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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

D3.2 – Definition of key durability parameters for each scenario

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Publishable summary

The report aims to justify durability Key Performance Indicators (KPI) and their target values to be achieved by the design of UHDCs for environmental scenarios XS and XA, taking into account the exposure conditions, the type of structure and the required application (new/retrofitting).

XS and XA scenarios are identified and described in relation to the specific exposure conditions of the pilots. The KPIs are described and defined accordingly to the Ultra High Durability Concrete (UHDC) material requirements and structural design in Durability Assessment-based Design (DAD) for long-term durability performance.

The development of KPIs moves from the current information of the concrete structures built in marine and in chemical aggressive environments in relation with the practical experience and uses the knowledge available in the literature in relation to concrete properties and compositions and the requirements provided by international standards and recommendations.

Since this information is referred in general to conventional concrete and in very few cases to High Performance Concrete (HPC), KPI definition takes into account differences between these concretes and UHDC for ReSHEALience project.

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List of acronyms, abbreviations and definitions

Acronym table

Agg	aggregate
BFS	Blast Furnace Slags
bwc	By weight of cement
CEB	Euro International Committee du Beton
DAD	Durability Assessment-based Design
D_{app}	apparent diffusion coefficient
D_{nssm}	non-steady-state migration coefficient
ECC	Engineered cement composites
FA	Fly Ash
FHWA	Federal Highway Administration of U.S
FRC	Fibre-reinforced concrete
GA	Grant Agreement
HPC	High Performance Concrete
HPCSF	High performance concrete with silica fume
HPFRC	High performance fibre reinforced concrete
HPFRCC	High performance fibre reinforced cement composites
HSC	High-strength concrete
ITZ	Interfacial transition zone
KPI	Key Performance Indicators
NAD	National Application Document
OC	Ordinary concrete
OPC	Ordinary Portland Cement
PDI	Pre-delivery inspection
Ppf	polipropylene fibres
RC	Reinforced Concrete
RPC	reactive powder concrete
SCC	Self-compacting concrete/Self consolidating concrete
SCI	Steel concrete interface
SCM	Supplementary cementitious material
SF	Silica Fume
SO	Specific Objective
TAC	Technical Activities Committee
UHDC	Ultra High Durability Concrete
UHPC	Ultra High Performance Concrete
UHPFRC	Ultra high performance fibre reinforced concrete
UHSC	Ultra High strength concrete
VHPC	Very high performance concrete
VHPFRC	Very high performance fibre reinforced concrete
X_{VR}	Corrosion depth

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1. Scope of the document

The scope of the document is based on the general assumption that any concrete structure is submitted simultaneously to structural and environmental actions. It has to support mechanical actions, but also it has to be durable all along its service life. Each concrete characteristic within the structure can follow different performance with respect its interaction under the intended and anticipated actions.

Durability of a concrete structure can be endangered depending on the mechanism and consequence of the environmental leading parameters, originating from a mechanical cause, such as abrasion, or physical, such as freeze and thaw, or chemical, as due to chloride penetration, carbonation, or in expansion due to sulphate penetration and reaction or even in a combination of actions.

This Deliverable 3.2 is a report justifying durability Key Performance Indicators (KPI) and their target values to be achieved by the design of Ultra High Durability Concretes (UHDCs) for different scenarios related to:

1) XS: marine environment and
2) XA: chemical attack.

The influence in durability performance aspects under consideration are:

1) the characteristic exposure conditions,
2) the type of structure and
3) the required application for new structures or retrofitting.

In the document, XS and XA scenarios are identified and described and for each one, the KPIs are described, to be considered for the UHDC and structural design in a Durability Assessment-based Design (DAD) procedure.

The development of KPIs takes place from the current information of the concrete structures built in these scenarios (data collected in D3.1), the analysis of the literature with reference to main durability indicators and concrete characteristics and the considerations of the requirements provided by international Standards and Recommendations.

2. Input data from D3.1

In this chapter, the scenarios XA and XS are described and analysed from the current practice of ReSHEALience partners and from literature review, as well as from the objective that KPIs must reach.

The practice experience for XS and XA application related to the main concrete components and characteristics are analysed. Current pathologies in these type of structures are described with respect to the damage. Besides, description of pilots in relation to the expected exposure conditions, type of structure and required application are described for the main characteristics of the environmental exposure class, location and basic general description of the pilots.

2.1. Current partners concrete practice data

➢ **<u>Concrete components and requirements</u>**

Main concrete characteristics from industrial partners current practice were described in D3.1 and summarized here in Table 1, to have a more comprehensive understanding of the information obtained. These characteristics are related to concrete structures for maritime works, in which cast in situ and precast concrete elements are exposed to the marine environment (offshore and onshore), and for concrete employed in geothermal energy infrastructures that have to support chemical aggressive environments in operation conditions. The typology of infrastructures considered refers to those type selected for the pilots.

Table 1. *Background from partners*

	XS Environment					XA Environment
Parameter	**Offshore platforms (HPC general)**	**Offshore floating caisson**	**Docks (from partners practice and literature)**	**Precast Element for marine application**	**Aquaculture Precast element**	**Cooling concrete Basins for geothermal power plants**
Rc/ MPa	C40-C85	C40	HA25-HA30	C40-C60	C140-150	C25-45
w/c	<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 (binder)/ kg/m^3	380-500	360	300-350	425-450	975-1175	360
Max. Size agg/ mm	10-14 (basalt, granite)	22	-	14	-	-
Steel fibre/ kg/m^3	-	-	-	-	80-160	-
Consistency	18-22	18	-	-	-	-
Water Permeability (m/s)	-	$10^{-12} – 10^{-8}$		-	-	-
Water Penetration (mm)			30-50			
Total chlorides %bwc	-	<0.3	-	-	-	-
Type of cement	-	OPC+ SF, BFS, FA	-	CEM I + GGBS	CEM I42.5 R/SR + SF	-

Main conclusions from current practice of the ReSHEALience industry partners in concrete constructions:

- Concrete classes are in a normal range for general maritime works exposed to marine environment (caissons) and industrial works exposed to chemical aggressive environments. For offshore platforms and precast elements exposed to marine environment the category related to compression resistance is between 40 and 60 MPa, but always in the range of a normal reinforced concrete (RC). Ultra High Performance Concrete (UHPC) manufacturing as a recent current practice is only performed in aquaculture works.
- Water/cement ratios are around 0.4 for normal concretes (RC) and around 0.2 for UHPC.
- Cement content in the concrete is never below 350 kg/m^3 and higher for higher resistance categories, reaching close to 1000kg/m^3 for UHPC application.
- Pozzolanic mineral additions, as silica fume (SF) and Blast Furnace Slags (BFS) and Fly Ash (FA), are frequently used in concrete mixes to improve durability and reduce water permeability.
- Concrete cover is around 40-50 mm for normal concretes (RC). For marine precast elements is even higher, even above 70 mm and it is reduced to 20 mm in the case of UHPC.

➢ **Current pathologies**

Complete studies on reinforced concrete pathologies are included in D3.1, but a general outline is incorporated in this section to provide the most important conclusions about them.

According to pathology studies included in D3.1, from pilot partners experience, typical pathologies in structures under marine conditions, XS, include excessive and premature corrosion of reinforcements, accompanied by cracking and spalling of concrete cover. This kind of pathologies can appear at any time, sometimes 5 to 10 years after construction while the structures have been designed for a service life of 50 or 100 years. Besides the reinforcement corrosion, other mechanisms of degradation contribute to concrete damage, as the precipitation of solid phases inside the pores due to the penetration of ions such as Cl^- or $SO_4^=$ and a continuous dissolution and recrystallization due to water drying process. In some cases, it has been observed in marine environment the growth of organisms on the surface of the concrete or deposition of faeces of seagulls that induce also a chemical attack of the concrete and loss of cover.

Typical pathologies in structures under chemical aggressive environments, XA, including cracking produced by expansion in the case of sulphate attack, cement paste dissolution and efflorescence, in the case of acid attacks, the dissolution of cement paste. With reference to the pathologies observed in the chemical industrial environment in concrete basins containing water or muds coming from geothermal power cooling towers, the main cause of degradation is the ageing of waterproofing products applied to the walls and to bottom of the basins, due also to the action of geothermal water. The acid and ion content in the water in this case induce an attack and dissolution from the concrete surface and also expansions in concrete due to the formation of expansive solid phases inside the pores due to sulphate attack, as gypsum or secondary ettringite due to the high sulphate ions content of the waters and together with other anions as chloride induced corrosion of steel in reinforced zones.

Although the differences that have been highlighted above hold between marine and industrial chemical aggressive environments, as far as the deterioration mechanisms are concerned, both scenarios and sub-scenarios will have a combination of actions, and structures subjected to them will benefit from the use of Ultra High Durability Concretes. The pilots to be considered in ReSHEALience project will also address mechanical actions due to fluid pressure or water abrasion erosion actions.

2.2. Description of pilots: exposure conditions, type of structure and required application

The choice of the type and location of the pilots stands is an innovative concept itself. Pilot locations, the main concept addressed and owner type are detailed in Table 2Table 2 and Table 3. The pre-design of pilots and the specific physic-chemical exposure conditions of each pilot have been detailed in Deliverable 3.1. Further information about the pilots is included in this section, regarding class/subclass of exposure, the identification of geographical location where the pilots are going to be placed in the case that is not an isolated element, and a basic description of the type of structure.

Table 2. *Summary of pilots. Main characteristics*

Pilot responsibility	Pilot Location	Pilot Concept	Image
EGP	Tuscany Power Plant	Two pilots: Water basin /Mud basin Concrete basin of 7.0 m x 20.0 m, divided into 2 or 3 sectors by means of internal reinforced walls.	

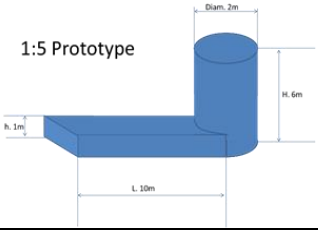
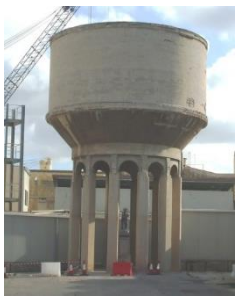
RDC	Valencia Coast (Port)	Floating structure with a size of 20 m x 27 m, composed of beams with a section of 23 x 23 cm and others with a section of 35 cm (width) x 23 cm (depth) Mussel farming raft;	
CMW	Spanish Mediterranean Coast (Castellon Port, Valencia Port, Cartagena Port)	Platform for offshore wind turbines, prototype of 6 m height, 10 m arm length, cylinder of diameter 2 m	
BANAGHER	West Irish Coast (Galway)	6 x 3 m breakwater elements, about 1,2m deep for no-tipping will be produced	
University of MALTA	Marsa (Malta)	Height of tower: 13.12m Drum diameter: 9.45m Textile reinforcement and highly flowable UHDC will be used to restore the tank and the support columns respectively.	

Table 3. *Pilots scenario identification, main degradation and failure modes*

Pilot number	Pilot responsibility	Pilot type	Environment	Main degradations	Failure modes
1	EGP	Geothermal power plant cooling tower basin	**XA**: Concrete under acid attack and sulphate attack Addit.: erosion	Leaching of $Ca(OH)_2$, CSH attack, new phases formation, pH decrease, loss of mass, elastic modulus, compression strength decrease.	Cover loss due to loss of material. Mechanical loss. Volume change due to expansive phases. Abrasion by water movement.
2	EGP	Drilling platform mud collection basin	**XA**: Concrete under acid attack and sulphate attack	Leaching of $Ca(OH)_2$, CSH attack, new phases formation, pH decrease, loss of mass, elastic modulus, compression strength decrease.	Cover loss due to loss of material. Volume change due to expansive phases. Mechanical loss. Abrasion by solid particles in suspension

3	RDC	Aquaculture mussel raft	Main: **XS3** Addit.: **XA** Cl intrusion / organic attack faeces	Chlorides transport and reinforcement corrosion/ possible concrete attack marine organism.	Spalling/abrasion and scaling due to reinforcement corrosion/ loss of material at the surface.
4	CMW	Offshore wind floater	**XS1/XS2/XS3** Saltspray & Cl intrusion/ocean water soft temperature	Chlorides transport and reinforcement corrosion.	Spalling/wind abrasion and scaling due to reinforcement corrosion.
5	Banagher	Floating pontoon	**XS1/XS2/XS3**: Salts pray &Cl intrusion/ocean cool water temperature	Chlorides transport and reinforcement corrosion/freeze and thaw.	Spalling/abrasion and scaling due to reinforcement corrosion.
6	UoM	Damaged water tower	**XS1:** Aerial marine	Chlorides transport and reinforcement corrosion/Wet drying/carbonation.	Spalling/abrasion and scaling due to reinforcement corrosion.

2.2.1. Pilots addressing concrete chemical attack: XA

➢ **Main Characteristics of XA environments**

XA exposure conditions occur when concrete is exposed to chemical attack from natural soils and groundwater, industrial activities as well as rain in a polluted environment. Concrete will perform satisfactorily when exposed to most waters and soils containing chemicals, since solid or dry chemicals rarely attack concrete. However, some chemical environments may severely impair the service life of the concrete. To significantly attack concrete, aggressive chemicals should be in solution and above specific threshold concentrations. As ACI 201-2R13 reports, in the case of external sulphate once penetrated into the concrete porous structure may induce attack of the cementing materials. If evaporation takes place, the sulphate ions can precipitate near that surface and increase even further the degradation potential. Depending on the nature and concentration of the chemicals, the XA exposure class is divided into three sub-classes, described in Table 4:

Table 4. *Subclasses of XA exposure conditions*

XA1	Environment with low chemical aggressiveness
XA2	Environment with moderate chemical aggressiveness
XA3	Environment with high chemical aggressiveness

EN 206-112 provides recommendations for threshold values of different aggressive agents for each XA level, even though the final requirements are given in National Application Documents (NADs). Typically, structures in contact with geothermal fluids experience XA3 exposure, mainly due to the pH, high $SO_4^{=}$, Cl and CO_2 contents of the condensate waters from the wells. Carbonation and corrosion of concrete by geothermal fluids containing carbon dioxide are quicker than in air, due to the acidification, which may lead to severe attacks within just a few months. The temperatures of the surrounding fluids may reach even 300ºC and pressures of 50 bars.

➢ **Main characteristics of pilots in XA environments**

- **<u>Mud and water basin in a geothermal plant</u>**

The partner responsible is EGP, two type of pilots for this environment are under consideration: 1) a water basin and 2) a mud basin.

<u>Subclass:</u>

In case of XA conditions, both pilots in geothermal context fall into the XA3 category, highly aggressive environment although several durability aggressive parameters can be actuating as consequence of the specific chemical composition of the water interacting and the way of interaction, as it will be addressed later.

<u>Location:</u>

The facilities where the pilot is going to be located are described below. So far, a specific plant/ drilling pad has not been identified.

- Water basin of Cooling Tower in geothermal power plants, located in Tuscany. Operating conditions of geothermal plants are quite similar in terms of fluid composition and weather conditions, being part of the same geothermal area.
- Mud basin of a geothermal drilling pad, located in Tuscany. Operating conditions of geothermal drilling pads are quite similar in terms of water/ mud composition and weather conditions, being part of the same geothermal area and using the same drilling technologies

<u>Description of the structure:</u>

- The case of the mud basin is divided into two sub-basin, due to operating needs, different types of concrete and/or design could be applied.
- The case of the cooling tower, experimental activities concern the water-collecting basin, which will be divided into three sections: different types of concrete and/or structure could be applied.

2.2.2. Pilots addressing marine environments: XS

➢ ***Main* Characteristics *of XS environments***

In XS group environments the main risk of damage is the corrosion of the reinforcement due to the penetration of sea water chlorides.

This exposure occurs where and when concrete containing reinforcement or other embedded metals is in contact with chlorides from seawater or air carrying salts originating from seawater. Maritime structures suffer different actions, including mechanical stresses originated by sea movements due to currents and wind, and chemical attacks, which depend on the position of the structure in the sea and/or with respect to it. There are three different environmental exposures as far as the consequences of the corrosion, induced by the sea water, are concerned, as also shown in Figure 1:

- XS1 for structures near to the coast exposed to airborne salt,
- XS2 for submerged parts of marine structures,
- XS3 for tidal, splash and spray zones for semi-immersed parts of marine structures where oxygen can feed the reinforcement corrosion more effectively than in the corresponding submerged structures.

Marine structures with aerial and submerged parts will contemporary experience the three of them. Maritime structures could be submitted to different environmental actions. On the one hand, there are many physical stresses originated by sea movements due to currents, wind, etc. and on the other hand, there are chemical attacks, which depend on the position of the structure with respect to sea.

According to EN-1992-1-1, structures near the sea, up to 500 m from the coastline, and also elements located above the high tide level, are referred to as XS1. The permanently submerged areas of the structures are in a XS2 environment. Finally, elements between high tide and low tide level, which is the tidal range, are considered in a XS3 environment. Splash zone is also included in this environment.

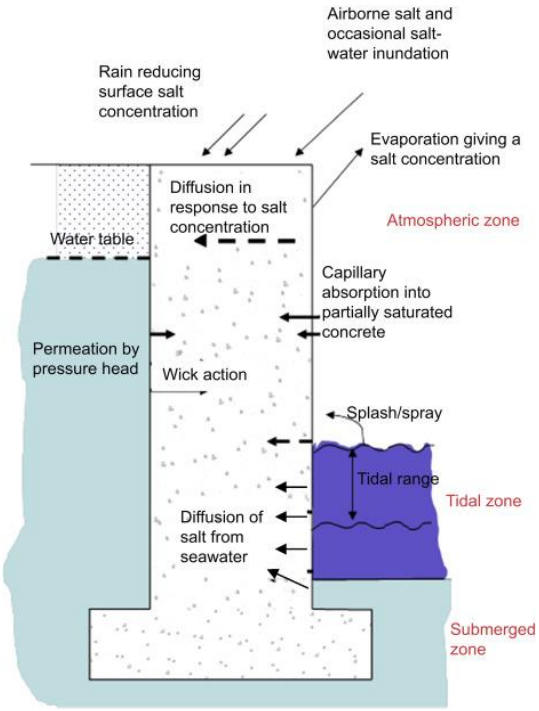


Figure 1. *Subclasses of XS exposure conditions*

Other environmental factors may significantly affect the service life of these structures as well, e.g. the temperature. Low temperatures such as those experienced in the Northern Sea and North Atlantic region may also include freeze and thaw actions, whereas higher temperatures are faced by structures in Southern Europe seas. In addition, differences in marine salinity (lower in Northern seas, higher in closed Southern seas, such as the Mediterranean) can also affect the environmental degradation and hence the service life of these structures. These differences between the different scenarios for ReSHEALience pilots were described in Deliverable 3.1.

In fact, ACI201.2R states that between different seas, the concentration of salts is lower in the colder temperate regions than in the warmer seas and is especially high in shallow coastal areas with high daily evaporation rates. These combinations and specific conditions are not considered yet in the standards.

A more detailed description of scenario/location than in Deliverable 3.1 for each one of the pilots is provided in this document.

