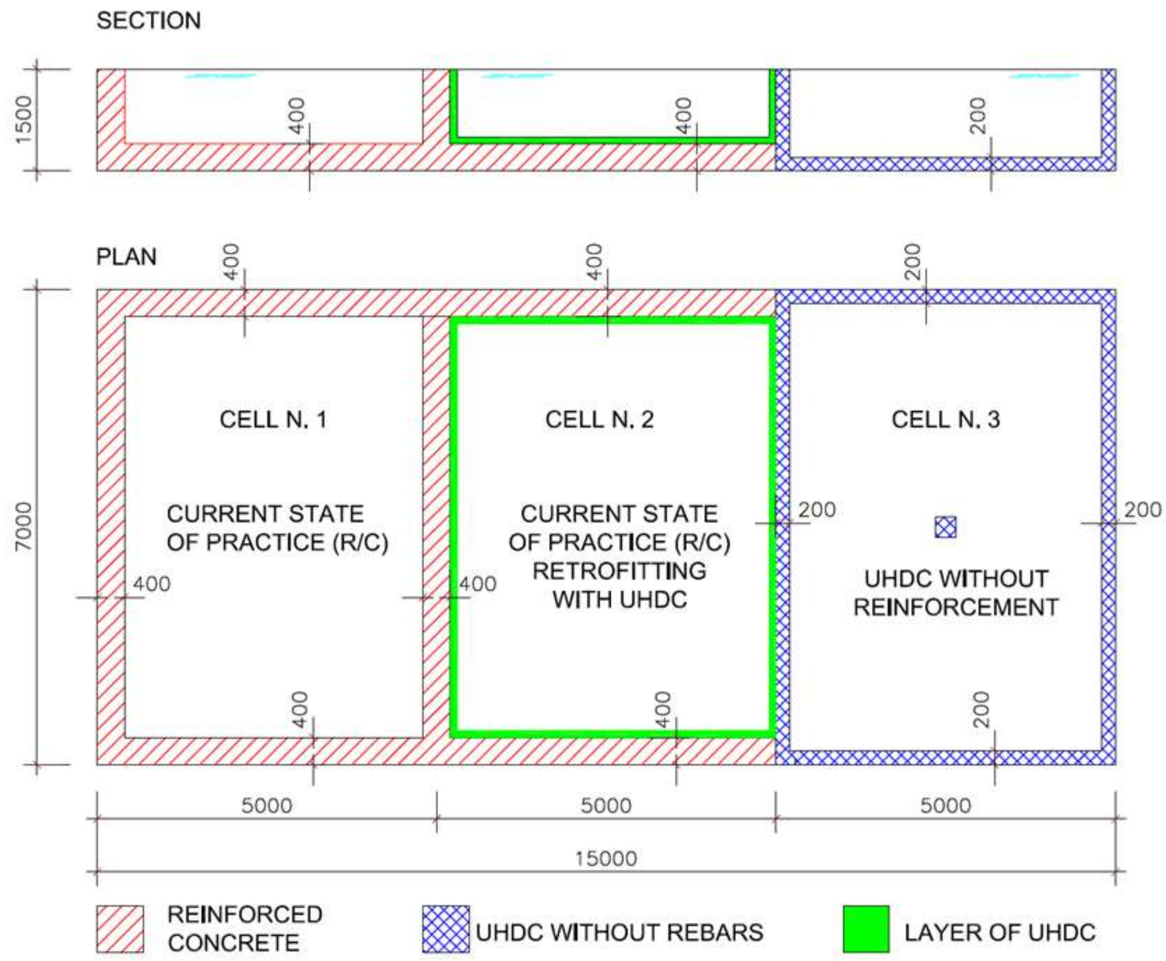


Figure 5.11. EGP pilot layout

5.5. ABATTOIR WATER TOWER

Description

The reinforced concrete water tower was constructed during the first half of the 20th century to serve the water supply requirements of the Civil Abattoir (Figure 5.12). It suffered severe degradation mainly on the side exposed to the sea due to combined chloride rich environment and erosion. It consists of a reinforced concrete structure, cast in situ. The reservoir consists of a drum, open at the top with the lower part consisting of a conical structure and a dome, resting on a ring beam at the top of 12 columns supported on a base. The columns had been encased in a reinforced concrete outer skin during past repair interventions in the second half of the 20th century. The water tower has been in a severely damaged state for a number of years, with severe degradation in specific structural elements, including the south – east external face of the upper drum and conical structure, in the reinforced concrete ring beams and in the columns. The corrosion of reinforcement in parts of the structure of the drum and the conical structure has resulted in the loss in section of about 70 mm, which amounts to about half of the thickness of the drum since the reinforcement is located in the centre of the cross-section. Concrete spalling and delamination as a result of corrosion is observed in various parts of the structure (**Errore. L'origine riferimento non è stata trovata.**) The Water Tower is considered to be a unique structure and the only one of its type in the Maltese Islands (see dimensions and sections in Figure 5.13).

Figure 5.12. Reinforced Concrete Water Tower (Malta).

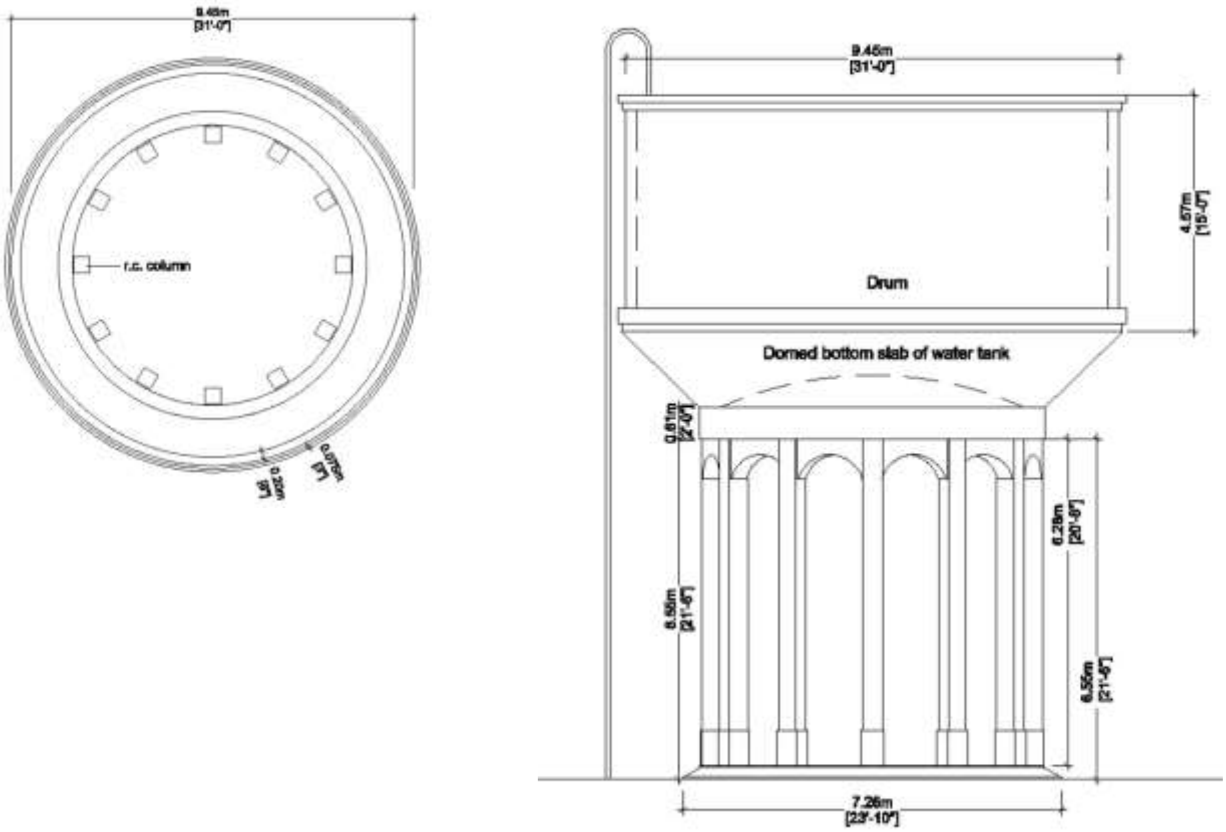


Figure 5.13. Reinforced Concrete Water Tower (Plan and Elevation)

Concrete characteristics
In general, UHDC textile reinforcement (tank) and highly flowable UHDC (support columns) will be used. Monitoring with follow-up using a sensor network system (This is also related to training events). Different UHDC needs to be applied to different parts of the structure depending on the nature of the elements; the materials used and technology applied are described for different elements.

- Columns: The excess concrete used in previous interventions shall be removed from the columns. The concrete is in general of a lower strength than the original concrete of the tower and in the range lower than 20 MPa. The loose spalled concrete parts shall be removed and the corroded steel reinforcement shall be thoroughly cleaned. An epoxy based protective layer shall be applied to the steel reinforcement. A new formwork shall be constructed and an ultra-high durability concrete UHDC, fibre reinforced self-compacting cement based material shall be applied,
- Ring Beams: Any loose spalled concrete elements shall be removed and the corroded steel reinforcement shall be thoroughly cleaned. An epoxy based protective layer shall be applied to the steel reinforcement. A textile reinforced ultra-high performance, fibre reinforced cement based material of low viscosity, shall be applied.
- Base Cone: Any loose concrete shall be removed from the structure together with loose steel reinforcement; the corroded steel reinforcement shall be thoroughly cleaned. An epoxy based protective layer shall be applied to the steel reinforcement. An orthotropic textile reinforcement, and an ultra-high performance concrete, fibre reinforced cement based material of high viscosity, shall be applied. The application using shotcrete with necessary care to the underlying structure shall be assessed.

- Drum: Any loose concrete and steel shall be removed from the structure together with loose steel. (This is particularly the case on the south east-sea facing face of the drum). Any loose spalled concrete elements shall be removed and the corroded steel reinforcement shall be thoroughly cleaned. An epoxy based protective layer shall be applied to the steel reinforcement. The drum area has experienced significant loss of reinforcement on the south east face. An Ultra high durability concrete shall be used to fill the gap of circa 70 mm arising in the south east part of the structure, together with a mesh reinforcement and anchors embedded into the original remaining concrete structure. An orthotropic textile reinforcement, and an ultra-high performance concrete, fiber reinforced cement based material of high viscosity, shall be applied. The application using shotcrete with necessary care to the underlying structure shall be assessed.
- Dome: Repair to the dome shall primarily consist in the removal of any detached concrete and steel, the protection of steel using epoxy based materials and the application of polymer based repair materials.
- Cracks: Epoxy injection shall be used throughout in the case of cracks in the concrete, both on the external surface and also the internal surface.
- Internal Water proofing: An internal water proofing shall be applied to the top of the dome, top of the conical lower part of the tank and the internal areas of the drum. A corrosion inhibitor and sealer shall be employed on all external surfaces.

Figure 5.14. Pathologies (spalling and delaminating) in water tower.

Assessment survey of the condition of the structure:
The reinforced concrete water tower was constructed more than 80 years ago and has experienced severe degradation as a result of its proximity to the sea, within the Grand Harbour area in Malta. The state of degradation was assessed following the compilation of a catalogue of defects and the degradation was mapped with respect to different defects. The main defects include cracking and corrosion of reinforcement spalling and delamination including loss of section in the south facing parts of the drum. The degradation mapping in different parts of the structure including the shell elements and the columns, is presented in Figure 5.15-Figure 5.17.

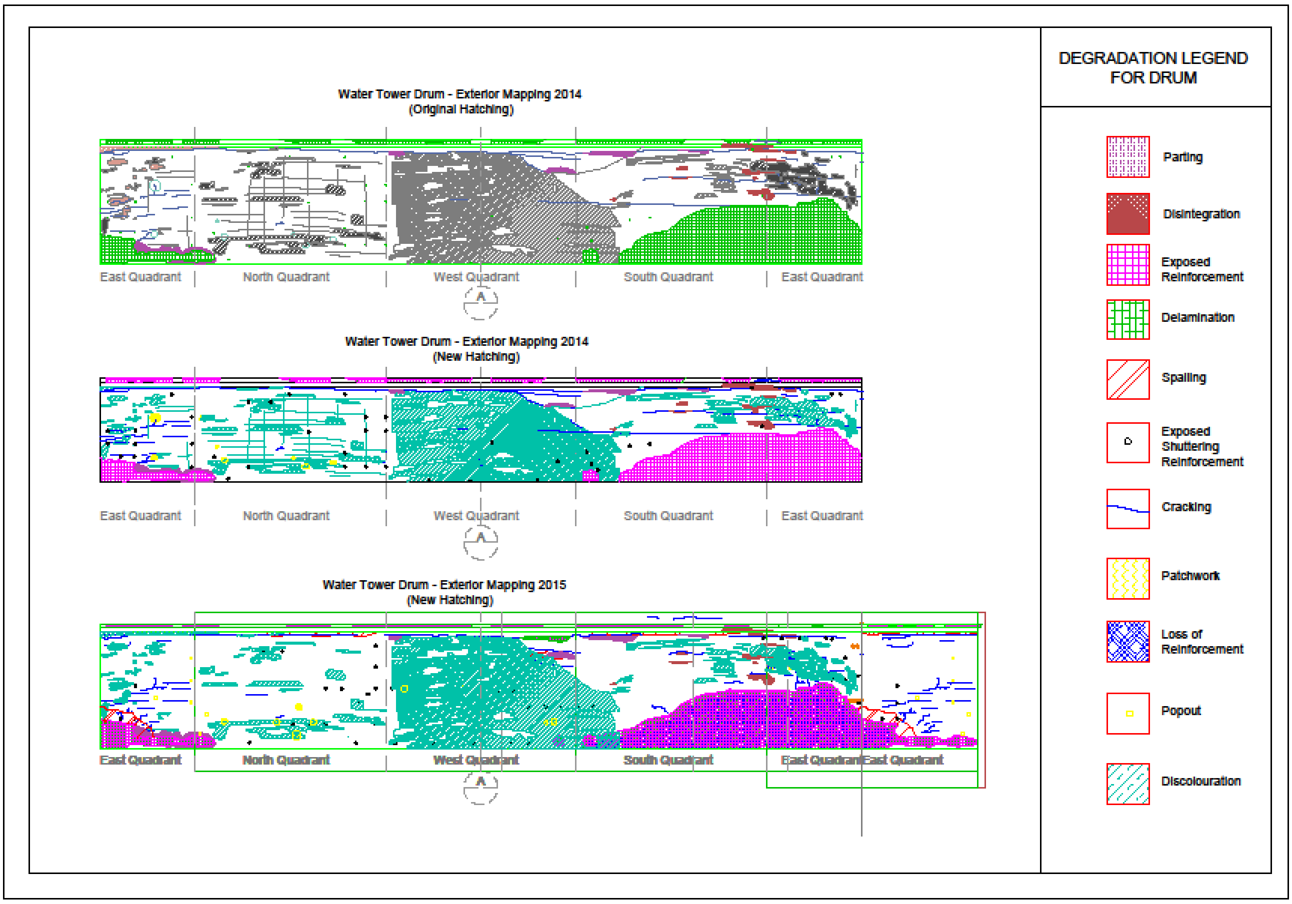


Figure 5.15. Mapping of Degradation in the Drum (Reinforced Concrete Water Tower)

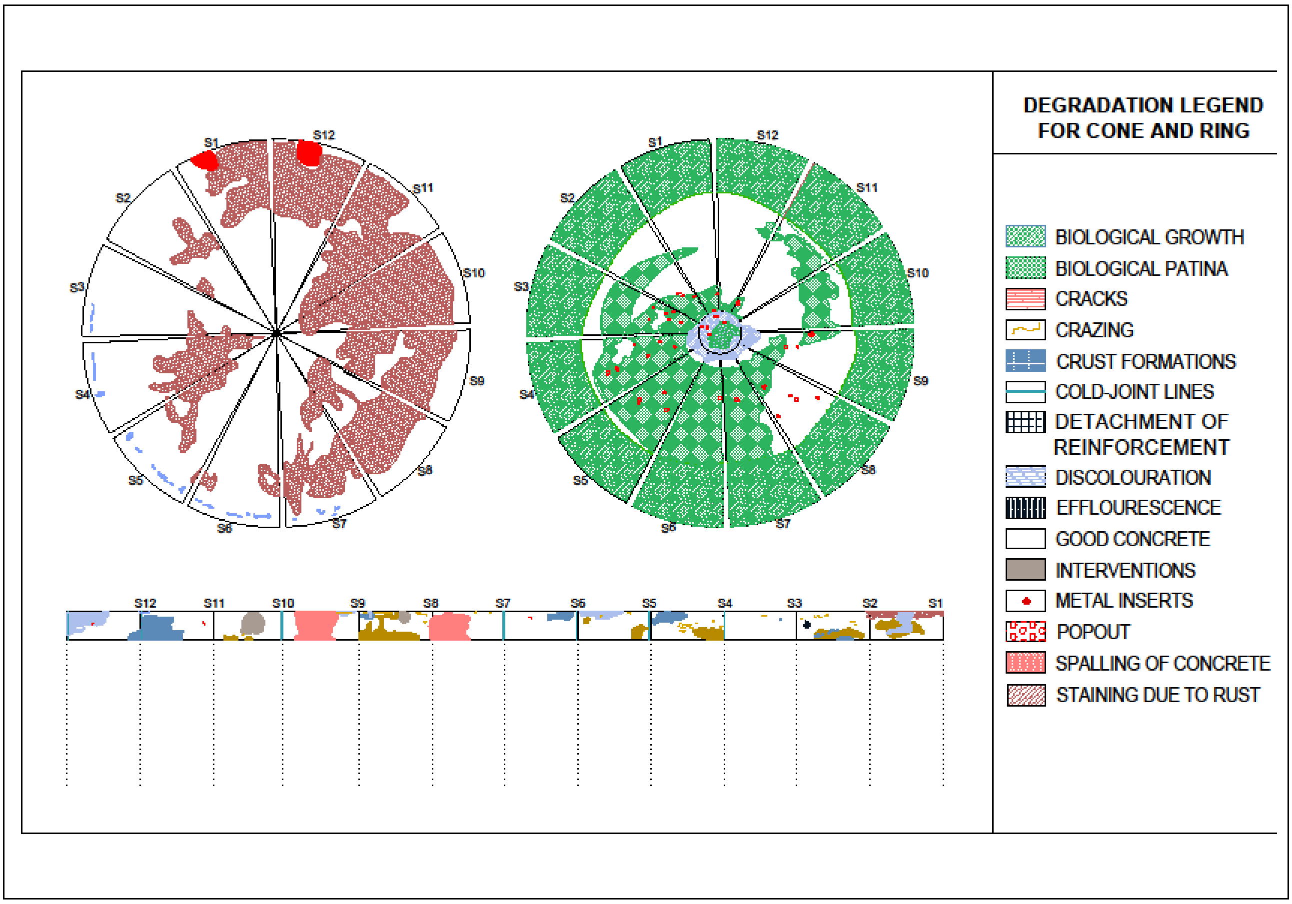


Figure 5.16. Mapping of Degradation in the Dome and ring beam (Reinforced Concrete Water Tower)