➢ ***Main* characteristics *of pilots in XS***

- **<u>Valencia coast Aquaculture mussel raft</u>**

The partner responsible of this pilot is RDC. This pilot represents the aquaculture sector. A floating structure for mussel farm raft will be constructed with different types of UHDC and UHPC and different levels of prestressing. Figure 2 and ***Figure 3*** show pictures of the structure.

<u>Subclasses</u>

In Figure 2, the section of a UHPC raft during the process of floating is shown. There are UHPC beams at three different levels. All of them are exposed to environment XS3, as they are in frequent contact with seawater but not permanently submerged, as the maximum buoyancy accepted is the provided by the blue steel floaters.

Besides, some parts of the raft, as the external perimeter and the bottom face suffer an abrasion due to the constant impact of the waves (Figure 3-left) that can be considered XM1 or XM2 according to the levels indicated in the standard NF P18-710.

Figure 2. *RDC previous construction with a similar structure to ReSHEALience pilot*

Finally, the upper surface of the beams is the perfect rest area for the seagulls, whose faeces have a pH between 4 and 4.5 (acid), so an acid attack of the concrete can be also expected as damage. ***Figure 3*** below shows the seagulls sleeping on the raft and the white stains of their faeces on the surface. This implies that the upper part of the raft is also submitted to an exposure class XA3.

Location

The floating farm will be located in Valencia port, and most likely it will be in the coordinates: 39°26'4.71"N 0°18'11.47"W, in the location indicated in the map of Figure 4. The goal is to have it accessible from the port for monitoring of performance, but also submitted to the significant impact of the waves. This dock of the port is named Dique del Este (Eastern dock). The average depth in this part of the port is -11.5 m, as required to have at least 10 m of ropes to hang the mussels. The distance of the raft to the closest land will be approximately 70 m.

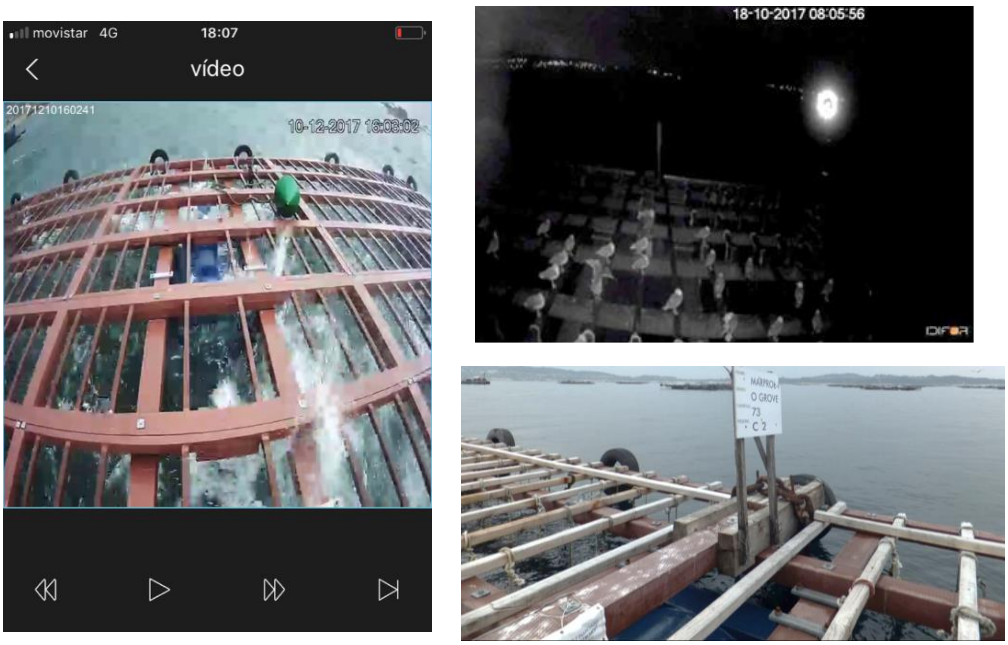


Figure 3. *RDC previous construction with similar structure to ResHEALience pilot. Impact of waves (left), seagulls sleeping (right-up) and seagulls faeces (right-down).*

Figure 4. *Mussel farm pilot planned location*

Description of whole structure

The UHDC raft designed by RDC will be a frame of beams made of UHDC, but in order to have the required buoyancy it needs 4 to 6 steel floaters reinforced with polyester, and it needs between one and four dead weighs, depending on the climatologic conditions and the region, in order to set it in a fixed position. An example of floaters used for a UHPC oyster raft floating in Valencia port is shown in

Figure 5. The connection between the concrete beams is made with bolts through high density polyethylene elements integrated in the precast beams, as shown in Figure 6. This type of connection provides the required flexibility to the whole element, reduces the stresses in the concrete D-region around the connection and also protects the bolts from the corrosion of the sea water.

Figure 5. *RDC previous work: floaters for an UHPC oyster raft in Valencia Port*

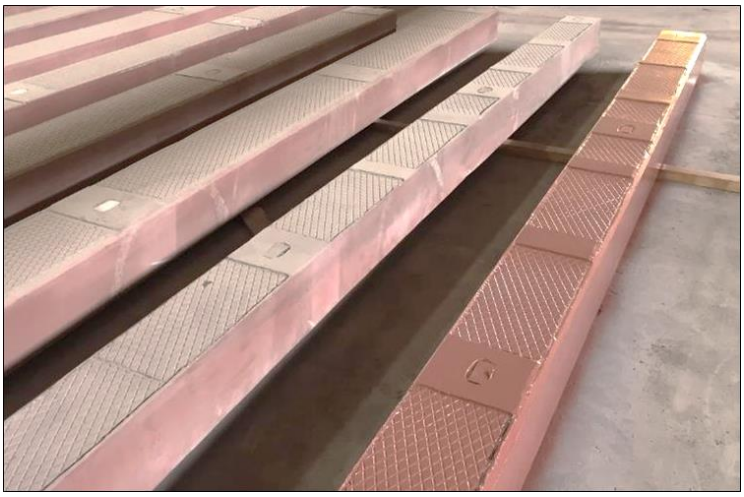


Figure 6. *Connection system used for the beam of the raft*

- **<u>Valencia coast Offshore wind floater</u>**

The partner responsible of this pilot is CMW+Ener Ocean. The type of structure represents an offshore element, as shown in Figure 7.

<u>Subclasses</u>

The offshore platform will have a submerged part and an aerial part so three XS subclasses will be found in it, as indicated in Figure 7.

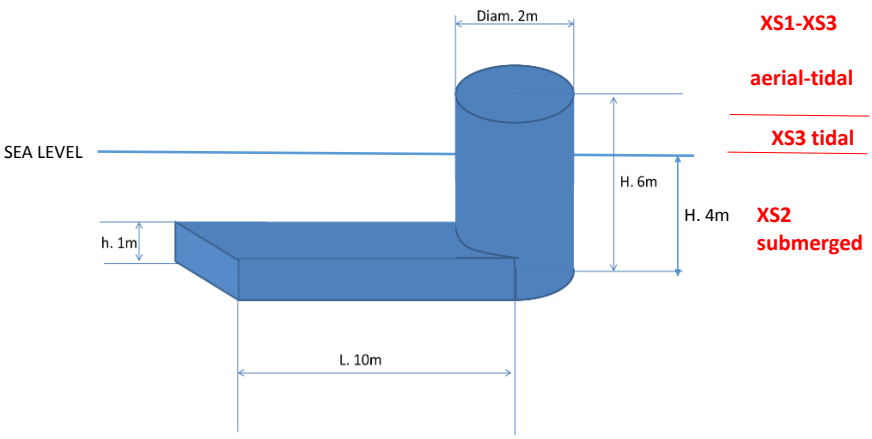


Figure 7. *Exposure subclasses in CMW pilot*

<u>Location</u>

There are three possible Mediterranean locations for the CMW pilot. The final decision will depend on the works in progress when the pilot building operations will start and the possibility of managing the corresponding permit with each Port Authority.

The distance to coast won't be high since, due to operational reasons, the pilot should be built near the workplace, not longer than 25 m, but the final exact location will depend on the Port Authority permission. The three possible locations in the Mediterranean are described below and shown in Figure 8, Figure 9 and Figure 10.

- Castellón Port: It is a work of north dam wall. Coordinates are around 40, 0,02 and water depth in that zone is 12m, shown in Figure 8.

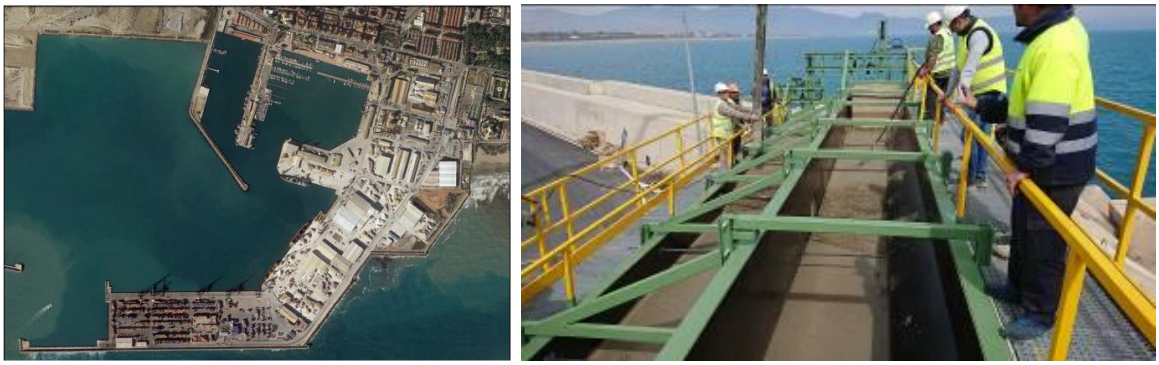


Figure 8. *CMW works in Castellón Port (location and dam wall)*

- Valencia Port. It is a work of a dock of piles for the execution of a new installation of P.D.I. (Pre-delivery inspection) of vehicles in Valencia Port. The works are located on a surface of 40.000 m^2 in the Eastern Dam. Water depth is between 14-16 m approximately. Shown in Figure 9.

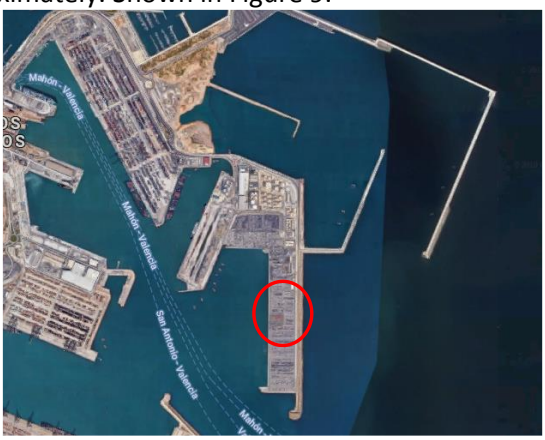


Figure 9. *Location of CMW works in Valencia Port*

- Cartagena Port. It is work of the Southwest dam wall reinforced with glass fibre instead of steel fibre in order to avoid corrosion. Coordinates of the pilot could be around 37, -0,9. Water depth in this dam is around 16 m. Figure 10

Figure 10. *CMW works in Cartagena Port (location and dam wall)*

<u>*Description of whole structure*</u>

The pilot will consist of a single element not belonging to a larger structure and it will be located offshore, close to the works that CMW will be developing in one of the locations described above. So the scenario is properly defined by means of weather conditions (described in D3.1) and geographical location.

- **Floating pontoon in Atlantic Irish West coast**

The partner responsible of this pilot is Banagher Prec. The aspect of the pilot structure is shown in Figure 11.

Subclasses

The pilot will have a submerged part and an aerial part so the three XS subclasses will be found in it.

Location

It will depend on permits. The Atlantic Ocean near to Irish West Coast is the proposal at this moment. Management for permits are in progress at this moment and it is not possible to provide more specific information.

Description of whole structure

The pilot will consist of 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.

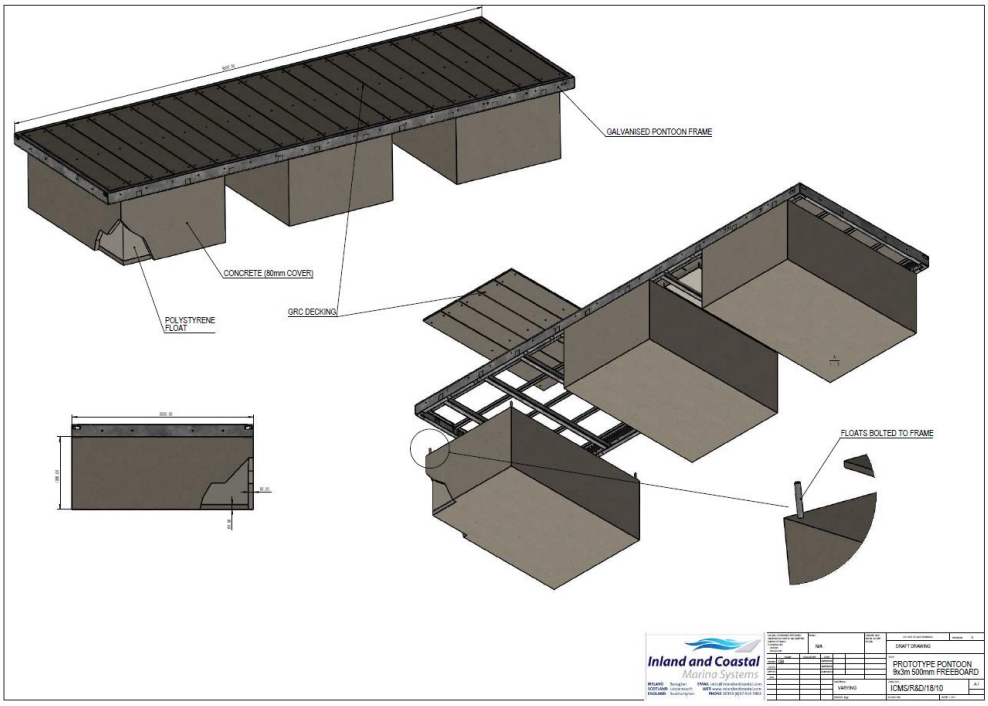


Figure 11. *Banagher pilot proposal*

- **Public abattoir Damaged water tower**

The partner responsible of this pilot is the University of Malta. The structure of this pilot will be repaired due to problems of corrosion of reinforcements. The aspects of the structure is in Figure 12 and Figure 13.

Subclasses

Based on environmental conditions at the water tower, the following possible exposure subclasses have been identified:

- XC3 (corrosion induced by carbonation) – due to humid climate in Malta
- XS1 (corrosion induced by Cl from sea water) – due to the proximity of water tower to the coast (~90 m)

Location

The water tower is located within the precincts of the Civic Abattoir in Marsa, Malta. The coordinates are 35.877450 N, 14.498556 E, and the coast is ~90 m away.

Description of whole structure

This 1930s reinforced concrete water tower shows severe degradation, mainly on the side exposed to the sea, in particular, due to the action of a chloride-rich environment. UHDC textile reinforcement and highly flowable UHDC will be used to restore the tank and the support columns respectively.

Figure 12. *Reinforced Concrete Water Tower, Malta*

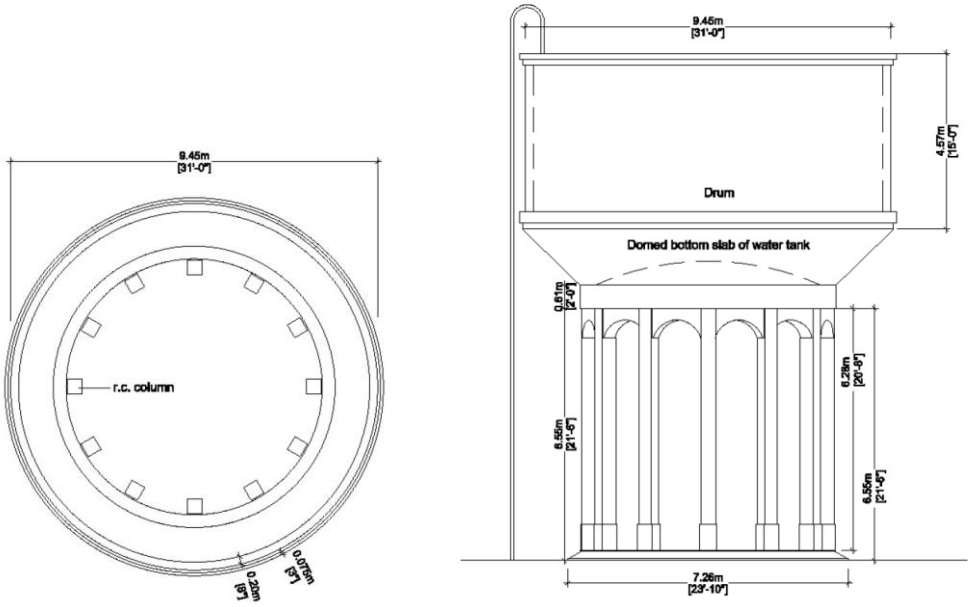


Figure 13. *Reinforced Concrete Water Tower (Plan and Elevation)*

3. Durability and innovation in concrete. From components to structures

3.1. Concrete technology evolution

The technology of concrete is in a continuous evolution and relevant advances and innovations are being developed. However, the distance between real practice and laboratory is significant. The scale-up of new technologies is needed. The most common types of concrete is referred to in Table 5. The main differences are highlighted.

Table 5. *Concrete types and technologies*

Norm or Recommendation		EC	Annex	Annex	Annex							FR		FR	JP	EEUU
International or country where the name is used		international														
		OC	FRC	SCC	HSC	UHSC	HPC	HPFRC	VHPC	VHPFRC	UHPC	UHPFRC	UHDC	RPC	HPFRCC	ECC
Components	Fibres															
	Selected aggregates													fine	fine	fine
Rheology	Slump > 500 mm															
Mechanical properties	50 Mpa < CS < 120 Mpa															
	120 Mpa < CS < 150 Mpa															
	CS > 130 Mpa															
	CS > 150 Mpa											*				
	Dir Tensile Strength > 7 Mpa											**				
	multimicrocracking															
	no multicracking															
Durability	high durability															
	Extreme durability															
Curing	Steam curing															

Mandatory to be qualified as

No mandatory, but generally accepted by the majority

Also possible

Concepts underused

Most common concepts

* According to FR Norm. In the Swiss is 120 Mpa

** According to FR Norm

- **Ordinary Concrete (OC)**

Ordinary Concrete as composite material contains coarse and fine aggregate integrated in the cement paste. Many structures have been built using ordinary concrete technology. OC requires vibration during casting to favour the form filling and a throughout homogeneous distribution of its constituents. The compressive strength generally ranges from 20MPa (low strength and low quality concrete for structural application) to 50MPa. The w/c ratio is usually above 0.5 and the content of cement between 250 to 350kg/m^3 [Mehta, 2006]. The durability of ordinary concrete can be enhanced incorporating mineral additions as fly ash (FA) or blast furnace slags (BFS) to its mix-composition.