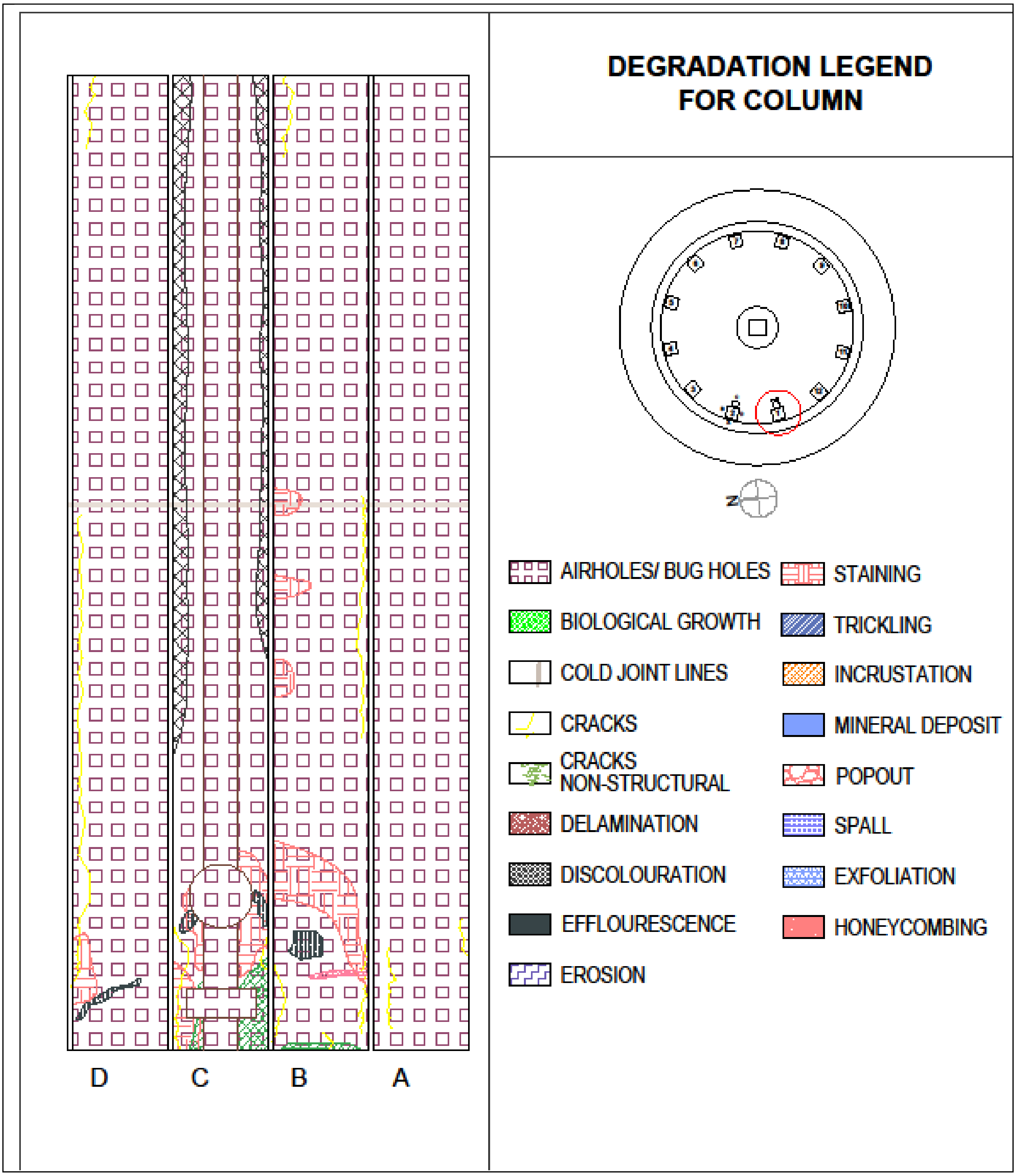


Figure 5.17. Mapping of degradation in a column (Reinforced Concrete Water Tower)

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Research and Innovation Action

GRANT AGREEMENT NUMBER — 760824

RESHEALIENCE

Rethinking coastal defence and Green-energy Service infrastructures through enHancEd-durAbiLity high-performance cement-based materials

D3.1 – Definition and description of the scenarios for WP8 pilots

Annex I. Detailed description of current pathologies case studies

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Related WP		3
Deliverable Title	Definition and description of the scenarios for WP8 pilots	
Deliverable Date		M6 (30 Jun 2018)
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Dissemination level		PU
Author(s)	Cristina Suesta and Lisardo Fort (CMW)	CMW
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Publishable summary and Introduction

This is the Annex I of Deliverable 3.1. This deliverable includes a detailed description of the different scenarios in which the pilots of Reshealience project will be built and operating. This description is addressed from geophysical point of view, taking into account weather conditions in case of outdoor pilots and chemical characteristics of the fluids that will contain, in the case of indoor pilots.

The document also contains the compilation of current practice in the field of each one of the involved partners, together with pathologies that can be associated with those current practices. This represents the starting point to define the requirements and characteristics of the UHDC that is going to be applied in the project, in order to improve durability and avoid the aforementioned problems in the service life of the intended types of structures.

This Annex provides additional information about pathology studies included in D3.1, in order to make them more understandable.

Finally, a pre-design of each one of the pilots of the project including main performance demands is detailed, including a general description, main sections, dimensions and concrete characteristics with manufacturing conditions.

Table of contents

1. Scope of the document

The goal of this document is to provide additional information about three pathologies studies included in Reshealience project deliverable 3.1 “ Definition and description of the scenarios for WP8 pilots”:

- Durability of marine concrete: docks case study (section 4.1.1 in D3.1)
- Concrete reinforcement corrosion in marine (Mediterranean) environment: submerged zone and intertidal zone (section 4.1.2 in D3.1)
- Concrete reinforcement corrosion in ocean environment worldwide: submerged zone and intertidal zone (section 4.1.3 in D3.1)

In D3.1 a detailed description of the scenarios in which the pilots of the project are going to be developed in order to establish current practices in those situations related to type of structures, concrete characteristics, evolution of the concrete properties with time and analysis the most common pathologies which can occur along the structure service life.

In order to compose this complex framework, an important collection and analysis of data have been carried out. The compiled data come not only from the literature but also, if not mainly, from industrial partners own experience. The final goal has been to find out which are the key durability factors depending on the application, which is determined at the same time by environment and type of structure.

Moreover, besides current practice data related to marine and industrial environments, several case studies have been analysed in order to investigate the main factors that contribute to pathologies appearance in each situation. Additional information of these case studies is include in this Annex for better understanding.

2. Case studies in marine environment.

2.1. Durability of marine concrete: docks case study [1].

Environment

All structures are located along the coast or in river estuaries and harbours of the South-western part of The Netherlands. Exposure class: XS

Type of structure

Six structures were selected for field investigations that were considered as representative of a larger group of structures. Selected criteria were age, cement type, production (cast in situ or prefabricated), availability and interest of the owner in the study. Some characteristics of the structures are described hereafter. For each of these structures, one to six test areas were investigated in detail.