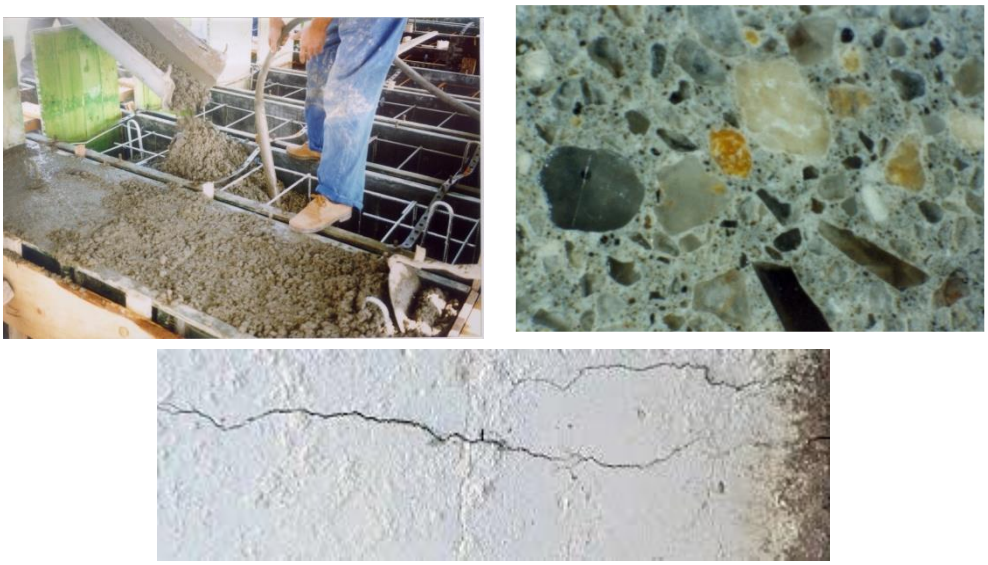


Figure 14. *OC production, vibration (left), distribution of aggregates in paste (right) and crack failure (down)*

The structures made of reinforced concrete generally contain cracks when they are under service conditions, because of the low concrete tensile strength. The steel bars are the main elements providing tensile capacity in the cracked state, which is the normal service state for reinforced concrete structures. In most of the concrete standards, the maximum crack width under service is the parameter used to guarantee the durability of the structure under certain exposure environment all along the target lifespan.

- **Fibre-reinforced concrete (FRC)**

FRC is a composite material consisting of a concrete matrix containing discontinuous, discrete and uniformly dispersed fibre reinforcement. Workability classes for ordinary concrete can also be adopted for FRC [Prisco, M. di et al, 2009]. The compressive strength of the concrete is not particularly influenced by the fibres.

The main characteristic of FRC is the enhancement of its fracture toughness depending on the type and dosage of fibres added.

High modulus or low modulus fibres are used for different applications.

High modulus fibres, such as steel or some types of polymer fibres, are generally employed with the aim of replacing the conventional reinforcement, Low modulus fibres, as short size polypropylene fibres, are used to reduce shrinkage cracking and to enhance fire resistance. The FRC property is activated once the crack is initiated, which usually starts from first days of the concrete life, as in the case of shrinkage cracks, or after loads and other actions are applied to the structure [Prisco, M. di et al, 2009].

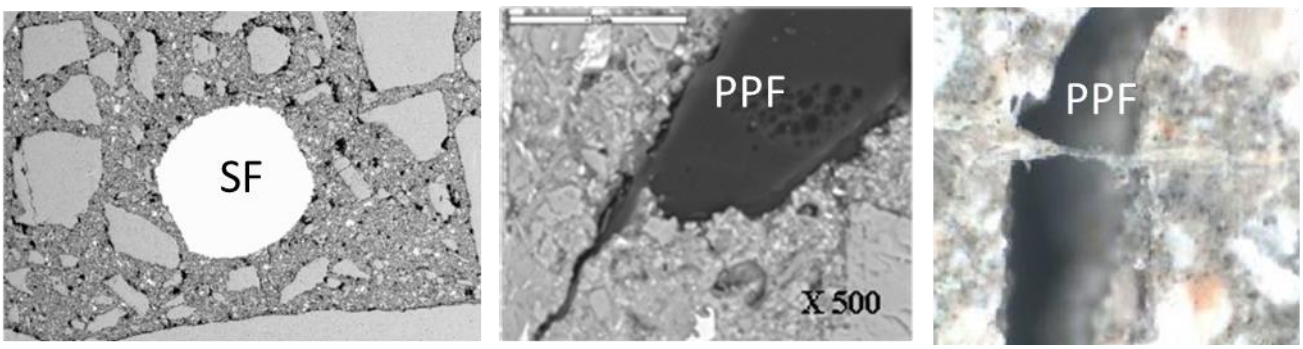


Figure 15. *FRC appearance: steel fibre (left) and polypropylene fibre (middle) interface with concrete and fibre bridge of a crack (right) [Alonso, MC et al; 2013]*

- **Self-compacting concrete (SCC)**

Also known as Self-Consolidating Concrete, it is able to flow under its own weight and completely fill the formwork, while maintaining homogeneity even in the presence of congested reinforcement, and then consolidate without the need for vibrating compaction.

- **High-strength concrete (HSC)**

Until the last decade of the 20th century, engineers were concerned almost exclusively with concrete compressive strength as the relevant parameter to measure its quality. Then, the concept of High-Strength Concrete (HSC) was used to allude to these concretes with significantly higher compressive strength than that of typical mixtures. Obviously, the border value that defined an HSC increased with the time. In 1950, a concrete with 35 MP) was considered HSC, while nowadays design codes, such as ACI 318-9, are working to implement designs for strength levels up to 125 MPa.

The compressive strength of this type of concrete usually exceeds 50MPa with a maximum of 100MPa [Mindess, S. 2008]. Having a significantly higher compressive strength than the ordinary concrete, the material has a low porosity and a high density which means higher strength, but also an increased brittleness.

Moreover, due to the low water to cement/binder ratio autogenous shrinkage in the cement paste is generally higher than for ordinary concrete, as shown in Figure 16.

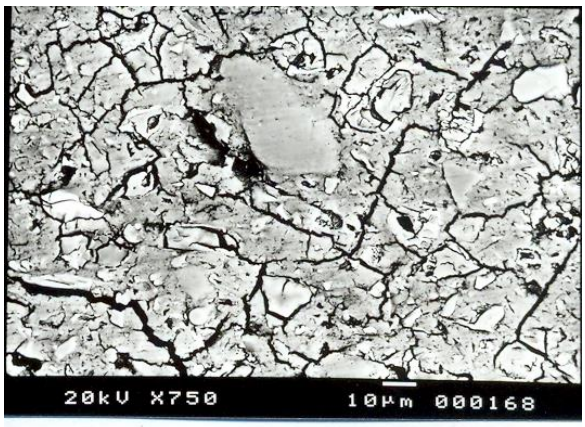


Figure 16. *Autogenous shrinkage micro-cracks in cement paste in HSC [Diederichs, U. et al; 2009]*

- **Ultra-high strength concrete (UHSC)**

Concrete with compressive strength higher than 150 MPa. Currently, this concept has fallen into disuse, and when used it is considered a synonym of Ultra-High-Performance Concrete (UHPC).

- **High-performance concrete (HPC)**

In the last years, the definition of High-Performance Concrete (HPC) has expanded to encompass both durability and strength. This terminology has been used relatively often in the industry without an official definition. It was generally accepted that a HPC was a concrete with enhanced properties (compressive strength and durability) compared to ordinary concretes. In 1998, the ACI Technical Activities Committee (TAC) and its subcommittee on HPC derived and approved an official ACI definition for this material, as: concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely using conventional constituents and normal mixing, placing, and curing practices [Russell, H.G.].

In the report published by the Federal Highway Administration of U.S. (FHWA) in October 2006, in Chapter 5, the evaluation of “High-Performance Concrete Defined for Highway Structures”, Publication Number: FHWA-HRT-05-056 The HPC is defined in detail according to several performance characteristics that encompass both durability and structural design. In Table 6 those related to structural design are considered. A higher number for the grade indicates a higher level of performance. A given HPC mix design is specified by a grade for each desired performance

characteristic. For example, a concrete may perform at grade 4 in strength and stiffness, grade 3 in shrinkage and scaling resistance, and grade 2 in all other categories.

In Europe, HPC has not been officially defined in any Norm or Standard. There is no definition of HPC or HSC in Eurocode 2, but the formulae correlating different mechanical properties to compressive strength change when the concrete strength is higher than C50/60, which may thus stand as a reasonable working definition. The design methods in Eurocode 2 are applicable to concrete with a cube strength of up to 100 N/mm^2, covering three of the four grades established for HPC by the FHWA.

Table 6. *Main characteristics for structural design of HPC from FHWA, 2006.*

Performance Characteristic	Standard Test Method	FHWA HPC Performance Grade			
		1	2	3	4
Strength (x = compressive strength)	AASHTO T 2 ASTM C 39	41 ≤ x < 55 MPa (6 ≤ x < 8 ksi)	55 ≤ x < 69 MPa (8 ≤ x < 10 ksi)	69 ≤ x < 97 MPa (10 ≤ x < 14 ksi)	x ≥ 97 MPa (x ≥ 14 ksi)
Stiffness (x = modulus of elasticity)	ASTM C 469	28 ≤ x < 40 GPa (4 ≤ x < 6 x 10^6 psi)	40 ≤ x < 50 GPa (6 ≤ x < 7.5 x psi)	x ≥ 50 GPa (x ≥ 7.5 x 10^6 psi)	
Shrinkage (x = micro-strain)	ASTM C 157	800 > x ≥ 600	600 > x ≥ 400	400 > x	
Creep (x = micro-strain/pressure unit)	ASTM C 512	75 ≥ x > 60/MPa (0.52 ≥ x > 0.41/psi)	60 ≥ x > 45/MPa (0.41 ≥ x > 0.31/psi)	45 ≥ x > 30/MPa (0.31 ≥ x > 0.21/psi)	30 MPa ≥ x (0.21 psi ≥ x)

Key attributes of HPC include the use of supplementary cementitious materials (SCMs) and chemical admixtures. Being much finer in size than cement particles, SCMs are able to fill voids in the granular structure and thus provide higher concrete strength. Chemical admixtures, such as superplasticizers, give the concrete mix adequate workability at low water/cement ratios [Hasan & Noël, 2017].

- **High-Performance Fibre Reinforced Cement Composites (HPFRCC)**:

HPFRCCs are a class of fibre-reinforced cement composites whose stress-strain response in tension features strain hardening behaviour accompanied by multiple cracking, leading to a high strain prior to failure, defined as the onset of unstable localization of one single crack. The figures below show the multi-microcracking of two HPFRCC used in two different footbridges. The concrete used for the picture below also accomplish the conditions of the French Norm NF P 18-470, so it can also be classified as UHPC (also named UHPFRC).

Figure 17. *Multi-microcracking response of a tie in a truss footbridge designed by RDC and produced by IDIFOR. Guadassuar, Valencia (Spain)*

- **High-performance Fibre Reinforced Concrete (HPFRC)**:

High-performance fibre reinforced concrete is still a material for which no internationally accepted design recommendations exist. It is a cementitious material considered to be an improvement on ordinary fibre-reinforced concretes and high-performance concretes. For some authors, it is generally accepted that this material is HPC with fibre reinforcement, also seen as a UHPFRC with lower performance. For some others, it is a concrete with the strain-hardening response, so it is a type of HPFRCC.

- **Very-high Performance Concrete (VHPC)**

This concept has fallen into disuse. When used, VHPC is considered to be a concrete with features between HPC and UHPC.

- **Very-high Performance Fibre-Reinforced Concrete (VHPFRC)**

This concept has fallen into disuse. When used, VHPFRC is considered to be a concrete with features between HPFRC and UHPFRC. The concrete of the Figure 18 is used in the handrail of a footbridge designed by RDC. It has a characteristic strength of 130 MPa so it can be classified as VHPC (also named VHPFRC). The material is also a HPFRCC as can be seen for the multi-microcracking that can be appreciated in the Figure 18.

Figure 18. *Multi-microcracking response of handrail precasted without ordinary steel by IDIFOR for the municipality of Puçol, Valencia (Spain)*

- **Ultra-High Performance Concrete (UHPC)**

Currently, the most relevant definition of UHPC is the one provided by the French Norm NF P 18-470. It states that is a concrete with at least 150 MPa of characteristic compressive strength, steel fibres to avoid brittle failure and possibly other fibres to achieve other specific features. The Norm inherently accepts that it has to be a self-compacting concrete, as it suggests three different grades of high-fluidity in fresh state.

In the last years and for its use in structures, the use of fibres that was generally accepted has become part of the definition, because it is necessary to obtain the required ductility. It is generally accepted that fibre reinforced concretes with compressive strength slightly below 150 MPa can be called UHPC as well, since their mechanical response and performance is very similar to the materials that fit the definition.

According with the French Norm NF P 18-470, some but not all the UHPC are UHPFRCC, having a multiple stable microcracking behaviour in tension. Figure 19 shows a UHPC with a characteristic strength of 155 MPa after a bending test. The material also had a multi-microcracking response, so it can be also classified as HPFRCC.

Figure 19. *Multi-microcracking response of panel tested under distributed load in RDC laboratory. The content of high tensile strength steel fibers was 160 kg/m^3*

- **Ultra-High Performance Fibre-Reinforced Concrete (UHPFRC)**

The name refers to UHPC containing fibre reinforcement. Currently, it is a synonym of UHPC.

- **Reactive-Powder Concrete (RPC)**

Material defined by [Cheyrezy & Richard, 1995] as an ultra-high strength and high ductility cementitious composite with ultra-high strength and durability, very dense mix and very low porosity. The concept was progressively replaced by the name UHPC.

- **Ultra-High Durability Concrete (UHDC)**:

A strain-hardening cementitious composite with tailored components especially oriented to obtain an extreme durability in service even under cracked state.

Though not included in the definition, to make it as open as possible, it is understood that UHDC contains fibres to obtain the strain-hardening response (so it is a FRC, and also a HPFRCC) and it has a special selection of aggregates and fines to obtain the required durability. In general, it is also a self-compacting material (SCC) to obtain a random and homogeneous fibre distribution. Figure 20 compiles and integrates all the new innovations and trends in concrete technology evolution

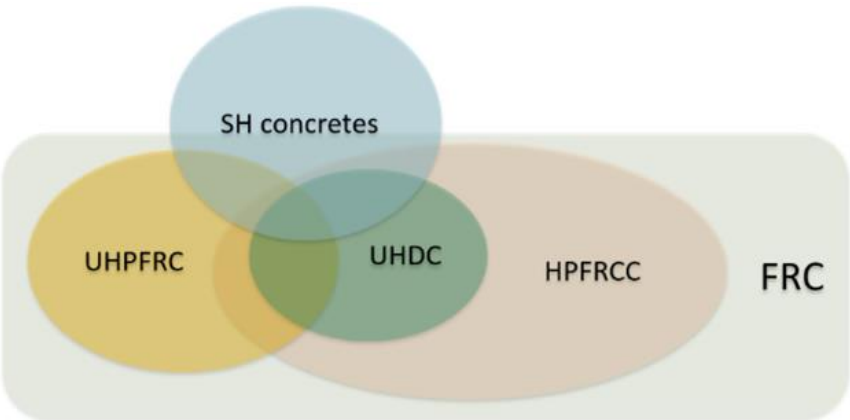


Figure 20. *Integration of Concrete technology evolution*

- **Engineered Cement Composites** (ECC)

Strain-hardening cementitious composite especially ductile due to its materials design and tailored ingredients. It can be intended as a peculiar category of HPFRCC/UHPFRC where the strain hardening behaviour in tension is obtained with low strength, thanks to a composition designed through micro-mechanical concepts based on the balance between crack tip toughness and fibre pull-out energy.

3.2. The challenge of UHDCs under service actions

Concrete under service conditions can be working in two different states: un-cracked or cracked. When it is under compression, as in prestressed concrete, it can be assumed that concrete remains in the un-cracked state. However, for other cases, concrete works in a cracked state, and the reinforcement is needed.

Because the second state is the most common in structures, crack width and distance between cracks are the analysed parameters characterizing the cracked state.

The response against transport of aggressive outdoor-borne agents is different whether if there are cracks or not. Anyway, transport can be slowed down by two ways: improving the imperviousness of the material, through a more compact matrix, or improving it structural behaviour (reducing crack width, distance and quantity).

On the one hand, to achieve a more compact material, low w/c has to be used, together with maximum size of aggregates, and finer cement substitutes such as silica fume.

On the other hand, cracking can be controlled by fibres addition, although the presence of cracks won´t be completely eliminated. In general, the more slender the fibres and the better their dispersion, the better the achieved cracking control.

Moreover, when supplementary mineral additions are used, or simply with high cement content, small cracks can be sealed in the presence of water or moisture, generally because of delayed hydration of anhydrous binder particles.

With the above characteristics, those based on UHDC design, concrete would answer differently to durability performance.

When the crack is controlled by fibres, a crack appears, but it cannot propagate. Instead of this, a new crack appears in the vicinity, but since the new crack cannot propagate either, a new one appears. The result is a surface full of very small and close cracks (multicracking), shown in Figure 1Figure 21. The typical tensile response is modified arising a strain-hardening response. Once the strain capacity is exceeded, a single crack ultimately localize and the material continues to tension-soften throughout failure [Fischer G. & Li, V.C., 2006].

Then, for this reason, it has non-sense to limit the maximum crack width in UHDC, but the evolution of this new cracking behaviour regarding loading states (or deformation).

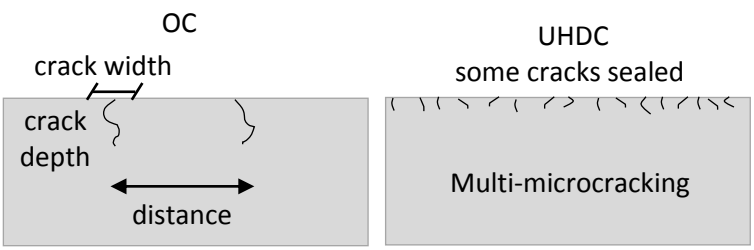


Figure 21. *OC vs. UHDC behaviour*

3.3. Performance-based evaluation and indicators for concrete durability

In order to ensure a long service life of concrete structures, very often over 100 years, when operating under aggressive conditions the development of durability-based design approaches is of relevant interest. From a material point of

view, the approaches is exclusively based on requirements on concrete composition and properties such as compressive strength have shown their limits. Therefore, a performance-based approach seems particularly relevant for durability issues and has to be applied from the design stage.

The performance of concretes when exposed to aggressive environments needs to be evaluated accordingly to parameters measured directly in the concrete that relate with durability and the conditions and damages expected to be developed in the concrete as induced from the aggressiveness of the scenario. A durability approach has to be based on key material properties that have a clear physical meaning call durability indicators. The purpose is to design concrete mixtures capable of guaranteeing protection of structures against degradation, such as reinforcement corrosion or sulphate attack or acid attack etc. for a target lifetime and given environmental conditions [Varoghel-Bouny, V., 2006].

Concrete durability indicators related to parameters evaluated in concrete can be defined in two levels:

1) Universal or indirect durability indicators, and
2) Specific or direct durability indicators, related to the specific aggressive factor causing the damage, different for the XS and XA or interaction of both

3.3.1. Indirect durability Indicators in concrete

For many durability issues, a single parameter is not sufficient to characterise the behaviour of concrete due to the various driving forces that are involved in the transport of aggressive species and the complex physical and chemical properties that can be considered as universal durability indicators since they are the most relevant in many degradation processes. All the processes of interaction of concretes with the environment occur with the presence of moisture, as consequence of the concentration of aggressive substances and temperature. The interaction of the environment with the concrete generally takes place through the penetration of the aggressive element in the concrete pore structure and chemical interaction with pore solution and hydrated cement phases inducing changes and evolution of them in the physic-chemical parameters of the concrete, so microstructural characteristics and transport properties are:

- Accessible porosity
- Permeability to gas or liquid water
- Capillary suction
- Calcium hydroxide content at initial

The analyses of these parameters can inform on the potential durability of a specific type of concrete. Threshold values with respect these universal durability indicators ranging from the very low to very high level has been proposed [Varoghel-Bouny, V., 2006] in Table 7 related to OCs and HPCs.

Table 7. *Classes to the potential durability and thresholds associated with indirect durability indicators*

Potential durability	Very Low (VL)	Low (L)	Medium (M)	High (H)	Very High (VH)
Water Accessible porosity (%)	>16	14 to 16	12 to 14	9 to 12	6 to 9
Apparent gas permeability, K_{app}(gas) ($10^{-18}m^2$) CEMBUREAU P=0.2MPa & dry 105ºC	>1000	300 to 1000	100 to 300	30 to 100	< 30
Intrinsic liquid water permeability K_{liq} (10^{-18} m^2)	>10	1-10	0.1 to 1	0.01 to 0.1	<0.01
Initial $Ca(OH_2)$ content %bmc	<10	10 to 13	13 to 20	20 to 25	≥25 (XS) <10 (XA)

The Euro International Committee du Beton (CEB) proposes a classification into three grades of concrete by capillary absorption coefficient, values below 0.1 for the capillary absorption coefficient are associated with high quality of concrete.

3.3.2. Direct durability indicators in XA environments

The specific exposure conditions of the pilots in the scenario XA for the geothermal fluid, coming from the plant condenser and in contact with the concrete, vary between plants. The expected variability is summarized in Table 8 with the aim to identify the aggressiveness and the most suitable indicators for its characterization.

In view of Table 8 the aggressiveness of the XA scenario for concrete basins could be affected by a combination of environmental actions and can be described in two levels depending on their location respect to the structure:

1. **Phenomena occurring inside the concrete due to interaction with waters:**
   - The high content of sulphate in the fluids can induce a damage of expansion in the concrete as consequence of the formation of new phases inside, as ettringite or gypsum. The indicator to identify this risk of damage is the % of volume change in the concrete along the exposure time. According to [Biczok, J., 1967] the mechanism of reaction changes with the concentration of sulphates. At low concentration of sulphates (< 1000ppm $SO_4^=$) the primary product is ettringite, at high concentration (>8000ppm $SO_4^=$) gypsum is the main product, in the intermediate both products are possible.
   - The presence of chlorides can also be expected in this XA scenario and their transport in the concrete would allow corrosion of steel if reinforcement were included in specific parts of the concrete basin. The durability indicators for this type of damage will be the same described for XS, identified as chloride diffusion coefficient and chloride threshold.
2. **Phenomena occurring from the surface of the concrete**
   - The dissolution of the cement paste as consequence of the lower pH of the waters inducing acid attack and leaching are the processes taking place and progressing as a degradation front. The indicator, in this case, could be the surface mass loss and the identification of the thickness rate increase of the degradation front.
   - The movement or impact of the water on the surface of the concrete can also be one type of damage, inducing erosion phenomena that can even accelerate the previous damage actions. The indicator, in this case, will be the mass loss of the concrete, being the rate of mass loss of particular interest.

Table 8. *Identification of damage and durability indicators for XA scenario*

	Temperature (ºC)	pH	$SO_4^=$ (mg/l)	Cl^- (mg/l)	Water movement
Water characteristics from cooling tower in the basin	15 to 20	6.5 to 8	200 to 10000	<200	Dynamic (water falls down >10m from the top of the cooling tower to the collecting basin)
Type of damage	-	Slightly aggressive acid (XA1)/ Leaching	external Sulphate attack inducing expansion (XA1 to XA3)	Cl transport & corrosion if reinforced	Erosion
Durability indicator	-	concrete surface mass loss (Kg/m^2)	concrete volume change (%)	Cl diffusion D (m^2/s)/ Cl threshold, % Cl at rebar level bwb	concrete surface mass loss (Kg/m^2)

- **Durability indicators for sulphate attack**

Solutions containing sulphate can lead to a sulphate attack of concrete and produce expansion, but decreased stiffness, strength and increased permeability are also important effects. However, the expansion data provide a general indication of the effect of mixture parameters on concrete performance [Taylor, H.F.W. 1990; Monteiro, P.J.M. et al 2003; Bensted, J. et al, 2007]. The effects are minimised in a dense concrete with low permeability and by using sulphate resistance cement.

The main factors that affect the rate and severity of the sulphate attack are: the nature of the reaction products, impermeability of the matrix, concentration and mobility of sulphates, nature of the accompanying cation, pH of the sulphate solution, the presence of other dissolved phases such as Cl, the temperature of the exposure, the C_3A and $Ca(OH)_2$ content of the cement and extend to which stresses resulting from expansive reaction [Bensted, J. et al 2007, Menendez E., et al, 2011]. Most standard tests are carried out by immersion of specimens in alkaline sulphate solutions; however, interpretation and quantification of results are difficult. The need for developing standardized methods for service life design of concrete resistance to sulphate attack is of relevant interest as pointed by [Santhanam M., et al 2001]. Laboratory indicators of sulphate damage can be applied to the monitoring of field structures.

Expansion data from different concrete compositions during 40 years in natural testing with specimens continuously submerged in sulphate rich environment are collected in [Santhanam M., et al 2001; Kurtis K.E., et al, 2000]. Sulphate attack could be predicted as a function of w/c, duration of exposure and C_3A content as shown in Figure 22 [Monteiro, P.J.M. et al 2003]. A safe region appears with w/c ratio below 0.45 with C_3A content lower than 8%. The time to failure is defined as the time at which the expansion exceeded the value of 0.5%.

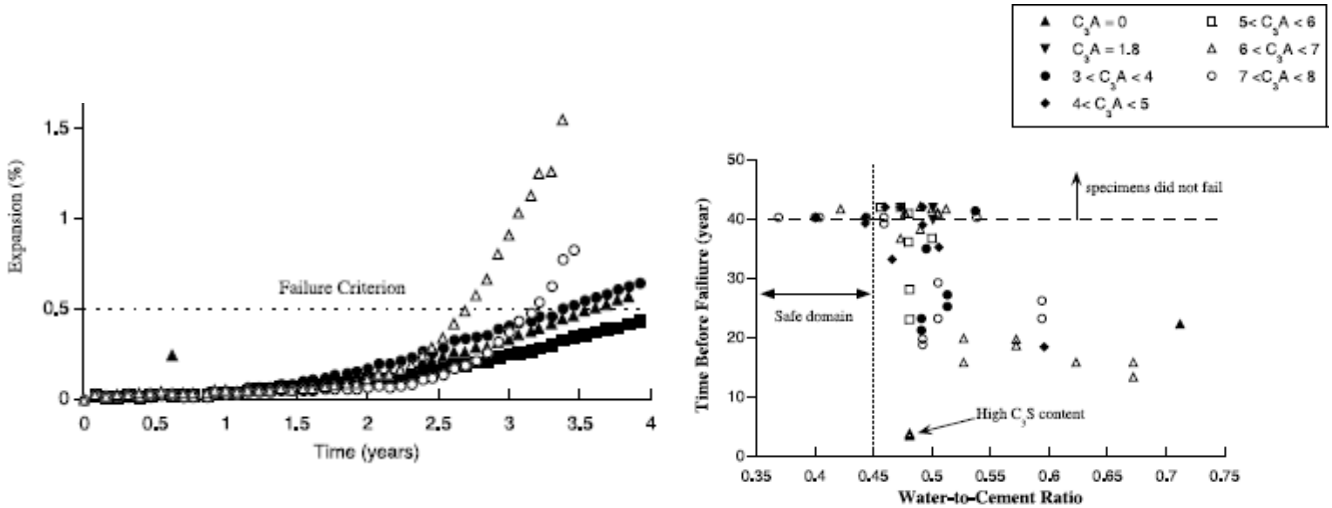


Figure 22. *Time to failure (left) as function of w/c ratio (right) [Monteiro, P.J.M. et al 2003]*

In [Kurtis, K. et al, 2000] the partial replacement of OPC by SCM resulted in considerable improvements in sulphate resistance of concretes. In many cases, the criteria of no-failure are defined as ≤0.05-0.1% after 6 or 12 months of exposure. Besides, at higher temperatures (≅20ºC), the supplementary cementitious materials, particularly the fly ashes, greatly improved the resistance to external sulphate attack as compared to lower temperatures (5ºC) [Barcelo, L. et al, 2015; Ashlee, M. et al, 2015; Ashlee, M. & Thomas, M.D.A., 2015]

- **Durability indicators for acid attack/leaching**

In a leaching process, due to the interaction of concrete with water, a dissolution of lime at different stages from the compounds in the cement matrix takes place, resulting in increasing of permeability. In pure leaching, the process is very slow, except in very permeable concretes and waters flow through. The presence of acids increases the rate of attack by reacting with or dissolving the basic constituents of hydrated cement [Bensted, J. et al, 2007].

The quality of the concretes is more important than the cement type in resisting acid attack. Well cured and compacted concrete with low w/c ratio is dense and low permeable and therefore limits the rate of fluid ingress and as a result,

the attack of the concrete restricted to surface erosion. The effect of inclusion of SCM is less significant, although some improvements have been found with Silica Fume [De belie, N. et al, 1997].

The solubility of the lime will depend on the hardness (acidity) of the water and on the type of lime in the concrete. The more acid the water, the higher its ability to dissolve concrete. In soft concentration of acids, the damage phenomena taking place can be closer to a leaching process.

The characteristics and time process depend on the permeability of concrete and on the pressure gradient of water in the concrete structure. It also depends on whether erosion occurs or not. Two leaching processes are analysed as possible to occur in the XA pilots:

1) *Surface leaching with no erosion and no water pressure gradient*: In dense concrete, the reaction only occurs on the surface, so that a dissolution process occurs as a moving boundary and is diffusion controlled as shown in Figure 23 [Fagerlund, G.; 1996]. In this case, the thickness of the dissolved zone grows with the square-root of time. Leaching rate can increase if water outside is moving. The diffusion controlled dissolution occurs as long as lime is the only substance leached. Then the leaching process will become linear with time as represented in Figure 23.
2) *Surface leaching with erosion*: If the water is streaming along the surface at high speed and if it brings with it sand and other erosive particles, the dissolved and weakened surface layer can be eroded. When this occurs the leaching process does no longer feature a linear dependence with time in log-scale [Fagerlund, G.; 1996], as shown in Figure 24. Concrete of good quality will not be significantly eroded. The erosion rate is indicated by erosion depth with time (m/s).

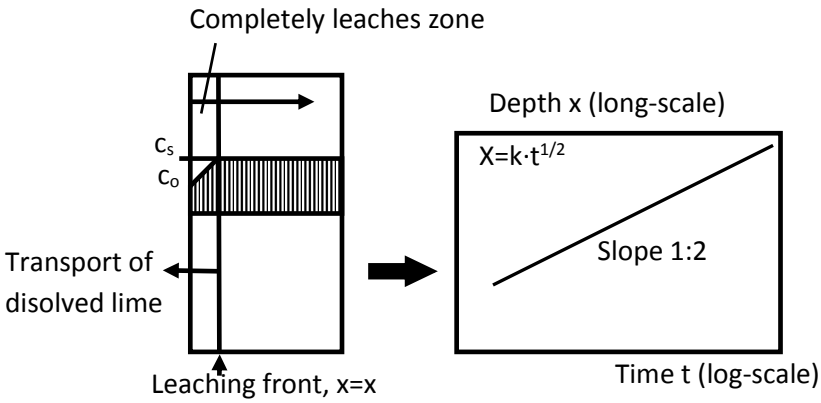


Figure 23. *Surface leaching and no erosion [Fagerlund, G.; 1996]*

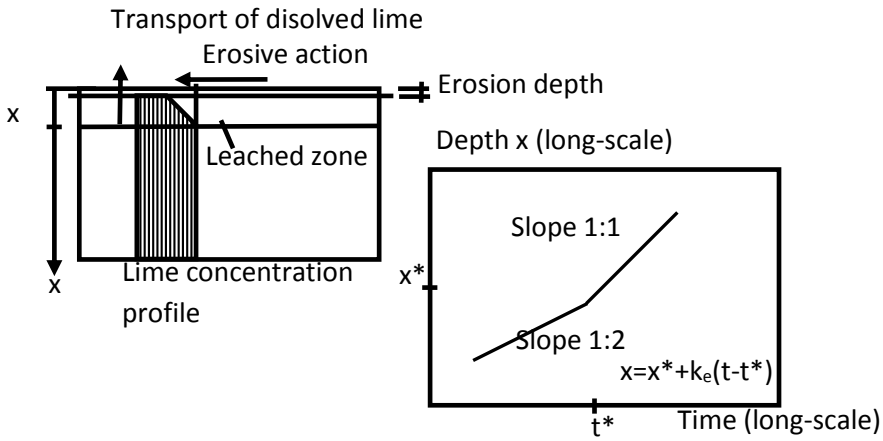


Figure 24. *Surface leaching with erosion [Fagerlund, G.; 1996]*

The prediction of performance of concrete under chemical degradation is complex because of the numerous chemical reactions and variety of conditions potentially involved. Models have been developed for the leaching of calcium ions by water even in case of UHPC [Matte, V., et all, 2000] where the depth of degradation is expressed as the depth of the altered zone, leaching kinetics and the evolution of calcium concentration in the material. Also for external sulphate attack/decalcification [Maltais, V., et al, 2004], w/c ratio was found to have a major influence on the kinetics of the degradation, emphasizing the interest of using low w/c ratios to increase the durability of cement based materials to chemical attack.

Few studies have been carried out comparing and modeling acid resistance of ordinary concrete with HPC, in [Schmidt, H. et al; 2008], a parameter of corrosion depth is calculated, (X_{VR}), through the determination of specific parameters. X_{VR} being:

$$x_{VR} = \frac{H_{M,t} \cdot 100}{H_M} \ (mm)$$

Where H_M is the total consumption of protons of a powered specimen after oven drying at 105ºC, and $H_{M,t}$ is the consumption of protons of the embedded specimens in the acid solution for the interested exposure time.
X_{VR} were found for UHPC to be half than for OC, < 1mm, depending of the pH in sulfuric acid media.

In [Beddoe, R.E., 2016] the dissolution of UHPC in acetic acid gives the same degradation depth of 3mm (pH=9) for both experimental and modelling, while for Ca^{2+} release, a slightly lower values in the experimental (19.8 mol/m^2) with respect to the model (25.1 mol/m^2) were obtained. The molar dissolution volume of K_{Ca} is essentially the pore volume produced by the dissolution of calcium in the cement paste. The main contributors to K_{Ca} are from the dissolution of the hydration products K_h and the dissolution of residual unreacted cement, K_u. Simulations with K_h=19cm^3/mol yields K_{Ca}=19cm^3/mol for OC while K_{Ca}=10cm^3/mol as consequence of the high unreactive cement content.

While in [Alonso, C. et al, 2006] found more than 100% reduction of Ca leached in ground water (pH=8) for UHPC respect to good quality OC and HPC, expressed as the D_{nsCa} (cm^2/s), UHPC from 5 to 3 x10^{-11}, HPC from 1.2 to 0.5 x10^{-10} and for OC from 2.5 to 1x10^{-10}, between 3 and 30 months ANSI test. The depth altered by the water interaction vary from 100µm in UHPC, 250µm for HPC and 2mm for the good OC.

3.3.3. Direct durability indicators for XS environments

The main parameter responsible for the loss of durability in a reinforced concrete structure exposure to a marine environment is the chloride dissolved in the sea water. Although sulphates are also present in sea water and they are aggressive for concrete, their effect in combination with chloride ions Cl^- are less evident. Chlorides penetrate more easily than sulphates mainly due to the ion size. Besides, the main reason for failure of durability is the risk of corrosion of the steel rebar once a critical content of Cl^- ions is reached at the reinforcement level. The thermodynamic model of Tuuti [Tuuti, K., 1982] is generally accepted in practical application and standards for prediction of damage and service life of concrete structures. This model, Figure 25, is divided into two stages, the first t_i starts with the transport of the aggressive in the concrete cover and ends with the corrosion initiation. The second stage, t_p, follows the damage evolution. For the aim of the ReSHEALience project, only the first period is considered to define the end of the service life until corrosion initiation.

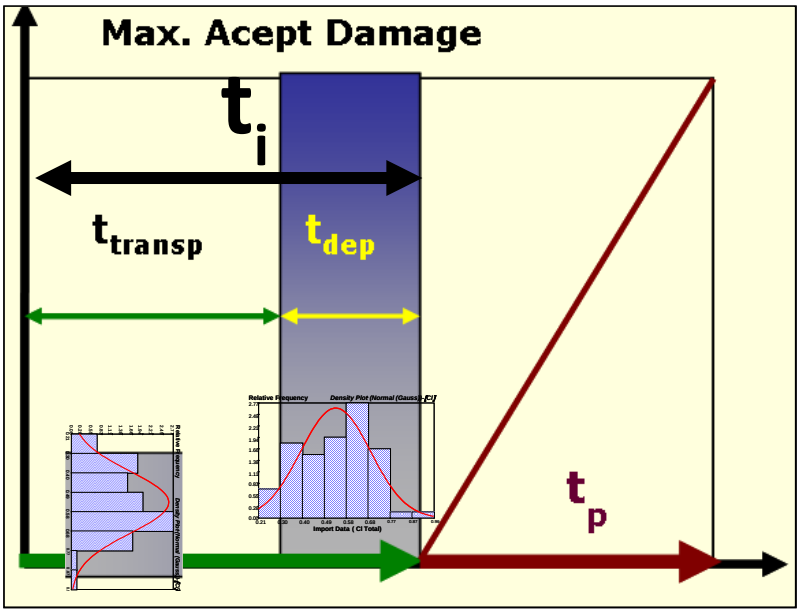


Figure 25. *Modified thermodynamic Tuuti model for chloride corrosion service life [Alonso, M.C.; 2012-2015]*

The Tuuti model has been modified in Figure 25 differentiating the transport from the initiation of corrosion activity, this last is not an instant event and requires for the accumulation of the aggressive ions at the rebar. The two parameters that better identify this two stages period are: for chloride transport, the apparent diffusion coefficient (D_{app}) and for the second, related to corrosion initiation, the Cl threshold. The first one is significantly related with the type of concrete composition and pore structure, and the second one with the local environment generated at the steel concrete interfacial zone (ITZ). However, both processes are affected by a large scatter, shown in Figure 26 and analysed later to contribute to identifying the minimum requirement for long-term durability.