Table 2.1. List of studied structures and main data

Structure	Year of Construction	Cement type	Production
Dock Scheveningen	1960	Portland cement, Blast furnace slag cement	Precast Cast in situ
Discharge sluice Haringvliet	1960	Blast furnace slag cement	Cast in situ
Quay wall Calandkanaal	1968	Blast furnace slag cement	Cast in situ
Quay wall Hartelhaven	1973	Blast furnace slag cement	Cast in situ
Quay wall Europahaven	1982	Blast furnace slag cement	Cast in situ
Eastern Scheldt Storm Surge Barrier	1980-1984	Blast furnace slag cement	Precast in field plant, cast in situ

Pathology description

Visual general inspection showed no major defects; observed pathology was mainly limited to some growth of marine living organisms and light erosion of the concrete surface; physical damage was present in a few of the quay walls. Corrosion related damage (rust staining, cracking, spalling) was observed in part of the precast slabs and in a part of the cast in situ concrete of Dock Scheveningen. In both areas, old repairs were present where corrosion and cracking had reappeared. It appeared that about 25% of the precast slabs had minimum cover depths of 20 to 25 mm, which was where corrosion related damage had occurred; slabs without visible corrosion had 30 to 35 mm cover depth (Table 1.2). The cast in situ deck with about 35 mm cover depth also showed extensive corrosion damage (cracking, spalling and ineffective repairs). None of the other structures showed visual signs of corrosion. This would be an indicator of better quality of precast concrete.

Carbonation depths were measured by spraying phenolphthalein in freshly made holes. Cores were taken for chloride profile analysis (six cores of ϕ 50 mm per test area), for polarizing and fluorescence microscopy (three of ϕ 50 mm) and for other tests (six of ϕ 100 mm), including strength testing.
Carbonation depths were low in all cases, typically about 2 mm with occasional values of 5 mm.

Concrete compressive strengths ranged from 50 to 75 MPa for blast furnace slag cement concrete and were about 120 MPa for Portland cement concrete (precast beams Dock Scheveningen). Polarizing and fluorescence microscopy (PFM) showed that all concretes were of good quality, with a quite homogeneous texture, as an indication of appropriate mixing of raw materials, compaction and curing. All samples had been made with Blast Furnace Slag cement with a high percentage of slag (>65%), except for precast concrete of Dock Scheveningen, which was made using only Ordinary Portland cement. The apparent water-to-cement ratios inferred from comparison of the capillary porosity were low, generally 0.45 or less. In some cases, historical documentation suggested that a w/c of 0.55 had been used. It appears that hydration, in particular of slag particles, of concrete exposed to marine environment over prolonged time is able to cause a substantial reduction of the capillary porosity.

In both the present fieldwork and other study on submerged specimens, the electrical resistivity of the concrete was measured using a four-point surface probe. Submerged OPC concrete specimens had resistivity ranging from 100 to 200 Ωm, precast OPC field concrete at Scheveningen had resistivity measured on site ranging from 300 to 500 Ωm. Concrete resistivity is very sensitive to moisture content. The increased resistivity suggests that OPC concrete in the splash zone in this investigation has dried out considerably, thus explaining the lower effective diffusion coefficients as compared to submerged OPC concrete. After 16 years submersion, blast furnace slag cement concrete had a resistivity ranging from 400 to 1000 Ωm. The present results from the splash zone were either 300 to 600 Ωm (Dock Scheveningen, Haringvliet low, Eastern Scheldt Barrier) or between 1300 and 3300 Ωm.

This tentative analysis suggests that drying out of Portland cement concrete, in the higher marine splash zone, may slow down chloride transport considerably as compared to transport in water saturated (submerged) concrete. Eventually, effective diffusion in Portland cement concrete may be about as slow as in (wet) Blast Furnace Slag cement concrete (Table 1.3). **Experiments from bibliography on saturated concrete in the laboratory and from natural submersion tests until now have indicated that chloride diffusion in slag cement is much slower than in Portland cement based materials**.

Characterization of concrete

In that area, all reinforcement was located using a scanning cover depth meter and the concrete surface was inspected visually.

Table 2.2. Cover thickness

Structure	Test area	Cover thickness (mm)	
		Average	Desviation
Dock Scheveningen	Precast slab (+ 7 m)	26 *	9 *
Dock Scheveningen	Cross beam (+5 m)	42.4	5.0
Dock Scheveningen	In situ (+ 5 m)	36.5	1.7
Haringvliet	Dock 11, low (+ 1 m)	71.1	4.9
Haringvliet	Dock 11, middle (+9m)	79.5	4.1
Haringvliet	Dock 11, high (+14m)	90.1	5.1
Calandkanaal	Quay wall (+1 m)	42.2	4.0
Hartelhaven	Quay wall (+1 m)	54.6	6.6
Europahaven	Quay wall (+1 m)	56.2	3.9
Eastern Scheldt Barrier	Dock Hammen 8 (+1m)	57.5	6.0
Eastern Scheldt Barrier	Upper beam (+4m)	69.1	2.5
Eastern Scheldt Barrier	Bridge element (+9m)	41.1	1.4

Cores for chloride analysis were cut in the laboratory by diamonds sawing in slices of about 10 mm thickness, which were dried, crushed and dissolved in hot nitric acid. Chloride was determined in the liquid using Volhard's titration. Chloride was expressed as percentage of cement mass assuming that all acid soluble mass was hardened cement paste (including 18% hydration water). This is justified, as virtually all aggregate was siliceous. The resulting chloride profiles are fitted to obtain an effective diffusion coefficient, using a commonly used least squares procedure based on the solution of Fick's second law of diffusion. Results are shown in Table 2.3.

Table 2.3. Chloride profiles

Structure	**Test area**	**Surface content (% chloride by mass of cement)**		**Diffusion coefficient (10^{-12} m^2/s)**	
		Average	Desviation	Average	Desviation
Dock Scheveningen	Precast slab (+ 7 m)	**3.5**	1.3	**0.14**	0.03
Dock Scheveningen	Cross beam (+5 m)	**2.6**	1.0	**0.28**	0.14
Dock Scheveningen	In situ (+ 5 m)	**3.2**	1.3	**0.33**	0.14
Haringvliet	Dock 11, low (+ 1 m)	**2.8**	2.0	**0.12**	0.04
Haringvliet	Dock 11, middle (+9m)	**0.4**	0.0	**0.14**	0.05
Haringvliet	Dock 11, high (+14m)	**0.7**	0.2	**0.10**	0.02
Calandkanaal	Quay wall (+1 m)	**3.9**	1.9	**0.19**	0.02
Hartelhaven	Quay wall (+1 m)	**2.9**	0.3	**0.12**	0.01
Europahaven	Quay wall (+1 m)	**3.9**	1.3	**0.12**	0.01
SVKO	Dock Hammen 8 (+1m)	**2.2**	0.6	**0.24**	0.07
SVKO	Upper beam (+4m)	**4.1**	0.3	**0.27**	0.06
SVKO	Bridge element (+9m)	**5.3**	1.5	**0.28**	0.12
Noordland	Quay wall (+1 m), 8 years	**1.8**	0.3	**0.84**	0.1
Noordland	Quay wall (+1 m), 18 years	**2.2**	0.1	**0.36**	0.0

Neglecting minor differences of w/c and age, **Portland cement concrete in the splash zone has a much lower effective chloride diffusion coefficient than in submerged exposure**.

2.2. Concrete reinforcement corrosion in marine environment: submerged zone and intertidal zone [2].

This study is specially focused on pathologies of concrete caissons in 7 ports of Spain. Samples from them were characterized from corrosion point of view. All these caissons had been executed properly.

Exposure environment

- Docks A, B; Mediterranean Sea, submerged
- Docks C: Mediterranean Sea, intertidal zone
- Dock D: Athlantic Ocean, intertidal zone
- Docks E, F, G: Cantabric Sea, intertidal zone

XS Environment, according to EHE Spanish Standars that is environment Qb, IIIc submerged marine environment, IIIb intertidal zone marine environment

Type of structure

Caissons in several docks in Spanish ports (Docks A, B, C, D, E, F, G). In Spain, the caissons are built with pontoons or floating dams, so they are in contact with sea water from a very early age. A caisson equivalent to the structure of a building of 8 storeys can be built in a week, by progressive sinking. In Spain docks are usually made with piles or caissons. In other countries, there are other common docks: sheet piles, metal piles, jackets.

- Dock A consists of 31 caissons of 25.33 m long and 17.0 m high each. The crowning of the caisson is only about 300 mm above sea level. Structural lightening by circular cells.
- Dock B is composed of 26 caissons of 42.25 m long and 16.5 m high each. The crowning of the caissons is only about 100 mm above sea level. Structural lightening by circular cells.
- Dock C consists of 18 caissons of 29.25 m in length and 13.5 m in height each, with a depth of 12 m. The type of lightening is circular. On the concrete caissons, a cantile beam was built in situ.
- Dock D consists of 10 caissons of 22.14 m in length and 14.0 m in height each, with a draft of 11 m. The type of lightening is square. A cantilever beam was built in situ on the concrete caissons. The tidal range here is 4.00 m.
- Dock E consists of 29 caissons of 31.28 m in length and 24.0 m in height each, with a depth of 20 m. The type of lightening is circular. In this Port, the tidal ranger zone is 4.50 m, but on the concrete caissons a cantilever beam was built in situ, which partially reduces this zone of influence of sea water.
- Dock F consists of 11 caissons of 28.70 m in length and 14.0 m in height each, with a draft of 10 m. The type of lightening is circular. A cantilever beam was built in the concrete crates on the sea side.
- Dock G consists of 20 caissons of 24.70 m in length and 15.7 m in height each, with a draft of 14 m. The type of lightening is square. In this port there is an inspection strip of 1.70 m above the level of B.M.V.E. The tidal range is 4.50 m, since this dock is in the same port as dock E.

In all the docks, the caissons were cast in situ; due to the construction procedure, the curing was carried out with seawater. The horizontal reinforcement is the one closest to the surface in contact with the sea, except in the case of dock E.

Table 2.4. Characterization of initial concrete

		Dock A	Dock B	Dock C	Dock D	Dock E	Dock F	Dock G
Age (years)		4,5	6,5	7,5	5	2	2	31
Location		Mediterranean	Mediterranean	Mediterranean	Atlantic	Cantabrian	Cantabrian	Cantabrian
Distance to B.M.V.E. of specimen /tide range (m)		- 0.1/0.8	- 0.1/0.4	+ 0.5/0.7	+ 2.5/4.0	+ 3.5/4.5	+2.0-3.5/5.4	+ 1.3/4.5
Exposure environment		Submerged	Submerged	Intertidal zone	Intertidal zone	Intertidal zone	Intertidal zone	Intertidal zone
Concrete (N/mm^2)		C25	C25	C25	C25	RC-30	RC-30	-
Cement (kg/m^3)		300	-	300	-	300	350	-
w/c		0.50	0.50*	0.55*	0.65*	0.50	0.50	0.60*
Coating (Project) (mm)		40	40	40	40	50	45	-
Type of cement		I/52,5-SR	-	I/45-SR/MR	-	IV/A 32,5 SR/MR	IIIB/32,5-SR	Pozzolanic
Type of steel		AEH-500S	AEH-500S	AEH-500S	AEH-500S	AEH-400S	AEH-500S	-
Horizontal reinforcement(**)		5016/m	5012/m	5012/m	4012/m	5016/m	5012/m	7.5018/m
Vertical reinforcement		7012/m	408/m	08 c30 cm	408/m	5016/m	408/m	4012/m
Strength (construction quality control) (N/mm^2)	**Different caissons**	A1-2 32.6	B1-2 29.8	C1 36.3	-	E1 31,4	F1 39,1	-
		A3-4 37.5	B3 28.7	C2 38.2	-	E2 33,6	F2 36,4	-
		A5-6 38.4	B4-5 26.8	C3 35.2	-	E3 37,2	F3 38,6	-
		-	-	-	-	E4 41.1	F4 42.2	-

* Calculated according to: c/a= 0,035 f_{cm} + 0,5

** Horizontal reinforcement is the closest to the surface in contact with water in all the docks except dock E.

In submerged concrete there is no corrosion until 30 years. In tidal range there is corrosion but not deterioration due to low coating or high penetration. In the three environments (air, tides and submerged) cement-based concrete with additions behaves better (lower water penetration and lower diffusion coefficient).

Pathology description

- **Specimens dock A**

In the specimens there are no cracks, joints, presence of foreign bodies, high porosity or any other notable anomaly, although coke cells are observed. The concrete has a uniform colour, and the aggregate and the paste are uniformly distributed. In no case corrosion of the bars has been observed. The concrete has been exposed to seawater in the submerged zone for 4,5 years and the theoretical cover is 40 mm.

- **Specimens dock**

In the specimens corresponding to the concrete of the caissons no cracks are observed, joints, presence of foreign bodies, high porosity or any other type of remarkable anomaly. Corrosion is observed in the bars that have been left outdoors after the exploration. The concrete has been exposed to sea water in the submerged zone for 6,5 years and the theoretical cover is 40 mm.

- **Dock C**

In general, the concrete of the caissons shows a good appearance. The most striking defect corresponds to the erosion suffered by the corners of the caissons. There are caissons that have lost the upper slab due to the action of the storms and that suffer a very severe deterioration in the exposed edge.

A generalized pathology is due to horizontal cracking in the upper part of the caisson, but in these cracks the corrosion symptoms of the reinforcement are not observed at first sight. Occasionally, visible corrosion processes are detected with the naked eye, which are associated with bars placed almost superficially and even externally, with considerably less than 4 cm of theoretical coating. The thickness of the concrete cover is quite variable and in some cases it is around 20 mm, even though the project indicated a minimum thickness of 40 mm.

Specimens

The reinforcements located at 30 and 35 mm depth showed signs of corrosion, not the one located at 150 mm depth, as a result of a lower chloride content. Concrete has been exposed to sea water for 7,5 years in a tidal range zone.

- **Specimens Dock D**

In the specimens there is no coke, cracks or any other notable anomaly. In no case there were symptoms of corrosion observed, although the studied bars are located deep inside the concrete. The concrete has been exposed to seawater for 5 years in a tidal zone.

- **Dock E**

The most striking defect corresponds to the erosion and/or breakage suffered in the corners of the caissons. The erosion occurs coinciding with projections with respect to the general face of the dock while the cases of breakage may be due to the striking of corners during the placement of the caissons.
Another pathology detected is the horizontal cracking and breaking of the upper edge of the

caissons, so that the reinforcement remains visible. The origin of this pathology can be found in the process of execution of the caissons, as it has been found in other ports, since the push of formworkb in the last part of the caisson damages the finishing of upper edge. No corrosion observed in the reinforcements that were discovered during the visual inspection.

It was carried out a study with ultrasound measurements, half potential cell and resistivity in different caissons of the dock.

Specimens
In the specimens there are no hollow concrete blockworks, cracks or any other notable anomaly. The reinforcements located caisson E4 showed no signs of corrosion but the coating is very high. Concrete has been exposed to seawater for 2 years in a tidal zone.

- **Dock F**

In general, the concrete of the caissons shows a good appearance. The most striking defect corresponds to the erosion suffered by the corners of the caissons, in most of them coinciding with projections with respect to the general face of the dock. Another pathology detected is the cracking and horizontal breakage of the upper part of the caisson 11, so that the reinforcement remains visible. In addition, efflorescence of no apparent importance have been found in several caissons, some surface finish defects.

The ultrasound pulse velocity is very high, which would allow to infer an excellent quality of concrete.

Specimens
In the Specimens there are no cokes, cracks or other notable anomaly. Some bars with signs of corrosion have been extracted.

- **Dock G**

The only repetitive defect corresponds to the erosion suffered by the corners of the caissons, although in this dock it has only been observed in 4 caissons. No appreciable corrosion processes have been detected with naked eye. The concrete has been exposed to sea water for 31 years in a tidal zone and with very small covers, so a generalized corrosion of the reinforcement closest to the face in contact with seawater is beginning. The same situation can be expected to occur in the reinforcement closest to the vertical lightening of the cage, which is filled with seawater, which is another source of chloride penetration.

Specimens characterization
Different concrete parameters were studied in specimens: permeability, porosity, absorption, Cl^- content at different depths and diffusion, carbonation depth.
The quality of the concrete was assessed according to obtained values and reference value in standards and bibliography.

In the following tables 1.5 and 1.6 some data are reported about the studied samples and according to these data a qualitative assessment is made in order to classify the concrete depending on several parameters. The point here is to highlight that the quality for the same concrete can be different depending on which parameter is chosen for the study.

It is observed that the classification criterion is not uniform for the different properties. Therefore, the porosity criterion establishes an intermediate quality of concrete with a fairly narrow range, while those of capillarity and absorption are excessively wide and even more the criterion of water permeability.

Depending on the environment in which the concrete is located (tidal or submerged), the relative quality limits associated with each of the tests will be different, since different transport mechanisms prevail in different environments. Besides, the addition of mineral additions also distorts these limits indicated by the CEB, since they have a different influence on some or other physical properties of the concrete.

Table 2.5. Characterization of the specimens

DOCK	AGE (years)	ENVIRONMENT	SPECIMEN	Ultra Sound Speed (m/s)	Comp. Strength N/mm^2)	A R. (%)	DENSITY (kg/m^3)
A	4.5	Submerged	A2	4419	44,3	36	2270
			A3	4581	48.7	30	2300
			A6	4529	44.6	16	2270
B	6.5	Submerged	B2	4.467	44.5	49	2290
			B5	4.439	42.8	60	2230
C	7.5	Intertidal zone	C1	4.500	40.0	10	2240
			C2	4.620	39.4	3	2260
			C3	4.650	38.3	9	2260
D	5	Intertidal zone	D1	4025	36.4	-	2209
			D2	3860	-	-	2150
			D3	4000	33.4	-	2190
			D4	3670	24.0	-	2160
E	2	Intertidal zone	E1	5093	38.1	22	2273
			E2	5000	47.2	40	2300
			E3	4810	54.5	47	2290
			E4	4910	31.8	- 23	2270
F	2	Intertidal zone	F1	4670	34.6	- 12	2250
			F2	4910	50.5	39	2300
			F3	4770	29.3	- 24	2280
			F4	5093	-	-	2303
G	31	Intertidal zone	G1	4840	-	-	2280
			G2	5080	31.9	-	2260
			G3	4920	-	-	2270
			G4	4672	28.1	-	2242
			G5	4510	-	-	2240