- **Concrete parameters influencing Chloride transport**

The Chlorides can penetrate in concrete following different transport mechanisms accordingly with the environment, in submerged and high humid marine environment the Cl transport occurs mainly under diffusion, in aerial marine also suction transport can take place, and in deep marine water also the water pressure can contribute. However, most of the transport of Cl^- in concrete has been characterised considering pure diffusion transport under non-stationary conditions (D_{app}). The main parameters of concrete affecting this transport are the w/b ratio, the content and the type of cement used. In Figure 26 the three parameters and their interaction have been analysed in data from a specifical literature review carried out [Shin, H-O. et al (2016); Yoo, S-W. et al (2016); Ryan, P.C. et al (2016); Lee, B. et al (2016); Álava, H.E. et al (2016); Bernal, J. et al (2016); Maes, M. et al (2014); Teng, S. et al (2013); Fraj, A.B. et al (2012); Lizarazo-Marriaga, J. et al (2009); Vedalakshmi, R. et al (2009); Alonso, MC et al; 2013; Luna, F.J. (2018)]. The large scatter is clearly appreciated for very similar concrete characteristics. Concerning OPC concretes, the influence of the cement content in the transport cannot be easily appreciated without considering the effect of the w/c ratio. High w/c predominate in the transport respect to the cement content due to the high porosity. High w/c are more affected in the Cl transport for moderate to low w/c and cement contents above 600kg/m^3 are needed if transport around or below $1{\cdot}10^{-13}$ m^2/s is required, according to Figure 26 up-left. For more effectiveness in the transport, also low w/c ratios, as below 0.3, are required, as shown in Figure 26 up-right. Another parameter influencing the transport is the binder composition, the inclusion of supplementary cementitious materials (SCM) can significantly retard the Cl transport, as shown also in Figure 26 down, as a consequence of their contribution to the Cl binding ability and fineness of the pore structure.

For the requirement of long-term durability service life in ReSHEALience, concretes with at least HPC characteristic has to be defined. These concretes are characterised by a very refined micropore structure. Natural diffusion could take a long time, so that accelerated methods are being developed and standardised. Being the diffusion coefficient a

controlling parameter Table 9 compile rage of magnitude variation of chloride diffusion coefficient according to concrete type from accelerated migration test.

Table 9. *Magnitude of chloride diffusion coefficient variation according to concrete type*

	Ordinary concrete	HPC	VHPC	UHPFRC
Chloride migration diffusion coefficient (m^2/s)	$>10^{-11}$	10^{-12} to 10^{-13}	10^{-13} to 10^{-12}	10^{-13}

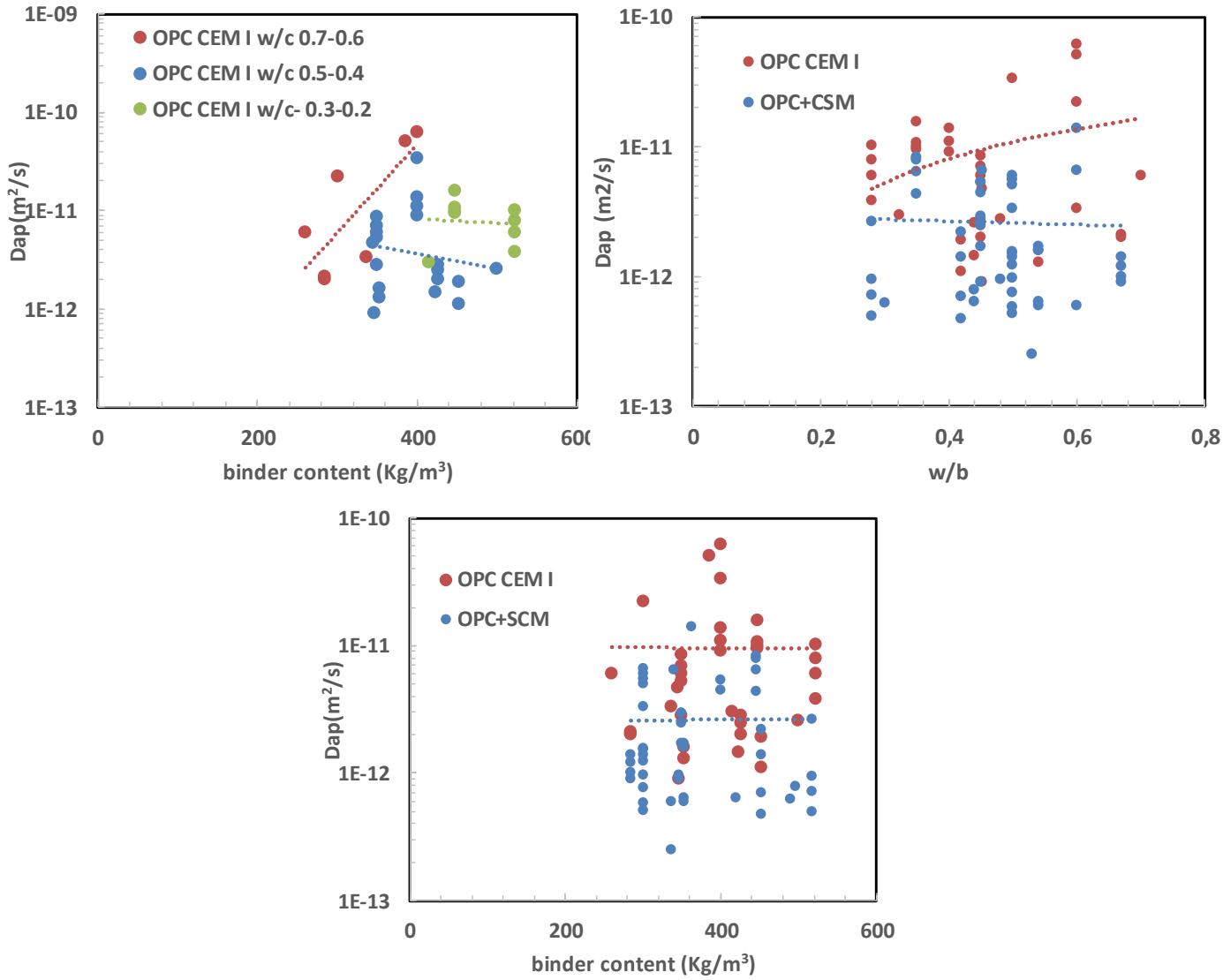


Figure 26. *Cl diffusion coefficient variability from literature data. Effect of OPC content, w/b and SCM content*

Considering this, in the ReSHEALience project, the HPC selected will be one that can be representative of the typical engineering practice. This is one, classified as Performance Grade 3 by the FHWA (strength between 69 and 97 MPa in cylinder), is also included within the Eurocode 2 prescriptions (compressive strength lower than 100 MPa on a cube). In Table 10 chloride penetration resistance from ASHTOO test are included classifying different grades according to FHWA.

Table 10. *Performance characteristics of concrete for different aggressive environments from FHWA*

Performance Characteristic[2]	Standard Test Method	FHWA HPC Performance Grade			
		1	2	3	4
Freeze-Thaw Durability (x = relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666 Proc. A	60% ≤ x < 80%	80% ≤ x		
Scaling Resistance (x = visual rating of the surface after 50 cycles)	ASTM C 672	x = 4,5	x = 2,3	x = 0,1	
Abrasion Resistance (x = avg. depth of wear in mm)	ASTM C 944	2.0 > x ≥ 1.0	1.0 > x ≥ 0.5	0.5 > x	
Chloride Penetration (x = coulombs)	AASHTO T 277 ASTM C 1202	3000 ≥ x > 2000	2000 ≥ x > 800	800 ≥ x	

- ***<u>Chloride transport in cracked concrete</u>***

Structural regulations focused on durability control allowed cracks widths based on exposure conditions, Eurocode 2 and ACI permit 0.3 and 0.15 mm respectively as the maximum values for marine environment, as will be detailed later in 4.2.1. However, the degree of protection conferred by controlling the crack width depends on more factors that are not considered in the codes, as concrete composition, cover depth, compaction, environmental conditions and curing.

Also, secondary cracks (internal cracks) can influence the Cl^- penetration but have not been sufficiently studied. The rate of chloride ingress through cracks is of relevance for determining the duration of the initiation period.

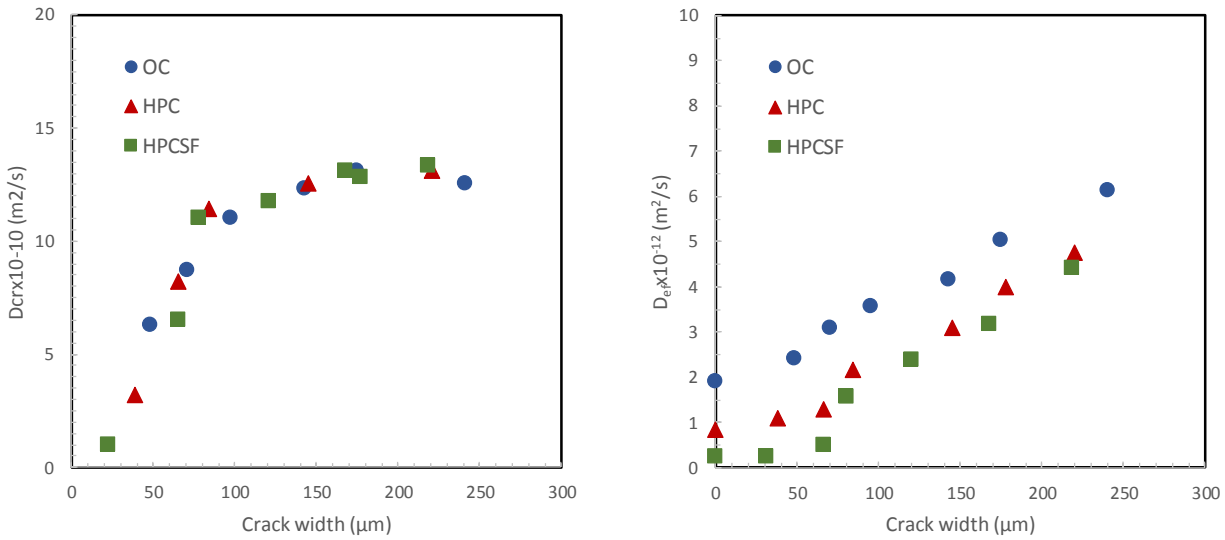


Figure 27 *Effect of crack width and concrete type on steady-state chloride diffusion [Djerbi, S. et al, 2008]*

Studies in cracked Portland cement concrete show that Cl^- ions penetrate through concrete beyond the crack whilst in cracked blast furnace slag concrete Cl concentrate in the crack [Savija, 2014]. Cracks narrow than 60µm were healed and the penetration of Cl in healed cracks was comparable to that of uncracked concrete. Other studies [Djerbi, S. et al, 2008] found that the diffusion coefficient in cracked concrete (D_{cr}) increase with linear increase of the crack width for cracks narrower than 80µm, and D_{cr} was constant for cracks wider than 80µm, independent of material effects, without differences between OC and HPC (figure 26-left). No differences were also detected with the addition of silica fume (HPCSF); although differences in D_{eff} are observed between OC and HPC with and without SF (Figure 27-right). [Boulfiza et al 2003] modelled the transport of Cl in cracked concrete and found that the process is more complex.

[Win et al, 2004] found a linear relationship between w/b ratio and Cl concentration for certain the crack width. [Konin et al, 1998] found that the composition of the concrete specimen with cracks was critical for the Cl ingress. The effects of crack width/cover depth ratio was addressed by [Gowripalan et al, 2000], a crack width-to-cover depth ratio of 0.01 produced an increase in the D_a in the tension zone of about two times.

- **Chloride *transport in real structures from D3.1***

As highlighted before, the main parameters of concrete affecting the chloride ion transport are the w/b ratio and the content and type of cement used. However, the age of the concrete is an important parameter too.

In Figure 28, it is analysed the diffusion coefficient from structures, in-put from D3.1 and [Bermúdez, M.A.; (2007)]. As also shown from the literature data in laboratory, analysing cement content, differences in Cl diffusion coefficient, D_{app}, are not evident; however, taking into account w/b ratio, the increase of D_{app} with the ratio due to the high porosity is clearly observed (left). This confirms the results obtained from the literature review in the previous point.

Blue: Portland cement without additions
Green: Portland cement with Fly Ash
Pink: Portland cement with Slag

Triangle: Mediterranean
Square: Atlantic
Circle: Cantabrian

Solid: submerged
Hollow: tidal zone

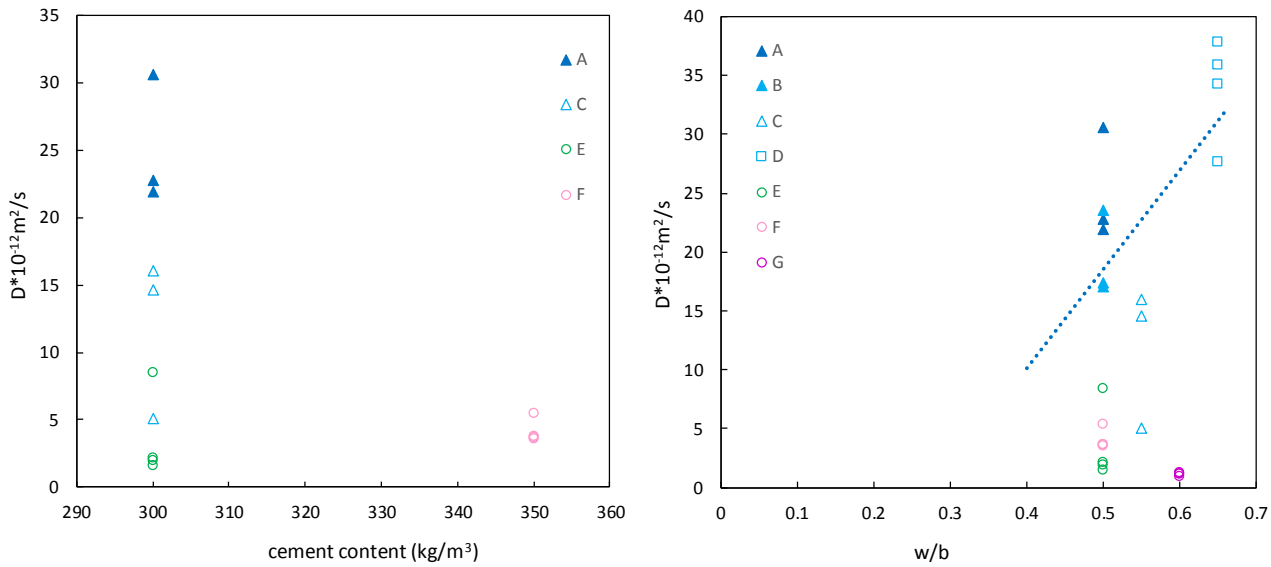


Figure 28 *Chloride diffusion coefficient in concrete structures exposure in marine environments*

The influence of the binder composition also confirms the previous results. The inclusion of SCM, FA (represented in green in Figure 28) and Slag (represented in pink) imply a reduction of D_{app}, even if the cement content and w/b are the same (green series).

Finally, the evolution of the coefficient with the time is represented in Figure 29. For OPC (blue series), the coefficient is reduced by half in less than five years. When SCM is added, the reduction is less pronounced, in the case of series G (OPC with slag located in Cantabrian environment), a reduction lower than 20% has occurred along 30 years. This will be important for the requirement of long-term durability service life in ReSHEALience concretes. D_{app} measured at 1 year will be very different from the measured at 5 years, while in the mixes with SCM the expected coefficient will be much lower than the one from the reference mix and will remain almost constant during the first years; which means that limiting the coefficient at early ages in concretes with mineral additions, service life will be extended much longer in concretes with SCM than in OPC.

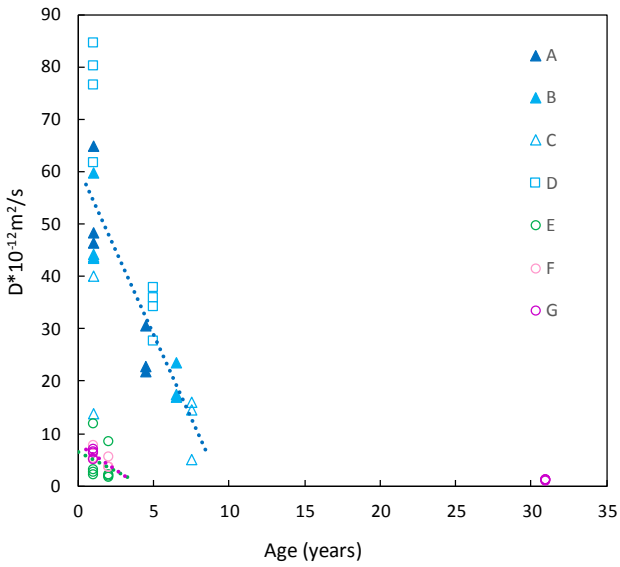


Figure 29. *D_{app} decrease with exposure time in real exposure concrete structures in marine environment*

- **Variability of Cl content for corrosion initiation**

The second stage of the initiation period in XS environment is characterised by the identification of the amount of Cl accumulated at the rebar surface that can cause rebar corrosion onset. There are several papers [Ann, K Y et al, 2007; Angst U., et al, 2009; Alonso M C, et al, 2009] that have analysed the variability of the Cl threshold. Total chlorides content variability is shown in Figure 30-left [Alonso, M.C. et al 2009] that can cause corrosion initiation have been found to vary from 0.3 to 3.5% total Cl with a value in the order of 1.2%, even twice than that accepted in some standard that considered 0.6%, EHE-2008. The lower values, however, are generally related to the presence of cracks. For cracks >0.4mm it has been found the Cl threshold to be reduced close to 0.2%, as compiled in Figure 31-right.

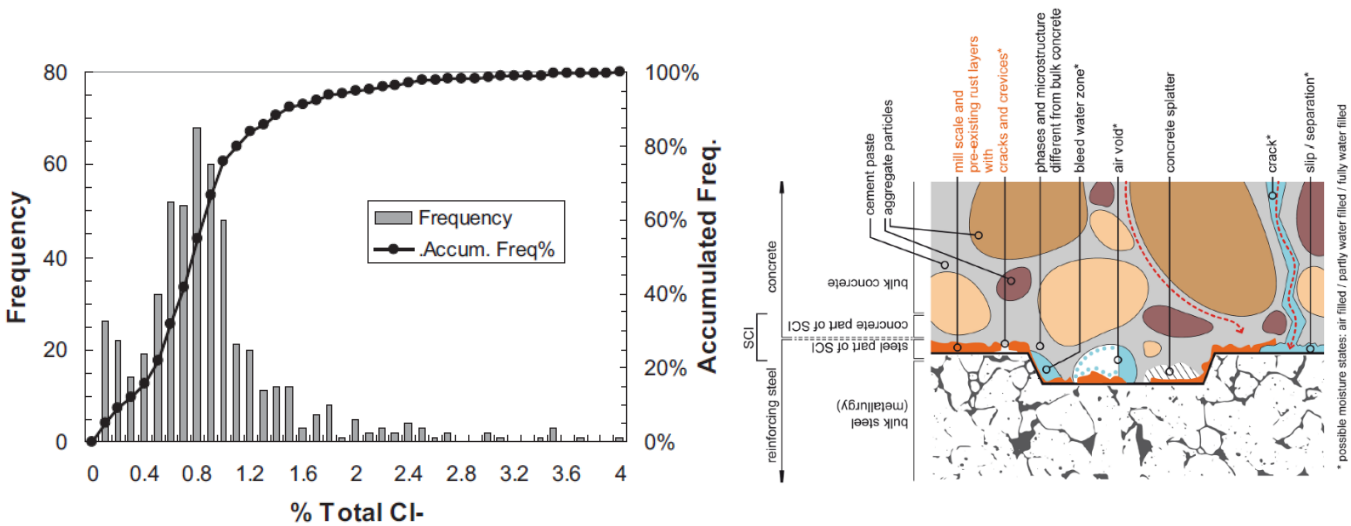


Figure 30. *Chloride threshold variability as from [Alonso, MC et al, 2009]. Characteristics at the ITZ steel/concrete [Angst, U., et al, 2017]*

The large variability in chloride threshold has been attributed to the large number of characteristics that are occurring at the rebar level. In [Angst, U. et al, 2017] a detailed analysis has been performed, as those shown in the schematic illustration of Figure 30-right, with reference to selected characteristics at the steel-concrete interface (SCI) that may or may not be present locally. Red dashed lines indicate preferential pathways for chloride ingress; blue dots represent adsorbed water (only shown for large pores).

Non standardise Chloride threshold test exist that also contribute to this variability; moreover, the method available using natural transport of Cl takes very long time. Differences between laboratory test and field data also hold, Figure 31-left.

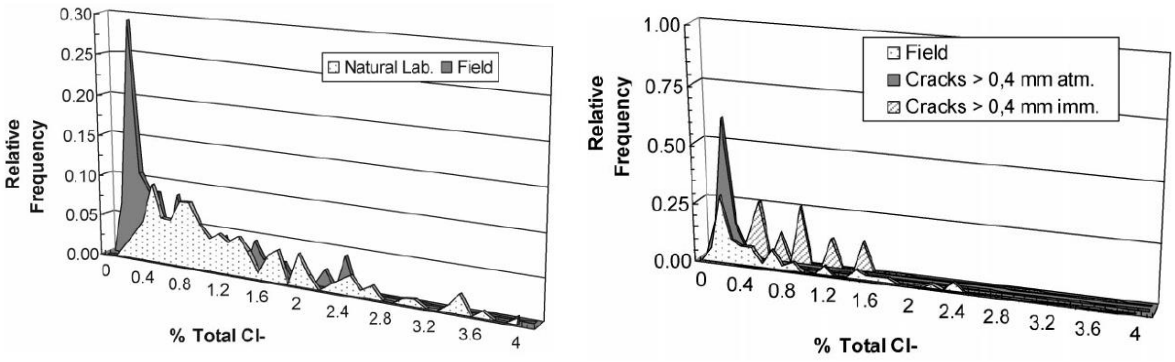


Figure 31. *Differences between Lab & field Cl threshold values (left). Crack width (right) [Alonso, MC., et al 2009]*

4. Durability construction regulations and recommendations

Several codes, regulations and recommendations were analysed in order to obtain the current durability requirements employed in the design of the pilots. The main information is summarised further and analysed with respect concrete components, and structural main parameters for exposure conditions of XS and XA. Main durability parameters are analysed.

4.1. Material and structural requirements for XS/XA exposures

Two approaches are followed: material requirements versus structure requirements. Table 11 to Table 18 summarize the value of the main parameters regulated accordingly with structural codes from Europe (EUROCODE) and national standards. Reference to ACI codes and standards has also been made. In the Annex the extensive information collected from construction codes is also included.

4.1.1. Materials requirements

With respect to materials, the cement content, type of cement and w/c ration recommended are analysed

Table 11. *Material requirements*

	Maximum w/c	minimum cement content	
	ratio	kg/m^2	
XS	0.40 - 0.65	300 - 400	
XA	0.45 - 0.65	275	- 400
		325	
		325	

➢ **Type of cement**

Before analysing the target values for w/c or minimum cement content, the first reasonable step would be to fix the type of cement according to the exposure.

However, the way in which the standard takes this into account is not restrictive. Even more, not all the Standards establish requisites or recommendations about that.

In general, in those Standards in which the type of cement is explicitly contemplated in relation to the environmental conditions, it is included as a mitigating circumstance that allows the target values to be relaxed. In the case of Spanish code, for example, the cover can be diminished; on the contrary, Irish Eurocode-Annex uses the type of cement together with w/c ratio, strength class and minimum cement content providing different combinations of them.

- **Maximum w/c**

The importance of the water/cement ratio in relation to durability is well known, and it is also closely linked to the other properties of the concrete, both in its fresh and hardened state.

However, not in all standards, there is a limiting value for this ratio regarding XS/XA exposure scenarios. The reason for this is probably that other properties may limit the w/c ratio to a greater extent.

In spite of this, in the case of the Irish annex and other national regulations, such as Danish, German or Spanish, there are limit values for XS, in general, more restrictive for XS3 environment. Overall, maximum w/c varies between 0.40 and 0.50, except in the case of the German standard, where a maximum of 0.65 is maintained for all XS.

Maximum w/c is quite similar for XA environment, although it should be pointed out that Irish Annex does not raise isolated values, but in relation with minimum strength class, minimum cement content and cement type or combinations.

- **Minimum cement content**

The cement content is limited analogously to the w/c ratio. Only two national Codes recommend a minimum cement content, which increases with the aggressiveness of the environment (from XS1 to XS3 and from XA1 to XA3).

Values are between 300 and 400 are recommended for all the XS and XA environments, except for the exceptional case of less aggressive environment XA1, for which the Spanish standard is less demanding, requiring a minimum cement content of 275 kg/m^3.

4.1.2. Structural design requirements

With respect to structural parameters, those analysed are: Stress limitation, compressive strength, minimum cover and maximum crack with.

Table 12. *Structural design requirements*

	minimum compressive strength	minimum concrete cover	maximum crack width
	Mpa	mm	mm
XS	25 - 40/50	25 - 75	0.1 - 0.4
	150 - 150	10 - 30	
XA	25/30 - 40/50	-	0.1 - 0.3

* French UHPC

- **Stress limitation**

In the absence of other measures, the maximum allowed compressive stress $k_1 f_{ck}$ is 0.6 f_{ck} for all the environments considered under the characteristic combination of actions, both XS and XA, and it doesn´t vary with the aggressiveness of the environment.

- **Minimum compressive strength**

Compressive strength is the most important parameter standardised. The Eurocode, as implemented through the national annexes, and the national codes, establish a minimum compressive strength or a minimum strength class.

In this case, it is important to highlight that the maximum of 150 MPa corresponds to a UHPC from the French code, while the other compressive strength classes are meant for conventional concretes. In this respect, there are not many

differences between XS and XA minimum compressive classes, with the exception of some cases of XA1 in which some standards are lenient and most of XA3, in which standards are more restrictive.

Generally, apart from these few cases, the strength required is around 35-45 MPa.

- **Minimum concrete cover**

The concrete cover is one variable considered in most of the structural design codes or regulations. According to the Eurocode, the concrete cover is identified as the distance between the surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant) and the nearest concrete surface.

The nominal cover shall be specified: It is defined as a minimum cover, c_{min}, plus an allowance in design for deviation, Δc_{de}v:

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

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

This C_{min} includes aspects related to the structure for bond requirements (C_{minb},) and also additional according to the environmental conditions (C_{mindur}). Cases for reduction of the C_{min} is also considered in the standards as for incorporation of additional protection, but always a minimum cover of 10mm is needed.

Even though the standards limit this requirement only for a XS environment, the analysis is much more complex because other issues are concerned, such as structural class or whether the structure is reinforced or prestressed.

On this matter, Eurocode includes the structural class classification for reinforced concrete as well as for prestressed, reinforced concrete corresponding to the least restrictive case S1; for a reinforced concrete class exposed to a XS1 environment, with c_{min}+Δc_{dev} of 20+10.

c_{min} increases with the exposure class from XS1 to XS3, as well as with structural class from S1 to S6, and it is higher in prestressed than in reinforced concrete; whilst Δc_{dev} remains constant.

In general, the value of Δc_{dev} is the same for all the XS, and sometimes it is even included in a general value of cover. Even so, some national codes, like Spanish or French, make Δc_{dev} to vary depending on the type of cement, reinforcement, XS subclass based on the aggravating or mitigating circumstances.

In addition to that, Spanish code demands the verification of Durability Limit State.

For the special case of DNV for off-shore structures, also the service life is considered as well as the corrosion sensitivity (from sensitive to slightly sensitive). Minimum concrete cover increases considerately (from 40 to 70).

On the other hand, for French UHPC, 40% lower concrete covers are allowed (from 10 to 30).

- **Maximum crack width**

The maximum crack width is limited by several standards, including the Eurocode. In general, it is limited to 0.2 mm, but in the more aggressive environments of both XA and XS exposure classes, it is restricted to 0.1 mm. On the contrary, for less aggressive environments, it could reach 0.3 mm for XA and 0.4 mm for XS.

For XS environment, standards consider the type of reinforcement and load combination. In the case of Eurocode Annex from Sweden, also working life (20, 50 or 100 years) and two level of corrosion sensitivity are considered.

In contrast to this, for XA environment only the general part of Eurocode limits the crack width, without any modification in National Annexes, and just the Spanish code provides further limitation for the crack width for reinforced concrete. In Table 13 maximum allowed crack width for reinforced concrete structures exposed to chloride contaminated environments, which is especially interesting for ReSHEALience project, is highlighted.

Table 13. *Maximum crack width*

Regulation	Crack width, mm
ACI 224	0.15
Fib-Model code	0.3
BS-8110	0.3
Eurocode 2	0.3

4.2. Evaluation of Durability parameters

Main parameters collected in the standards and recommendations are analysed according to the scenarios XS or XA. The parameters are classified into indirect and direct durability indicators.

4.2.1. Indirect durability parameters

Table 14*. Durability parameters*

	gas permeability m^2	water porosity %	water permeability mm
XS	$\leq 10^{-19}$ - $\leq 9\ 10^{-19}$	6 - 9	20 - 50
XA	$\leq 10^{-19}$ - $\leq 9\ 10^{-19}$	6 - 9	20 - 50

- **Gas permeability**

Gas permeability is a good indicator of durability, however, only the French standard PR NF P18-470 (for UHPFC) considers this test.

This test is regulated by the XP standard P18-463, written by l'Association Française de Normalisation. The longer is the working life for which the structure is designed, the more restrictive the limitations The established values are the same for XS and XA environments, although for this last exposure further restrictions hold with reference to the type of cement which can be used. The gas permeability allowed values ranges between $9{\cdot}10^{-15}$ and 10^{-19} m^2 depending on the impermeability of the concrete.

- **Water *porosity***

Some standards, such as the French PR NF P18-470 (for UHPFC) or the Spanish EHE-08, consider important to control this parameter, although there is no consensus on the type of test that must be performed. The standard PR NF P18-470 establishes for this purpose to define the water porosity, setting limit values. This test is regulated by the NF P 18-459 standard, written by l'Association Française de Normalisation. The longer is the working life for which the structure is designed, the more restrictive the limitations The established values are the same for XS and XA environments, although for this last exposure further restrictions hold with reference to the type of cement which can be used. The maximum water porosity allowed varies between 6 and 20% from UPC to OC.

- ***Water permeability***

The Spanish standard EHE-08 proposes for conventional concretes the test of penetration of water under pressure, regulated by the European standard EN 12390-8. According to EHE-08, a concrete has enough water permeability if maximum values of average depth and maximum depth are simultaneously met. These values are between 20 and 30 mm, and between 30 and 50 mm, respectively.

The most relevant reference to the HPC durability properties has been done in the document published by the AFGC-SETRA in June of 2013 "Ultra-High Performance Fibre Reinforced Concretes". The document contains a range of durability properties estimated for HPC. Table 15 and Table 16 show these ranges for different properties:

Water porosity shall be measured according to the French standard (AFREM recommendation) entitled "Determination of apparent density and water voids". The test consists on determining the following elements by weight: the mass of a dry test specimen, its mass when it is saturated with water and its apparent volume determined by hydrostatic weighing.

Table 15. *Water porosity*

	Ordinary concrete	HPC	VHPC	UHPFRC
Water porosity (%)	14-20	10-13	6-9	1.5-5

It is relevant to highlight that in the French Norm that has been published to substitute these Recommendations, "Complément national à l'Eurocode 2 — Calcul des structures en béton: règles spécifiques pour les Bétons Fibrés à Ultra-Hautes Performances (BFUP)", the value of maximum water porosity specified for normal UHPFRC is 9% and for improved sustainability UHPFRC is 6%.

Oxygen Permeability shall be measured according to the method included in the AFREM recommendation and entitled "Hardened concrete gas permeability test" [GRAN,07]. The test consists on measuring the mass flow rate of gas in steady-state passing through a sample of hydraulic-binder-based material subjected to a constant pressure gradient. Darcy's law is then applied to obtain the gas permeability. The measurement range for each type of concrete is shown in Table 16.

Table 16*. Oxygen permeability recommendations from standards*

	Ordinary concrete	HPC	VHPC	UHPFRC
Oxygen Permeability (m^2)	10^{-16}	10^{-17}	10^{-18}	$<10^{-19}$

It is relevant to highlight that in the French Norm that has been published to substitute these Recommendations, "Complément national à l'Eurocode 2 — Calcul des structures en béton: règles spécifiques pour les Bétons Fibrés à Ultra-Hautes Performances (BFUP)", the value of oxygen permeability for normal UHPFRC is $9 \cdot 10^{-19}$ m^2/s and for improved sustainability UHPFRC is $1 \cdot 10^{-19}$ m^2

4.2.2. Direct durability parameters for XS environments

Table 17*. Direct durability parameters*

	chloride content	chloride threshold	chloride diffusion coefficient
			m^2/s
XS	0.1 - 0.4	0.1 - 0.6	$\leq 10^{-13}$ - $\leq 5\ 10^{-13}$
XA	0.15 - 0.4	0.3 - 0.6	$\leq 10^{-13}$ - $\leq 5\ 10^{-13}$

- ***<u>Chloride content/Chloride threshold</u>***

In order to reduce the risk of corrosion of reinforcement, some construction regulations limit the total content of chlorides accepted in the production of concrete, which means in its fresh state, as reported in Table 17. In this case,

the chlorides could be present in the constituentts of the concrete, including water, aggregates or additives but never overpass the limit, that vary for passive or active reinforcement.

For the case of service life estimation some standards, as the Spanish EHE, consider the possibility of a certain content of chlorides before corrosion is initiated, referred to as chloride threshold. In Table 17 these values indicated as referred to hardened concrete. These values are those considered for service life calculation of durability of reinforced structures in XS environments.

In the Danish standard DNV-OS-C502 2012 and in French PR NF P18-470, the criteria are much more restrictive because it is a specific standard for offshore concrete structures.

In all the standards, the limitation is more restrictive for prestressed concretes in the fresh state. This is due to the greater sensitivity of these elements against bond losses and to avoid the problems caused by the corrosion under stress, named stress corrosion cracking.

- ***Chloride diffusion coefficient***

Concerning the Chloride transport only the French standard PR NF P18-470 for UHPFC provides coefficient limit values, obtained by accelerated chloride ion migration test in non-stationary regime (French standard XP P 18-462). The longer is the working life for which the structure is designed, the more restrictive the limitations The established values are the same for XS and XA environments, although for this last exposure further restrictions hold with reference to the type of cement which can be used. The maximum chloride diffusion coefficient allowed varies between $5 \cdot 10^{-13}$ and 10^{-13} m^2/s, as shown in Table 18

***Table 18**. Durability minimum levels for performance characterisation in HPC*

	Water porosity (%)	Oxygen Permeability (m^2)	Chloride diffusion coefficient (m^2/s)
durability levels for HPC	10-13	10^{-17}	10^{-11}-10^{-12}

4.2.3. Direct durability parameters for XA environments

The standards consider different exposure categories in relation to the chemical attack according to the aggressiveness of the environment. The parameter that defines the level of aggressiveness is the concentration of the aggressive component or ion. The most common are the sulphate content and the pH or acidic water solution in contact. In general, the standards define the aggressiveness depending on the sulphate content of the groundwater or the soil (*Table 19*).

***Table 19**. Concentration of sulphates as SO_4^{2-}*

		EN 206.1:2008 EHE-08			ACI 318 – 08 ACI 201.2R				Canadian Standard A23.1.94		
		XA1	XA2	XA3	S0	S1	S2	S3	S3	S2	S1
		Low	Moderate	High	Negligible	Moderate	Severe	Very severe	Moderate	Severe	Very severe
Ground water (ppm)		200–600	600–3000	> 3000	< 150	150–1500*	1500–10000	> 10000	150–1500	1500–10000	> 10000
Soil	**Total (ppm)**	2000–3000	3000–2000	> 12000							

	Water soluble in soil (% by weight)				< 0.10	0.10–0.20	0.20–2.00	> 2.00	0.10–0.20	0.20–2.00	> 2.00

(*) Seawater

The EN-206 considers three levels of aggressiveness for scenario XA, as described in Table 4.

- In case of acid attack, pH variations between: <4.5≥4 for XA3, <5.5≥4.5 for XA2 and ≤6.5≥5.5 for XA1to 6.5 from XA3 to XA1.
- In case of sulphate attack, $SO_4^{=}$ content variations (mg/l): >3000≤6000 for XA3, >600≤3000 for XA2 and ≥200≤600 for XA1. The minimum soluble sulphate concentration in groundwater for the S1 class of exposure in Canada (10000 mg/l) is much higher than the maximum of the XA3 category in Europe (6000 mg/l).

The Canadian standard, CSA A3004-C6, equivalent to the American ASTMC452, providing specifications for concrete sulphate attack, defines three sulphate exposure categories: S1 (very severe), S2 (severe) and S3 (moderate). A Concrete exposed to S1 or S2 environments should be made with highly sulphate-resistant cements (HSRC), with low C_3A content (<5%) and low permeability; while moderately sulphate resistant cements (MSRC) are permitted for concretes subject to S3 environments, with C_3A content less than 8% or cements with fly ash, pozzolana, silica fume or slag (low $Ca(OH)_2$ content).

Both, Canadian and American standards, limit the maximum length change at 6, 12 and 18 months (*Table 20*). Concerning expansion limits, maximum expansion (length increase) equal to 0.10% at 6 months for MSRC and equal to 0.05% at 6 months or to 0.10% at 12 and 18 months for HSRC are prescribed.