Table 2.6. Qualitative assessment of the concrete

Dock	Age	Caisson	Rc MPa	UltraSound	Porosity	Permeability	Absorption	Capilarity
A	4,5	A1-A2	44.3	Good-excellent	Bad	Medium	Medium	Medium
		A3-A4	48.7	Excellent	Bad	Medium	Medium-bad	Medium
		A5-A6	44.6	Excellent	Bad-medium	Medium	Medium	Medium
B	6,5	B1-B2	44.5	Good-excellent	Medium	-	Medium	Medium
		B4-B5	42.8	Good-excellent	Bad	Medium	Bad	Medium-bad
C	7,5	C1	40.0	Good-excellent	Good	-	Medium	Medium-bad
		C2	39.4	Excellent	Good	Medium-good	Medium	-
		C3	38.3	Excellent	Good	-	Medium	Medium-bad
D	5	D1	36.4	Good	Bad	Medium	Medium	Bad
		D2	-	Good	Bad	Medium	Medium	Bad
		D3	33.4	Good	Medium	Medium	Medium	-
		D4	24.0	Good-acceptable	Bad	Medium	Medium	-
E	2	E1	38.1	Excellent	Bad	Good	Medium	Medium
		E2	47.2	Excellent	Bad	Good	Good	Good
		E3	54.5	Excellent	Bad	Medium-good	Medium	Medium
		E4	31.8	Excellent	Bad	Medium-good	Medium	Medium-good
F	2	F1	34.6	Excellent	Bad	Medium	Medium	Medium
		F2	50.5	Excellent	Medium	Medium-good	Medium	Medium
		F3	29.3	Excellent	Medium	Medium	Medium	-
		F4	-	Excellent	Medium	Medium	Medium	Medium-good
G	31	G1	-	Excellent	Bad	Medium	Bad	-
		G2	31.9	Excellent	Bad	-	Bad	Medium
		G3	-	Excellent	Bad	-	Medium	Bad
		G4	28.1	Excellent	Bad	Medium	Bad	Medium
		G5	-	Good-excellent	-	-	Bad	-

From the profile of Cl^- content at different depths, the Diffusion coefficient is obtained (Table 1.7). D1 is the coefficient at 1 year to compare the values between the different specimens. The concretes of the docks A, B, C and D has been designed following the prescriptions of the EH-91, and they have also been manufactured using Portland cement without additions. In the four, very high chlorides diffusion coefficients are obtained at 1 year, which means that a high chloride penetration rate is achieved. In docks E, F and G, on the other hand, lower diffusion coefficients at 1 year are obtained, which implies a lower penetration rate of chlorides. Concretes employed for docks F and G contain slag and very similar diffusion coefficients are obtained, although one was built 2 years ago and the other 31 years ago; concrete of dock E contains fly ash and is the one with the lowest penetration rate of chlorides.

Table 2.7. Qualitative assessment of the concrete

DOCK	SPECIMEN	AGE (years)	DIFFUSION D ($*10^{-12}$ m^2/s)	DIFFUSION D1 ($*10^{-12}$ m^2/s)
A	A1-A2	4,5	22.76	48.3
	A3-A4	4.5	30.61	64.9
	A5-A6	4.5	21.92	46.5
B	B1-B2	6.5	17.37	44.3
	B3	6.5	17.02	43.4
	B4-B5	6.5	23.51	59.9
	C1	7.5	14.60	40.0
C	C2	7.5	5.03	13.8
	C3	7.5	16.00	43.8
	D1	5	34.20	76.5
D	D2	5	37.80	84.5
	D3	5	35.90	80.3
	D4	5	27.60	61.7
	E1	2	8.42	11.9
E	E2	2	1.90	2.7
	E3	2	1.53	2.2
	E4	2	2.09	3.0
	F1	2	5.40	7.6
F	F2	2	3.56	5.0
	F3	2	3.63	5.1
	F4	2	3.69	5.2
	G1	31	1.25	7.0
G	G3	31	0.91	5.1
	G4	31	1.18	6.6
	G5	31	1.11	6.2

In the Table 2.8 more data are reported about chloride content. The chloride content on the surface depends on the salt content of the surrounding water, the exposure time and the type of concrete (in terms of content and type of cement and w/c ratio). In this case, the Mediterranean Sea has a concentration of salts higher than that of the Atlantic Ocean and the Cantabrian Sea, but the difference is small, so that the data from the seven wharves investigated can be treated in a homogeneous manner.

Due to the influence of the exposure time, it is observed that the chloride content in the maximum measured surface of the Dock G concretes (0.5107% by weight of concrete as the average value of the five caissons, after 31 years of exposure) It is similar to that of Dock F (0.4454%, after 2 years of exposure to chlorides), so it can be considered that the content of chlorides on the surface does not evolve indefinitely over time, but from a content of chlorides in equilibrium with that of the surrounding seawater, increases to a maximum value in a short time (at most two years, according to the data of these two docks compared) and remains constant thereafter, except for possible fluctuations due to the effect of chloride washing in concrete in tidal zone.

For the analysed docks located in the tidal range, measured chloride concentrations in the first portion of the cover resulted between 0.20 and 0.62% by weight of concrete. As for the two docks located in the submerged zone, Dock A presents an average concentration of surface chlorides of 0.2367% by weight of concrete.

Several authors indicate values for surface chloride concentration between 0.36-0.77% respect concrete weight, while the results from other authors in real structures and from this study reveal that for tidal range zone that concentration is between 0.2-0.7%.

Considering tha analysis of al these data, it can be concluded that the surface chloride content in the long term that it can be used in the secon Fick law ecuation for calculating diffusion coefficient, adopts a conservative value of around 0.7% by weight of concrete, with slight variations depending on the dosage of the concrete, for the Spanish peninsular ports (submerged marine environment and tidal zone).

- Maximum Cl^- concentration

  The critical chloride content (expressed as total chlorides Table 2.9) depends on the type of cement and the marine environment. As for the type of cement, cements with low C_3A content (therefore, with lower chloride binding capacity) have lower critical contents, so that the most conservative results would correspond to Portland SR, MR cements (always used in structures in contact with seawater) or cements with high addition contents.

  With respect to the influence of the marine environment, this is a determining factor in the chloride threshold. Therefore, it is observed that in the concretes located in the submerged zone (docks A and B), the reinforcements show no signs of corrosion, although the chloride contents at this depth vary between 1.34 and 1.81% by weight of cement. Therefore, in a submerged marine environment, the critical chloride content must be at least more than 1.8% by weight of cement, for AEH-500S steels and SR type cements.

  In the other docks, where the analysed concrete is located in the tidal zone, reinforcement without corrosion with chloride content in the concrete of up to 1.06% by weight of cement has been found (so the value of 0.4% indicated in the regulations could be considered conservative in this environment), and reinforcements with corrosion with chloride content in concrete between 1.31 and 4.65% by weight of cement.

In conclusion, the results indicate conservative corrosion thresholds for the chloride content equal to of 1.8% and 1% (expressed in cement weight) in submerged and tidal range environments, respectively.

Table 2.8. Chloride content in specimens

DOCK	SPECIMEN	√(Cs – *Cb)*	Cb	Cs theoric	Cs measured	Cs measured (% cement)
A	A1-A2	0.5463	0.0107	0.2932	-	-
	A3-A4	0.4938	0.0150	0.1802	-	-
	A5-A6	0.2744	0.0170	0.0881	-	-
B	B1-B2	0.6410	0.0957	0.3257	-	-
	B3	0.4638	-	0.6498	-	-
	B4-B5	0.5791	0.1094	0.4566	-	-
C	C1	0.6484	0.0439	0.4643	0.4842	3.62
	C2	0.7327	0.0223	0.5591	0.4807	3.62
	C3	0.6141	0.0420	0.4191	0.4944	3.72
D	D1	0.5697	0.0034	0.3280	0.3258*	2.40
	D2	0.4410	0.0035	0.1980	0.2034	1.46
	D3	0.5894	0.0214	0.3688	0.4343	3.17
	D4	0.5145	0.0209	0.2856	0.2471	1.78
E	E1	0.6817	0.0102	0.4749	0.3577	2.71
	E2	0.9332	0.0090	0.8799	0.5945	4.56
	E3	0.8677	0.0076	0.7605	0.5064	3.87
	E4	0.7295	0.0101	0.5423	0.4195	3.17
F	F1	0.8321	0.0066	0.6990	0.6152	3.95
	F2	0.7182	0.0083	0.5241	0.3483	2.29
	F3	0.7941	0.0085	0.6391	0.5271	3.43
	F4	0.6345	0.0072	0.4098	0.2910	1.91
G	G1	0.7466	0.0293	0.5867	0.5010*	-
	G2	-	-	-	0.6183*	-
	G3	0.9494	0	0.9014	0.6244*	-
	G4	0.7788	0	0.6065	0.3604	-
	G5	0.8935	0	0.7983	0.4493*	-

CT = total chloride content, Cf = free chlorides, Cb = bound chlorides

Table 2.9. Critical Cl^- content (% weight cement)

DOCK	AGE	ENVIRONMENT	CEMENT	STEEL	SPECIMEN	Cx (%)	CORROSION
A	4.5	Submerged	I/52,5 SR	AEH-500S	A1-A2 A3-A4 A5-A6	2.05* 1.81* 1.49*	No No
B	6.5	Submerged	-	AEH-500	B1-B2 B3 B4-B5	1.34* - 1.24*	No - -
C	7.5	Intertidal zone	I/45 SR/MR	AEH-500S	C1 C2 C3	1.79 1.68 1.43	Yes Yes
D	5	Intertidal zone		AEH-500S	D1 D2 D3 D4	1.59 1.12 1.06 1.36	- - No
E	2	Intertidal zone	IV/A 32,5 SR/MR	AEH-400S	E1 E2 E3 E4	1.04 0.07 0.06 0.22	- - No No
F	2	Intertidal zone	IIIB/32,5 SR	AEH-500S	F1 F2 F3 F4	0.96 0.05 0.15 0.09	No No No
G	31	Intertidal zone	Pozzolanic (Slags)	- Smooth bars	G1 G2 G3 G4 G5	2.64 4.65 3.27 2.49 1.31	Yes Yes Yes Yes Yes

* Maximum concentration of Cl- estimated by extrapolation of Cl- profile and supposing cover of 4 cm.