***Table 20**. Maximum length changes of hydraulic-cement mortars exposed to a sulphate solution*

Exposure subclass	Maximum length change (%)		
	At 6 months	At 12 months	At 18 months
S1	0.10	---	---
S2	0.05	0.10 (*)	---
S3	---	---	0.10

(*) The limit of expansion at 12 months only applies when the limit at six months has not been achieved

5. Targets for UHDC

5.1. KPI challenge from ReSHEALience

H2020 programme specifies that "the proposals should define, collect and make available key performance indicators (KPIs) in support of operational, technical and socio-economic impact assessment". Following this recommendation, the ReSHEALience project has seven KPI associated to the different specific objectives. This will allow evaluating their percentage of compliance during the development of the project.

The KPIs defined in the GA are detailed in the following lines to provide a clear picture of how to obtain adequately the compliance ratios:

SO	Key Performance Indicator	Target values
1	**KPI 1a**: transport properties	>100% improvement in un-cracked state
		>30% improvement in cracked state
	KPI 1b: chemical attack resistance	

2	**KPI 2a**: service life	>30% of increase of service life
	KPI 2b: maintenance cost	>50% reduction of costs
3	**KPI 3**: accuracy of any modelling	75% of accuracy
4	**KPI 4**: business plans	One / industrial partner = 8 business plans
5	**KPI 5**: communication objective	300 new subscriber/year to the newsletter=1200

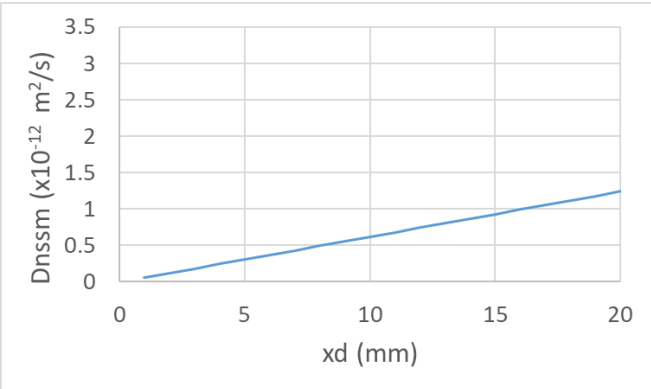
According to the proposal made in the ReSHEALience project and in the GA signed with the European Commission, all KPI are compared to high-performance Reinforced Concrete.

- ***KPI1a: Tailoring functionalized UHDC to target a 100% enhancement in material durability in un-cracked state compared to HPC***

A 100% enhancement in material durability will result into penetration of oxygen, water or ions through the uncracked material under assigned conditions divided by two.

Considering that UHDC is proposed especially for XS and XA environments, it has been agreed that the adequate durability parameters to estimate the uncracked material durability are the chloride diffusion coefficient (XS) and the penetration of the acid and sulphate attack (XA) in a standard test. The oxygen permeability test is less representative for the difficulty of obtaining perceptible values with extremely low porosity concretes, and the water porosity test is discarded because the durability does not depend only on the percentage of water pores, but also on their size distribution.

Regarding the chloride diffusion coefficient test (XS), for comparable concretes there is a linear relationship between the average value of penetration depth in the test (x_d, in mm) and the non-steady-state apparent diffusion coefficient obtained (D_{app}) or apparent migration coefficient, (D_{nssm}, in $\times 10^{-12}$ m^2/s). This relation can be observed in Figure 32 for the case of an initial current lower than 5 mA:



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

Figure 32. *Relationship of D_{nssm} vs. penetration depth Xd of Cl*

This shows that the value D_{nssm} obtained from the test is proportional to the value x_d of penetration depth along the uncracked section. Thus, it can be accepted that if the D_{nssm} value of concrete *A* is half of the value of concrete *B*, the penetration depth of concrete *A* will be also half than the penetration depth of concrete *B*. In other words, durability of concrete *A* will be 100% higher than durability of concrete *B*. On this basis, as the D_{nssm} of a standard HPC, in the test NT BUILD 492, has been assumed to be at least $0.5 \cdot 10^{-11}$ m^2/s, the compliance of the KPI1a will require that the UHDC developed in the project will provide a D_{nssm} value in the NT BUILD 492 test of $0.25 \cdot 10^{-11}$ m^2/s and lower in natural test D_{ns}.

Regarding the (XA) exposure condition, the issue has to consider the type of aggressive agent leading to damage: 1) when sulphate ions are the aggressive agents leading to damage, expansion phenomena in the concrete is taking place, and the % of volume increase is the representative parameter. If 0.1% after 12months is a limit value for a good

concrete resistant to sulphate attack, for a UHDC allowed % of volume expansion should be <0.05. 2). If the pH of the water is the aggressive parameter leading the damage (acid attack), the ion leached and mass loss are the main reason for the damage identification. The aggressiveness varies if it is diffusion controlled or combined action with erosion. An acceptable value for corrosion depth in UHDC could be below 1mm.

The compliance of a 100% improvement for both tests implies the compliance of the KPI 1a.

- **KPI1b: At least a 30% of increase of material durability in cracked state/service conditions**

Again, the KPI compares the performance of a solution designed with UHDC with other designed with reinforced HPC. The design in service conditions of a structure with HPC is associated to certain maximum crack widths and certain geometric covers, provided by the standards. The aggressive agents require certain time *t* to penetrate along the cracked cover to reach the reinforcement. This time can be determined with a specific test, being a parameter to estimate the material durability under assigned service conditions, as shown in Figure 33.

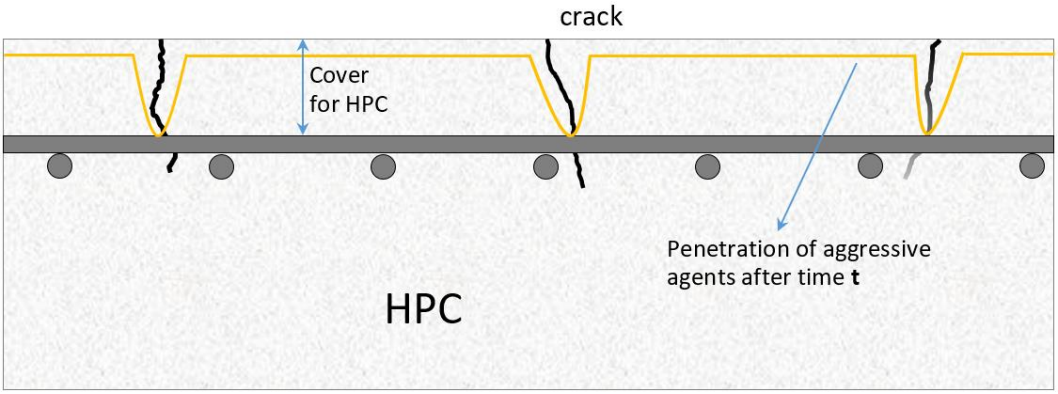


Figure 33. *Cracked over pattern for HPC affecting Cl transport*

The same test can be done for a structure designed with UHDC, and the goal of KPI1b is that the time obtained in this test is, at least, 30% higher than the value obtained for HPC. Being UHDC a strain-hardening material, the penetration of the agents through the cracks is slower. The covers for certain service conditions should be defined during the project to obtain in the test a time of, at least, 1.3*t (30% more), t being the target time determined for conventional HPC. Figure 33 shows a scheme of this concept.

This implies that, if under similar design conditions aggressive agents need a time t to penetrate through the cover c of a cracked HPC structure, they should need at least 1.3*t to penetrate through the cover c' of a cracked structure designed with UHDC, also the highest capacity of crack sealing of the UHDC will contribute. Obviously, the covers c' for each design condition will be defined in this project and it should be lower than the covers c to take the full advantage of the strain-hardening capacities of the material. In the test, the sections should be submitted to a similar tensile strain, (Figure 34).

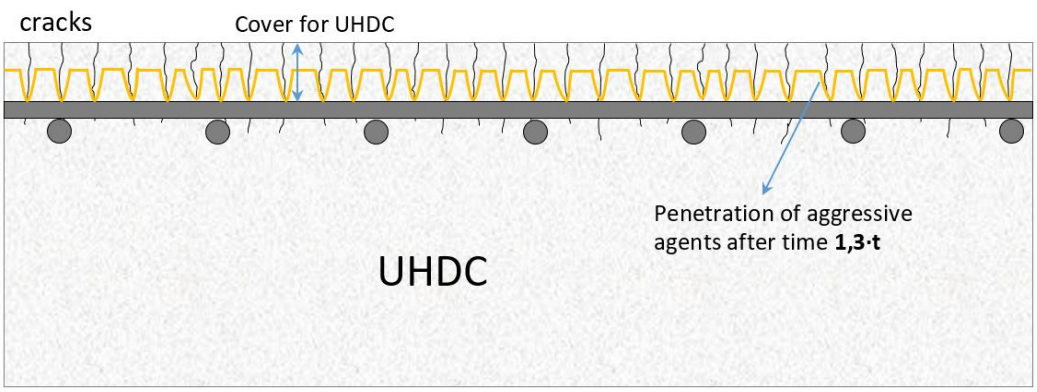


Figure 34. *Cracked cover pattern for UDC affecting Cl transport*

- **KPI2a: Increase of 30% of the service life**

It is needed to ensure that the structures designed with UHDC with cover c' have a service life at least 30% longer than a standard HPC structure. To determine the durability of a HPC structure the state of art of the structures already built with this material will be studied in deliverable 3.4. In the case of UHDC, the durability of the structures will be estimated with the DAD methodology developed in deliverable 3.3. For this KPI, some of the UHDC pilots of WP8 can be designed also with HPC and their service life can be compared.

In the definition of the service life of the whole structure should be considered:

- The effect of the microcracking in the localized and general corrosion of the steel bars, and in the evolution of corrosion, when conventional reinforcement, either passive or active, is employed.
- The influence of the robustness of each type of concrete in the variation of durability and mechanical properties.
- The influence on the durability of the variation of the cover under standard levels of control in production.
- The medium and long-term effect of the self-healing capacities of the concrete.

- **KPI2b: Reduction of 50% of the maintenance costs**

This KPI compares the maintenance costs of a structure designed with HPC and the same design with UHDC. For this evaluation, the first 50 years of service life of the structure will be considered. The parameter used will be the percentage of the total maintenance costs with regard to the construction costs of the structure. In the case of HPC, the value will be obtained from the state of art of the HPC structures analysed by different international authors. In the case of UHDC, this percentage will be calculated for similar structures designed with this material considering the self-healing capacities, multi-micro cracking, slower chloride penetration and higher structure resilience. To achieve KPI2b, the percentage of maintenance costs of the UHDC structure should be less than half of the value obtained for HPC structures.

- **KPI3: 75% of accuracy of any modelling**

The theoretical models to evaluate the long-term behaviour of UHDC structures under EAE should estimate with at least a 75% of accuracy the real progression of the phenomena (corrosion degradation, self-healing capacities, life cycle analysis costs). Each of the pilots of WP8 will be used to compare for each parameter the estimation of the theoretical models with its progressive evolution in the structure. The reliability of the comparison will be higher after a considerable time obtaining empiric data from the pilots, so measurements will be obtained also after the end of the project.

- **KPI4: 8 Business Plans, one per industrial partner**

In the context of WP7 and the PEDR (WP9), one Business Plan will be developed during the project for each partner, embracing market niches/sectors where the developments can be exploited, identifying the major risks and constraints for technology implementation.

- **KPI5: 300 new subscribers per year to the newsletter (1200 at the end of the project)**

This Communication KPI sets that the Newsletter (published quarterly) will receive at least 300 new subscribers per year, reaching 1200 at the end of the project. Subscribers will be recruited through the website, events and fairs, brochures and mailing lists. The communication team (WP9) will control through the website the evolution of the number of subscribers.

6. Requirements for design and durability performance evaluation of UHDC

6.1. Basic target values for the UHDC design

To reach the KPI basic values and recommendations for the design of the concretes within WP4 and in relation to the specific exposure conditions, the most commonly recommended values for the basic parameters of the concretes are included in *Table 22*, taking into account as a reference high quality concrete a HPC with the target values already accepted in standards and practice, summarized in *Table 21*.

Table 21 *Target values for HPC reference concrete in ReSHEALience*

Pilot/ environment	Concrete Class	Workability (mm)	w/c	Fibers	Reinforcement	Binder type
OPC	C80	SCC (700)	<0.4	no	yes	CEM I

Table 22. *Requirements for UHDC design in WP4*

Pilot/ environment	Concrete Class	Workability (mm)	w/c	Fibers	Reinforcement	Binder type
Mud and water basin in a geothermal plant/XA	C100	SCC (700)	≤0.2	yes	optional	CEMI + Blast Furnace slag
Valencia coast Aquaculture mussel raft/XS	C110-150	SCC (700)	≤0.2	yes	Yes	CEM I + Silica Fume
Valencia coast Offshore wind floater/XS	C100-150	SCC (700)	≤0.2	yes	yes	CEM I + Silica Fume
Atlantic Irish West coast/XS	C100	SCC (700)	≤0.2	Yes*	No	CEM I + Silica Fume
Public abattoir Damaged water tower/XS1**	Malta Columns:C120	SCC (700)	≤0.2	yes	No	CEM I + Silica Fume

*Possibly textile reinforcement will be used instead of short dispersed fibers

**Note: The different components of the Water Tower require different interventions. Drum and Conical components; option for high viscosity, textile reinforced high performance concrete.

6.2. Main Durability target values for the UHDCs

For the case of durability characterisation within WP5, the most commonly recommended indicators and the target values to reach are compiled in *Table 24* in order to reach the KPIs for long service life of concrete structures through the design of UHDC, also taking into account as a reference high quality concrete a HPC with the target values already accepted in standards and practice, summarized in *Table 23*.

Table 23 *Durability target values for UHPC reference concrete in ReSHEALience*

Pilot/ environment	Corrosion depth (mm)	Expansion (%)	D_{app} (m^2/s)	D_{mig} (m^2/s)	Porosity (%)	Gas permeability (m^2)	Capillary suction (%)	$Ca(OH)_2$ initial (kg/m^3)
HPC	2	0.1	$<10^{-12}$	10^{-11}-10^{-12}	10-13	10^{-17}	0.015	<90

Table 24. *Requirements for durability limit state for WP5*

Pilot /environment	Corrosion depth (mm)	Expansion (%)	D_{app} (m^2/s)	D_{mig} (m^2/s)	Porosity (%)	Gas permeability (m^2)	Capillary suction (%)	Ca(OH)2 initial (kg/m3)
Mud and water basin in a geothermal plant/XA	≤1	≤0.05	-	-	≤6	$<10^{-17}$	<0.01	<10
Valencia coast Aquaculture mussel raft/XS	-		$<10^{-13}$	$<10^{-12}$	≤6	$<10^{-17}$	<0.01	<20
Valencia coast Offshore wind floater/XS	-		$<10^{-13}$	$<10^{-12}$	≤6	$<10^{-17}$	<0.01	<20
Breakwater elements Atlantic Irish West coast/XS	-		$<10^{-13}$	$<10^{-12}$	≤7	$<10^{-17}$	<0.01	<20
Public abattoir Damaged water tower/XS1			$<10^{-13}$	$<10^{-12}$	≤7-8	$<10^{-17}$	<0.01	<20

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8. Annex

Table. *Target values 1*

Guideline	Type of concrete	Capillary coefficient ($g/m^2h^{0,5}$)	Capillary porosity (%)	Total porosity (%)	Water porosity (%)	Oxygen permeability (m^2)	Diffusion coef chloride ions (m^2/s)	Portlandite content (kg/m^3)
prSIA 2052	UPFRC	≤ 100						
AFGC	Ordinary concrete				14 - 20	10^{-16}	> 10^{-11}	76
	High perform concrete				10 - 13	10^{-17}	10^{-12} - 10^{-11}	86
	Very high perf concrete				6 - 9	10^{-18}	10^{-13} - 10^{-12}	66
	UHPFRC				1.5 - 5	< 10^{-19}	10^{-13}	< 20
Australia	Ductal prestressed concrete		< 1	2 - 6			0.2 10^{-12}	

Table 25. *Target values 2*

Guideline	Type of concrete	Carbonation depth (mm)	Freeze-thaw resistance	Abrasion resistance (loss index)	Salt-scaling resistance (loss of residue, g/m^2)
prSIA 2052	UPFRC				
AFGC	Ordinary concrete				
	High perform concrete				
	Very high perform concrete				
	UHPFRC				
Australia	Ductal prestressed concrete	< 0.5	100 % (after 300 cycles)	1.2	< 10

Table 26. *Maximum w/c*

	Code		XS1	XS2	XS3
DENMARK	DNV-OS-C502 2012 offshore concrete structures	maximum w/c ratio	<0,45	<0,45	<0,4
GERMANY	ZTV-W LB215 (BAW - Federal Waterways Engineering and Research Institute)	max w/c for all hydraulic structures	0.65	0.65	0.65
IRELAND	EC-ANNEX	maximum w/c ratio	0.5	0.5	0.45

SPAIN	EHE	maximum w/c ratio	type of concrete	mass	-	-	-
				reinforced	0.50	0.50	0.45
				pre-stressed	0.45	0.45	0.45

Code				XA1		XA2		XA3	
IRELAND	EC-ANNEX IRELAND	maximum w/c ratio		min. Strength class, min. Cem content, cem type/combinations					
				C32/40, 340	C30/37, 320	C35/45, 360	C30/37, 340	C40/50, 400	C32/40, 360
				CEMI CEM II/A-L(LL) CEM II/A-V CEM II/A-S	CEM I-SR, CEM I/B-V, CEM III/A, CEM III/B	CEM I, CEM II/A-L, (LL), CEM II/A-V, CEM IIA/A-S, CEM II/B-V, CEM III/A	CEM I-SR, CEM III/B	CEM I, CEM II/A-L, (LL), CEM II/A-V, CEM II/A-S, CEM II/B-V, CEM III/A	CEM I-SR, CEM III/B
				combination of CEMI, CEM II/A-L, LL, CEM II/A-V or CEM II/A-S with up to 49% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEMII/A-V or CEM II/A-S with (50-70)% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V OR CEM II/A-S with up to 65% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V, or CEM II/A-S with (66-70)% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V or CEM II/A-S with up to 65% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V or CEM II/A-S with (66-70)% total ggbs content
				0.5	0.55	0.5	0.5	0.45	0.45
SPAIN	EHE	maximum w/c ratio	type of concrete	mass	0.50	0.50		0.45	
				reinforced	0.50	0.50		0.45	
				pre-stressed	0.50	0.45		0.45	
GERMANY	ZTV-W LB215 (BAW - Federal Waterways Engineering and Research Institute)	max w/c for all hydraulic structures			0.65	0.65		0.65	

Table 27. *Minimum cement content*

	Code			XS1	XS2	XS3
DENMARK	DNV-OS-C502 2012 offshore concrete structures	type of concrete	reinforced	360 (max. Size 20 mm) 360 (max. Size 20-40 mm) Testing (max size > 40 mm)	360 (max. Size 20 mm) 360 (max. Size 20-40 mm) Testing (max size > 40 mm)	>400
			pre-stressed	360 (max. Size 20 mm) 360 (max. Size 20-40 mm) Testing (max size > 40 mm)	360 (max. Size 20 mm) 360 (max. Size 20-40 mm) Testing (max size > 40 mm)	>400
SPAIN	EHE	type of concrete	mass	-	-	-
			reinforced	300	325	350
			pre-stressed	300	325	350

	code	XA1		XA2		XA3	
IRELAND	EC-ANNEX IRELAND	w/c, min. Strength class, cem type/combinations					
		0.5	0.55	0.5	0.5	0.45	0.45
		C32/40	C30/37	C35/45	C30/37	C40/50	C32/40
		CEMI CEM II/A-L(LL) CEM II/A-V CEM II/A-S	CEM I-SR, CEM I/B-V, CEM III/A, CEM III/B	CEM I, CEM II/A-L, (LL), CEM II/A-V, CEM IIA/A-S, CEM II/B-V, CEM III/A	CEM I-SR, CEM III/B	CEM I, CEM II/A-L, (LL), CEM II/A-V, CEM II/A-S, CEM II/B-V, CEM III/A	CEM I-SR, CEM III/B
		combination of CEMI, CEM II/A-L, LL, CEM II/A-V or CEM II/A-S with up to 49% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEMII/A-V or CEM II/A-S with (50-70)% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V OR CEM II/A-S with up to 65% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V, or CEM II/A-S with (66-70)% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V or CEM II/A-S with up to 65% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V or CEM II/A-S with (66-70)% total ggbs content

					340	320	360	340	400	360
SPAIN		EHE	type of concrete	mass	275		300		325	
				reinforced	325		350		350	
				pre-stressed	325		350		350	

Table 28. *Stress limitation*

		XS1	XS2	XS3
EUROCODE	in the absence of other measures limit compressive stress $k_1 f_{ck}$*	0.6	0.6	0.6

		XA1	XA2	XA3
EUROCODE	in the absence of other measures limit compressive stress $k_1 f_{ck}$*	0.6	0.6	0.6

* the value of k1 for use in a Country may be found in its National Annex. The recommended value is 0.6.