For each environment and type of cement in Table 1.10 are shown the w/c ratios can be considered the initial limit to achieve an adequate impermeability of the concrete to the penetration of chlorides, and the diffusion coefficient with reference to one year period. These w/c ratios should be considered as a first upper limit, above which the concretes thus dosed turn out to be critically permeable to the penetration of chlorides.

Table 2.10. w/c ratio and Di for each environment and type of cement

Environment	Type of cement	w/c ratio	Di (x 10^{-12} m^2/s)
Intertidal zone	Withoud blenders	0.50	7.34
	Silica fume	0.70	10.90
	Fly ash	0.70	11.14
Submerged	Withoud blenders	0.45	16.06
	Silica fume	0.55	11.57

Conclusions

- **There is a necessity to apply sulphate resistant cements in marine environments. Execution of works is very important to avoid permeable concretes.**

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- **The control of the strength does not guarantee the durability**: In almost all the caissons studied, the resistance to compression of the extracted specimens was above 30 N/mm^2, and in some cases it was even higher than 50 N/mm^2. However, despite these good resistance values, the tests carried out to evaluate the impermeability of the concrete denote in general a medium or even bad quality.

- **The favourable effect of the additions on the durability is very important**. Two of the docks built with cement with additions (E and F docks) are in tidal zone, although the concrete dosage used did not meet the requirements of minimum cement content and maximum water/cement ratio required in this environment. However, these concretes did meet the specifications of water penetration in both cases, due to the beneficial effect of the additions on the permeability. The presence of C_3A is less determinant than the compactness of the concrete

- There is a good correlation between penetration and chloride diffusion coefficient. **Chloride content influence (surface and critical) on corrosion is not clear and it is different for aerial, tidal or submerged zone.**

- The critical content of chloride in submerged concrete could very superior to that indicated by the Spanish standard (0.4%) because of oxygen does not penetrate inside, so corrosion does not start even with greater concentrations of Cl^-.

The critical contents of chlorides collected in the literature vary between 0.17 and 2.2%, expressed as % Cl^- total weight of cement. The following table shows the critical content of chlorides (by weight of cement) that causes the onset of corrosion collected by different authors:

Table 2.11. Critical chloride content from bibliography

	Soluble in water	Soluble in acid
ACI 201	0.10 a 0.15*	-
ACI 222	-	0.20
ACI 318	0.15 a 0.30	0.20
BS 8110	-	0.40
Australian Standards	-	0.60
RILEM	-	0.40
Norway Standards	-	0.60
Hope e Ip	-	0.10 a 0.20
Everett y Treadaway	-	0.40
Thomas	-	0.50
Hussain, Al-Gahtani y	-	0.18 a 1.2
Page y Havdahl	0.54	1.00
Stratfull	-	0.15

2.3. Concrete reinforcement corrosion in ocean marine environment worldwide: submerged zone and intertidal zone [3].

This section describes corrosion examples in concrete exposed to marine environment all around the world compiled from bibliography. It includes several types of elements and environments but all exposed to XS environment.

Type of structure
The section compiles data from different structures exposed to marine environment in different countries, providing data about the structure, the original concrete that was applied and the pathologies and concrete characterization after a period of time. There is a comparative table very useful to find out current problems in this kind of environment and find out most common causes.

- **Portugal.**

Dams in Sado River.
Size 450x75 m and height 18 m.
Bad quality execution: improper compaction and curing, segregation, joints...
Marine environment in warm weather (water 16-21 g/L Cl^-).
Long dryness periods encouraged O_2 availability for corrosion.

Pathology description:
First problems appeared after 5 years in service in internal walls (detachment of cover). The loss of section in the bars was not very high in these walls, the main effect of the corrosion being the detachment of the concrete cover. No macro corrosion cells were observed, which are normally associated with attack by chlorides and very high levels of corrosion.

The north wall of this dam showed higher deterioration than the east and west walls. The north wall is more exposed to the sun than the other walls, so it is exposed to higher temperatures, which leads to a faster drying of the concrete, and therefore, a greater access of oxygen to the reinforcement. In addition, the temperature significantly increases the corrosion rate.

Dock in Sado River
Six pre-stressed concrete precast beams, 300 mm thick slab made in situ. Hollow piles founded on sea floor. Concrete cover: 30 mm
Dimensions 120 x 200 m, width 20 m. Concrete slab 30 MPa, concrete piles 35 MPa
Exposure conditions:
- Slab, upper beams: splash zone
- Lower beams, upper piles: intertidal
- Lower piles. Submerged

Pathology description:
The electrical resistivity of the field to measure in the zones and the upper part of the beams is between 5 and 15 kΩ cm, while the lower part of the beams and in the piles is below 5 kΩ cm. Using a silver/silver chloride electrode, results below -250 mV were measured at all test points, indicating a high probability of corrosion in the reinforcements. In the tidal zone, very low potential values were obtained (-450 to -550 mV), which has been associated to situations in which the corrosion rate is limited by the access to oxygen to the reinforcement. The splash zones were very deteriorated, with large surfaces where the concrete coating had come off. The tidal zone presented less damage.

On the edge of the docks there was a substantial loss of section in the reinforcements. In some areas the 25 mm diameter bars had been completely corroded, after 24 years of exposure, which means a corrosion rate of 500 μm/year. The corrosion is of macrocell mechanism of corrosion by chlorides, with large cathodic zones and small anodic zones.

Bridge in Sado River
25 m slab, 12 beams of 21.2 m
Concrete is never in direct contact with seawater. The central part of the bridge is more exposed to wind, and therefore to salt dragged by the humidity and the foam of sea. There had been a significant detachment of the coating in the beams of the board (especially on the faces downstream and in the lower corners).

Pathology description:
The main cause of the damage was poor execution, since the quality of the concrete was good. Gravel clumps and poorly constructed construction joints were frequent, and the thickness of the coating was quite variable, less than 20 mm in many points of the beams.

High values of the electrical resistivity of the concrete in the beams of the edge were measured. Normally these measures exceeded 40 kΩ cm, and only in some areas were 30 kΩ cm. The section loss of the reinforcement observed in the zones with detachment of the coating is small. This is due to the high resistivity of the concrete, which prevents the formation of corrosion-resistant macro-cells. The corrosion rate is controlled by the electrolytic process.

- **USA**

Rocky Point Viaduct
It is a continuous bridge of 5 spans and 114 m long, with a reinforced concrete board, built in 1954. It was replaced in 1994 due to the serious structural deterioration resulting from the loss of section of the bars and the deterioration of the concrete caused by the corrosion. The Viaduct was located in one of the most corrosive microclimates of the Oregon coast, 25 m from the ocean and at a height of 35 m. The west face of the beam was fully exposed to the ocean, while the east face was on the land side, so it was protected by the board.

Pathology description:
After 40 years of service, the bars and shear fences were embedded in a concrete with a chloride content well above the critical (7 and 8 kg Cl^-/m^3 of concrete, respectively). A severe environment and insufficient coating of the shear abutments are responsible for the premature corrosion damage of the viaduct.

Brush Creek Bridge
It is a bridge of 3 spans, 244 m long and continuous reinforced concrete board, built in 1954 and replaced in 1998, due to a severe structural deterioration caused by corrosion. The microclimate was not as severe as in the previous bridge. The bridge was less exposed, located 244 m from the ocean and at a height of 16 m. However, due to the terrain conditions, which accelerated the breezes, a large amount of salt came to the bridge from the mists and saline foams.

Pathology description:

Much of the concrete coating on the underside of the board had come loose, exposing the bottom grid of the frame and accelerating corrosion. In addition, the concrete cover of the lower part of many of the beams had been detached, exposing the reinforcements and the shear abutments, some of which had been completely oxidized. The damage was greater along the lower wing of the beams closest to the ocean, where the salt foam that drained through the beams core was collected. The load-deformation curve of the A4 beam was measured, presenting a loss of strength of 24%, with respect to the design capacity. The failure mode was by plastic deformation of the main reinforcement.