Table 29. *Minimum compressive strength/strength class*

	Code			XS1	XS2	XS3
	Eurocode	indicative minimum strength class		C30/37	C35/45	C35/45
DENMARK	EC-Annex	indicative minimum strength class		35	35	40
	DNV-OS-C502 2012 offshore concrete structures	type of concrete	reinforced	>C35 (directly exposed)	>C25 (not directly exposed)	
			pre-stressed		>C30	
FINLAND	EC-ANNEX	indicative minimum strength class		C35/45	C35/45	C35/45
FRANCE	EC-ANNEX	indicative minimum strength class	non pre-fabricated elements	C30/37	C30/37	C35/45
			pre-fabricated elements	C35/45	C40/50	C40/50
	NF P18-710 and NF P18-470 (UHPC FRENCH STANDARD)	indicative minimum strength class		>C150	>C150	>C150

GERMANY	ZTV-W LB215 (BAW - Federal Waterways Engineering and Research Institute) Additional Technical Contractual Terms - Hydraulic Engineering		$f_{cm,cube,28d}$ [MPa]		≤ 46	≤ 49
SPAIN	EC-ANNEX	indicative minimum strength class	reinforced concrete		C30/35	C30/35	C35/40
			pre-stressed concrete		C30/35	C35/40	C35/40
	EHE	minimum strength (N/mm²)	type of concrete	mass	-	-	-
				reinforced	30	30	35
				pre-stressed	30	35	35

	Code			XA1	XA2	XA3
	Eurocode	indicative minimum strength class		C30/37	C35/45	C35/45
DENMARK	EC-Annex	indicative minimum strength class		35	35	40
FINLAND	EC-ANNEX	indicative minimum strength class	non pre-fabricated elements	C30/37	C35/45	C40/50
			pre-fabricated elements	C35/45	C35/45	C40/50
FRANCE	EC-ANNEX	indicative minimum strength class	non pre-fabricated elements	C30/37	C35/45	C40/50
			pre-fabricated elements	C35/45	C35/45	C40/50
GERMANY	EC-ANNEX	indicative minimum strength class		C25/30	C35/45	C35/45
	ZTV-W LB215 (BAW - Federal Waterways Engineering and Research Institute) Additional Technical Contractual Terms - Hydraulic Engineering	$f_{cm,cube,28d}$ [MPa]		≤ 43	≤ 46	≤ 49

IRELAND	EC-ANNEX IRELAND	w/c, min. Cem content, cem type/combinations				
		0.5	0.55	0.5	0.5	0.45	0.45
		340	320	360	340	400	360

					CEMI CEM II/A-L(LL) CEM II/A-V CEM II/A-S	CEM I-SR, CEM I/B-V, CEM III/A, CEM III/B	CEM I, CEM II/A-L, (LL), CEM II/A-V, CEM IIA/A-S, CEM II/B-V, CEM III/A	CEM I-SR, CEM III/B	CEM I, CEM II/A-L, (LL), CEM II/A-V, CEM II/A-S, CEM II/B-V, CEM III/A	CEM I-SR, CEM III/B
					combination of CEMI, CEM II/A-L, LL, CEM II/A-V or CEM II/A-S with up to 49% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEMII/A-V or CEM II/A-S with (50-70)% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V OR CEM II/A-S with up to 65% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V, or CEM II/A-S with (66-70)% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V or CEM II/A-S with up to 65% total ggbs content	combination of CEM I, CEM II/A-L, (LL), CEM II/A-V or CEM II/A-S with (66-70)% total ggbs content
					C32/40	C30/37	C35/45	C30/37	C40/50	C32/40
SPAIN	EC-ANNEX	indicative minimum strength class	reinforced concrete		C30/35		C30/35		C35/40	
			prestressed concrete		C30/35		C35/40		C30/35	
	EHE	minimum strength (N/mm^2)	type of concrete	mass	30		30		35	
				reinforced	30		30		35	
				pre-stressed	30		35		35	
ITALY	EC-ANNEX	indicative minimum strength class	non pre-fabricated elements		C30/37		C30/37		C35/45	

Table 30. *Total chloride content (percentage by weight of cement)*

Code		Type of concrete	XS1	XS2	XS3	XS1	XS2	XS3
			Fresh concrete (Chlorides provided by components)			Hardened concrete (chloride threshold)		
SPAIN	EHE-08	Reinforced concrete	0.4	0.4	0.4	0.6	0.6	0.6
		Prestressed concrete	0.2	0.2	0.2	0.3	0.3	0.3
DENMARK	DNV-OS-C502 2012	Reinforced concrete	< 0.1					
		Prestressed concrete	< 0.1					

FRANCE	PR NF P18-470 (draft) UHPFC	UHPFC without rebars, type M (steel fibres)	0.2
		Prestressed UHPFC, type M (steel fibre) or A (other fibre)	0.15
		Other UHPFC	0.4

	Code	Type of concrete	XA1	XA2	XA3	XA1	XA2	XA3
			Fresh concrete (Chlorides provided by components)			Hardened concrete (chloride threshold)		
SPAIN	EHE-08	Reinforced concrete	0.4	0.4	0.4	0.6	0.6	0.6
		Prestressed concrete	0.2	0.2	0.2	0.3	0.3	0.3
FRANCE	PR NF P18-470 (draft) UHPFC	UHPFC without rebars, type M (steel fibres)	0.2					
		Prestressed UHPFC, type M (steel fibre) or A (other fibre)	0.15					
		Other UHPFC	0.4					

Table 31. *Impermeability*

			XS1	XS2	XS3	XS1	XS2	XS3	XS1	XS2	XS3	XS1	XS2	XS3
	Code	Requirement	Water						Gas			Chlorides		
SPAIN			Water permeability under pressure (mm)			Water porosity (%)			Gas permeability (m^2)			Chloride diffusion coefficient (m^2/s)		
	EHE-08	Maximun penetration	50	50	30									
		Average penetration	30	30	20									
FRANCE	PR NF P18-470 UHPFC	Lifespan: 50 years				≤ 9	≤ 9	≤ 9	$\leq 9\ 10^{-19}$	$\leq 9\ 10^{-19}$	$\leq 9\ 10^{-19}$	$\leq 5\ 10^{-13}$	$\leq 5\ 10^{-13}$	$\leq 5\ 10^{-13}$
		Lifespan: 100 years				≤ 9	≤ 9	≤ 6	$\leq 9\ 10^{-19}$	$\leq 9\ 10^{-19}$	$\leq 9\ 10^{-19}$	$\leq 5\ 10^{-13}$	$\leq 5\ 10^{-13}$	$\leq 10^{-13}$
		Lifespan: 150 years				≤ 6	≤ 6	≤ 6	$\leq 10^{-19}$	$\leq 10^{-19}$	$\leq 10^{-19}$	$\leq 10^{-13}$	$\leq 10^{-13}$	$\leq 10^{-13}$

				XA1	XA2	XA3	XA1	XA2	XA3	XA1	XA2	XA3	XA1	XA2	XA3
Code		Requirement					Water				Gas			Chlorides	
				Water permeability under pressure (mm)			Water porosity (%)			Gas permeability (m^2)			Chloride diffusion coefficient (m^2/s)		
SPAIN	EHE-08	Reinforced	Maximun penetration	50	50	30									
			Average penetration	30	30	20									
		Prestress	Maximun penetration	50	30	30									
			Average penetration	30	20	20									
FRANCE	PR NF P18-470 UHPFC	Lifespan: 50 years					≤ 9	≤ 9 *	≤ 9 *	≤ 9 10^{-19}	≤ 9 10^{-19} *	≤ 9 10^{-19} *	≤ 5 10^{-13}	≤ 5 10^{-13} *	≤ 5 10^{-13} *
		Lifespan: 100 years					≤ 9 *	≤ 9 *	≤ 6 *	≤ 9 10^{-19} *	≤ 9 10^{-19} *	≤ 9 10^{-19} *	≤ 5 10^{-13} *	≤ 5 10^{-13} *	≤ 10^{-13} *
		Lifespan: 150 years					≤ 6 *	≤ 6 *	≤ 6 *	≤ 10^{-19} *	≤ 10^{-19} *	≤ 10^{-19} *	≤ 10^{-13} *	≤ 10^{-13} *	≤ 10^{-13} *

* cements according to section 5.1.2 of French standard NF P 18-459

Table 32. *Minimum concrete cover*

Code					XS1	XS2	XS3
Eurocode	**$c_{min,dur}$ (mm)**	reinforcement steel	structural class	S1	20	25	30
				S2	25	30	35
				S3	30	35	40
				S4	35	40	45
				S5	40	45	50
				S6	45	50	55
		prestressing steel	structural class	S1	30	35	40
				S2	35	40	45
				S3	40	45	50
				S4	45	50	55
				S5	50	55	60
				S6	55	60	65
	Δc_{dev} (mm)				10	10	10

DENMARK	EC-Annex	$c_{min,dur}$ (mm)		reinforcement steel		20	25	30
				prestressing steel	pre tensioned tendons not bundled	30	35	40
					post tensioned tendon in ducts	35	40	45
		Δc_{dev} (mm)				10	10	10
	DNV-OS-C502 2012 offshore concrete structures	$c_{min,dur}$ (mm)	service life 50 years	type of reinforcement	sensitive to corrosion	50	50	60
					slightly sensitive to corrosion	40	40	50
			service life 100 years	type of reinforcement	sensitive to corrosion	60	60	70
					slightly sensitive to corrosion	50	50	60
FINLAND	EC-ANNEX	$c_{min,dur}$ (mm)		reinforcement steel	concrete strength ≥40/50 (XS1) or ≥45/55 (XS2, XS3): -5	20	25	30
				prestressing steel	design working life 100 years: +5	30	35	40
		Δc_{dev} (mm)				10	10	10
FRANCE	EC-ANNEX	$c_{min,dur}$ (mm)	reinforcement steel	structural class	S1	20	25	30
					S2	25	30	35
					S3	30	35	40
					S4	35	40	45
					S5	40	45	50
					S6	45	50	55
			prestressing steel	structural class	S1	30	35	40
					S2	35	40	45
					S3	40	45	50
					S4	45	50	55
					S5	50	55	60
					S6	55	60	65
		Δc_{dev} (mm)				10	10	10
		$c_{min,dur}$ (mm)	reinforcement structural class		S1	10	15	15

	NF P18-710 and NF P18-470 (UHPC FRENCH STANDARD)					S2	15	15	20
						S3	15	20	20
						S4	20	20	20
						S5	20	20	25
						S6	20	25	25
				prestressing steel	structural class	S1	15	20	20
						S2	20	20	20
						S3	20	20	25
						S4	20	25	25
						S5	25	25	30
						S6	25	30	30
		Δc_{dev} (mm)					10, 5, 0	10, 5, 0	10, 5, 0
GERMANY	EC-ANNEX	**$c_{min,dur}$ (mm)**	reinforcement steel	structural class	S1	compressive strength ≥C45/55: -5	compressive strength ≥C40/50: -5	20	25	30
					S2			25	30	35
					S3			30	35	40
					S4			35	40	45
					S5			40	45	50
					S6			45	50	55
			prestressing steel	structural class	S1			30	35	40
					S2			35	40	45
					S3			40	45	50
					S4			45	50	55
					S5			50	55	60
					S6			55	60	65
		Δc_{dev} (mm)						10	10	10
						S3		+10	+5	0
ITALY	EC-ANNEX	**$c_{min,dur}$ (mm)**		reinforcement steel	structural class	S1		20	25	30
						S2		25	30	35
						S3		30	35	40

					S4	35			40			45		
					S5	40			45			50		
					S6	45			50			55		
			prestressing steel	structural class	S1	30			35			40		
					S2	35			40			45		
					S3	40			45			50		
					S4	45			50			55		
					S5	50			55			60		
					S6	55			60			65		
		Δc_{dev} (mm)				10			10			10		
SPAIN	EC-ANNEX	**$c_{min,dur}$ (mm)**	reinforcement steel	structural class	S1	30			30			verif. DLS		
					S2	35			35			verif. DLS		
					S3	40			35			verif. DLS		
					S4	40			35			verif. DLS		
					S5	50			45			verif. DLS		
					S6	60			60			verif. DLS		
			prestressing steel	structural class	S1	30/verif. DLS			30/verif. DLS			verif. DLS		
					S2	35/verif. DLS			35/verif. DLS			verif. DLS		
					S3	40/verif. DLS			35/verif. DLS			verif. DLS		
					S4	40/verif. DLS			35/verif. DLS			verif. DLS		
					S5	50/verif. DLS			45/verif. DLS			verif. DLS		
					S6	60/verif. DLS			60/verif. DLS			verif. DLS		
		Δc_{dev} (mm)				**$\Delta c_{dur,\gamma}$**	**$\Delta c_{dur,st}$**	**$\Delta c_{dur,add}$**	**$\Delta c_{dur,\gamma}$**	**$\Delta c_{dur,st}$**	**$\Delta c_{dur,add}$**	**$\Delta c_{dur,\gamma}$**	**$\Delta c_{dur,st}$**	**$\Delta c_{dur,add}$**
					reinforced other cement	5	-25	-25	verif. DLS	verif. DLS	verif. DLS	verif. DLS	verif. DLS	verif. DLS
					prestressed other cement	verif. DLS			verif. DLS			verif. DLS		
					reinforced adequate cement	-5	-5	-5	0	-5	-10	verif. DLS	verif. DLS	verif. DLS
					prestressed adequate cement	0			-5			verif. DLS		
	EHE	c (mm)	reinforc	CEM III, CEM IV, CEM II/B-S, B-P, B-	structural life (years)	50	25		30			35		
						100	30		35			40		

			V, A-D or concrete with micro-silica									
			other useable cements		50		45		40		inadvisable	
					100		65		inadvisable		inadvisable	
		pre-stressed	CEMI/A-D, or with silica fume additive of more than 6%	structural life (years)	50		30		35		40	
					100		35		40		45	
			other useable cements according to article 26		50		65		45		inadvisable	
					100		inadvisable		inadvisable		inadvisable	
SWEEDEN	EC-ANNEX	**$c_{min,dur}$ (mm)**							w/c			
						0.45	0.4	0.45	0.4	0.35	0.4	0.35
				working life (years)	100	30	25	50	45	40	45	40
					50	25	20	40	35	30	35	30
					20	15	15	30	25	25	25	25
		Δc_{dev} (mm)					10		10		10	

Table 33. *Maximum crack width*

Code		Type of concrete		XS1	XS2	XS3
Eurocode		reinforced members and presstressed members with unbonded tendons		0.3	0.3	0.3
		quasi-permanent load combination				
		prestressed members with bonded tendons		decompression	decompression	decompression
		frequent load combination				
DENMARK	EC-ANNEX	Unstressed reinforcement		0.3	0.3	0.2
		Tendons		0.2	0.2	0.1
	DNV-OS-C502 2012 offshore	type of reinforcement	sensitive to corrosion	0.2	0.2	0.2
			slightly sensitive to corrosion	0.3	0.4	0.3

	concrete structures					
FINLAND	EC-ANNEX	reinforced concrete and unbonded tendons	quasi-permanent	0.2	0.2	0.2
		Pre-stressed members with bonded tendons	frequent	decompression	decompression	decompression
FRANCE	EC-ANNEX	reinforced members and presstressed members with unbonded tendons		0.2	0.2	0.2
		quasi-permanent load combination				
		prestressed members with bonded tendons		decompression	decompression	decompression
		frequent load combination				
	PR NF P18-470 (draft) UHPFC	UHPFC without rebars		0.1	0.1	0.1
		Reinforced UHPFC		0.1	0.1	0.1
		Prestressed UHPFC (active bars without bond		0.1	0.1	0.1
		Prestressed UHPFC		limited tensile	limited tensile	limited tensile
GERMANY	EC-ANNEX	Reinforced concrete and unbonded tendons	Quasi-permanent load combination	0.3	0.3	0.3
		Post-stressed elements	Frequent load combination	0.2	0.2	0.2
		Pre-stressed members with immediate association		decompression	decompression	decompression
			Rare load combination	0.2	0.2	0.2
SPAIN	EC-ANNEX	reinforced members and presstressed members with unbonded tendons		0.2	0.2	0.1
		quasi-permanent load combination				
		prestressed members with bonded tendons		decompression	decompression	decompression
		frequent load combination				
	EHE-08	Reinforced concrete		0.2	0.2	0.1
		Prestressed concrete		Compression	Compression	Compression

SWEEDEN	EC-ANNEX			Corrosion sensitive	Lightly corrosion sensitive	Corrosion sensitive	Lightly corrosion sensitive	Corrosion sensitive	Lightly corrosion sensitive
		working life (years)	100	0.15	0.2	0.15	20	0.1	0.15
			50	0.2	0.3	0.2	0.3	0.15	0.2
			20	0.3	0.4	0.3	0.4	0.2	0.3

Code		Type of concrete	XA1	XA2	XA3
	Eurocode	reinforced members and presstressed members with unbonded tendons	0.3	0.3	0.3
		quasi-permanent load combination			
		prestressed members with bonded tendons	decompression	decompression	decompression
		frequent load combination			
SPAIN	EHE-08	Reinforced concrete	0.2	0.1	0.1
		Prestressed concrete	Compression	Compression	Compression