The concentration of chlorides in the upper grid of reinforcement was below the critical content of chlorides, while the lower concentration was well above (between 2.5 and 5 kg Cl/m^3 of concrete), thus providing the opportunity for development a corrosion macrocells if the grills are connected and made electrically continuous. In the beams, surface concentrations were recorded that varied between 7 and 9 kg Cl/m^3 on the land side, and between 10 and 14.76 kg Cl/m^3 on the ocean side. The cover thickness was 25.5 mm. This very small thickness, the severe environment and the macrocell corrosion potential explain the severe corrosion damage.

- **Sidney dock**

The dock, built in 1966, is a structure formed by a slab and beams supported on circular concrete piles and with a rectangular service pipe located under the board.

Pathology description:
Most of the detected corrosion damage occurred on the underside of the dock and in the splash zone. At points where corrosion was due to insufficient cover, oxidation started in the fences and from there extended to the main frame, which was protected by a larger cover. Subsequently, the detachment of the concrete cover occurred. Chloride contents greater than 0.5% by weight of cement were found at depths between 60 and 80 mm in 4 of the 6 perforations made, and at depths equal to 68, 70 and 87 mm in the controls. These high levels of chlorides were not expected, due to the low moisture absorption, the high density and the high compressive strength of the concrete at these points.

- **Japan**

Caisson at Muroran Port
The breakwater is composed of reinforced concrete caissons, with a reinforcement amount of 0.33%, height of 7.9 m, with the upper level located 0,3 m above the tide line. The caissons were filled with concrete and a 1.3 to 2.7 m thick layer was concreted on them. Caisson made with slag and fly ash concrete.

Pathology description:
Minimum corrosion was detected.

Dam at Muroran Port
It is a dam of dimensions of 55 x 350 m, and a height of between 11.55 and 12.75 m. The internal surfaces of the walls are exposed to contact with salt water once a week, which is when the dam is filled. The external surfaces are underground and in contact with the sand filling, the water table being located 2,5 m below the surface of the land. The internal surfaces are considered to be located in a very aggressive environment, while the external ones are in a moderately aggressive environment, due to the almost complete saturation of the concrete pores and the difficulty of oxygen access.

The upper part of the walls is not in contact with water usually, so it is subject to significant variations in humidity. It is a situation similar to the splash zone, which is the worst situation in a marine environment. The northern wall is subject to greater solar radiation than the rest, which dries earlier and allows easier access of oxygen, which makes its environment more aggressive.

Pathology description:
After 10 years in service, the gallery and the walls (formed by a slab 0.35 m thick, supported on buttresses every 4 m) of the dam showed sheet separation of the concrete and exposure of a bar, due to corrosion.

The electrical potential on the outer surface of the walls (between the water table and the ground surface) was -89 to -306 mV (measured with a silver/silver chloride electrode), which indicates a medium/high probability of corrosion. In the internal walls, -616 to -667 mV (with copper electrode/copper sulphate) were measured, which indicates a high probability of corrosion.

The petrographic analysis showed a very heterogeneous matrix texture, due to bad mixing. In most of the samples a very dense surface layer of brucite/calcite had formed, which protected the interior concrete. The depths of carbonation were very small (0.5-3 mm).

- **Scotland**

Tay Road Bridge (Dundee)
The Tay Road bridge was built between 1963 and 1966, and consists of 42 spans supported on pairs of reinforced concrete piles between 5.5 and 30 m high; its length is 2.245 m and it crosses the Tay Estuary. The premature development of the corrosion of the reinforcement of the lower part of the piles has led to the installation of a cathodic protection system. Between 1985 and 1993 a series of investigations on the penetration of chlorides in the piles was carried out. The bridge is directly exposed to the waves and the wind of the North Sea, located to the east. The seawater within the Estuary contains the same salinity as the North Sea, which has been taken as 19 ppm of chlorides. The temperature in Dundee is cold, being the annual average of 8 °C. The pairs of reinforced concrete piles rest on concrete basement, at the level of the high level of tides.

Pathology description:
The leaks through the board joints of the flux salts applied on the road have contributed to the chloride content at the top of the piles. Severe corrosion deterioration was discovered in the area of splashes in the lower part of the piles, with part of the coating skipping; it was decided then to apply to this zone a cathodic protection and the beginning of a study of the state of the piles.

- **Comparison between international case studies**

Structure/element		Resist. (MPa)	Age (years)	Cover (mm)	w/c ratio	Quality execution	Resistivity (kficm)	Permeab. (m/s x10^{-11})	Porosity (%)	Corrosion	D1. (x10^{-8} cm^2/s)	Environment	Carbonat. (mm)	Cl- content		
														Surface	Bars	Bars*
Dam 20 Portugal	Walls	17.6	5	40	0.7	Bad	4 - 8	14	17.8	Very serious[f]	70.9 - 31.8	2 cycles/year	10 - 25	0.2 - 0.7 [a]	0.14 - 0.40 [a]	1.07 - 3.06
	Inferior slab	20.6	5	60	0.7	Bad	-	0.6	16.4	Serious[f]	-	2 cycles/year	10 - 25	-	-	-
Dam21 Portugal	Walls	17.6	5	40	0.7	Bad	4 - 8	14	17.8	Very serious[f]	248 - 378	2 cycles/month	10 - 25	0.18-0.22[a]	0.15 - 0.19 [a]	1.15 - 1.45
	Inferior slab	20.6	5	60	0.7	Bad	< 1.5	0.6	16.4	Serious[f]	-	2 cycles/month	10 - 25	-	-	-
Dam 22 Portugal	Walls	17.6	5	40	0.7	Bad	1.5 - 3	14	17.8	Very serious[f]	-	2 cycles/month	10 - 25	-	-	-
	Inferior slab	20.6	5	60	0.7	Bad	< 1.5	0.6	16.4	Serious[f]	-	2 cycles/month	10 - 25	-	-	-
Dock Portugal	Piles	-	24	-	-	-	< 5	-	-	-	-	Tidal-submerged	-	0.24 [a]	0.18 - 0.13 [a]	1.38 - 0.99
	Beams	35	24	30 - 50	-	-	5-15 < 5	-	-	Very serious[f] - Serious[h]	34.3- 1.695	Splash – tidal	-	0.18 [a]	0.14 - 0.17 [a]	1.07 - 1.30
	Slab	30	24	30 - 50	-	-	5 - 15	-	-	Very serious[f.g]	18.1 - 15.7	Splash	-	0.10 [a]	0.07 - 0.05 [a]	0.54 -0.38
Bridge Portugal		50	35	< 20	0.32	Bad	> 40	10	-	Media[f]	0.7 - 1.2	Air	-	0.16-0.34[a]	0.02 -0.09[a,c]	0.15 - 0.65
Viaduct	Land	52.8	40	49.6	0.53	-	-	-	10.6	replaced[1]	62.8	Air	-	5.26 [b]	4 [b]	1.33

Rocky Point USA	Ocean side	52.8	40	49.6	0.53	-	-	-	10.6	replaced [1]	48.8	Air	-	9.58 [b]	7 [b]	2.33
Bridge Brush Creek USA		-	30	25.5	-	-	-	-	-	replaced [1]	22.5 - 57.0	Air	-	4.5 - 7 [b]	3.5 - 6 [b]	1.17 - 2
Dam Muroran (Japan)		34	66	75-175	0.73	-	-	-	16	Minimum[i]	12.2 - 64.2	Tidal-submerged	0	3.8 [b]	1.4 [b]	0.75
Sidney Dock (Australia)		40	20	40	0.53	Good	2.4	-	-	Medium[f]	-	Splash	5 - 15	-	0.10 -0.50[d/e]	0.1 - 0.5
Bridge Tay Road (United Kingdom, Scotland)		22-68	25	75	0.44-0.66	Bad	-	-	-	Serious[f]	10.9-33.3	Splash	-	0.6-2.5[e]	0.15-0.89 [e]	0.15-0.89
Reinforced concrete Dam (Japan)		18-36	20	40	0.6-0.7	Bad	1.5-3	-	17-20	Serious[f]	42.9-20.1	1 Cycle/week	10	0.38-0.20 [a]	0.30-0.14 [a]	2.3-1.1

a = % Cl-/weight concrete
b = kg Cl-/m^3 concrete
c = at 25 mm
d = at 50 - 70 mm
e = % Cl-/weight cement
f = cracking and covering detachment. Section loss in bars is not very high
g = coorrosion macrocells, high section loss.
h = black or green corrosion
i = undamaged concrete, weight loss in bars up to 1.11%
* = % Cl-/weight cement, estimated 300 kg of cement in 2300 kg of concrete, Muroran Port 186 kg.

Conclusions

The chloride content on the surface of the concrete, which is a fundamental data to be able to determine the diffusion coefficient of chloride, is not clearly defined, since very different values are found in the surveyed literature. The same happens with the critical content of chlorides that it is supposed to begin the corrosion. The international standard is not unanimous with respect to the requirements adopted for the dosage of concrete located in the marine environment; the water/cement ratio, the cement content, the cover thickness and the maximum allowable chloride content vary from one to the other.

3. References.

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