ACES

Towards Improved Assessment
of Safety Performance for LTO of Nuclear
Civil Engineering Structures

Overview of state-of-the-art knowledge for the quantitative assessment of the ageing/deterioration of concrete in nuclear power plant systems, structures, and components

D1.1
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1. About ACES

ACES addresses the EURATOM Work Programme 2019-2020, dedicated to Nuclear Fission and Radiation Protection Research (H2020 NFRP-2019-2020). Specifically, the proposal addresses the following work programme topic: A - Nuclear Safety - NFRP 1: Ageing phenomena of components and structures and operational issues.

The main objective of ACES is to advance the assessment of safety performance of civil engineering structures by solving the remaining scientific and technological problems that currently hinder the safe and long-term operation of nuclear power plants reliant on safety-critical concrete infrastructure. Proper understanding of deterioration and ageing mechanisms requires a research strategy based on combined experimental and theoretical studies, following a multidisciplinary approach, and utilizing state of the art experimental and modelling techniques. Material characterization at different length scales (i.e. nano, micro, meso, and macro scales) is necessary, focusing on the physical understanding of the degradation processes (e.g. neutron and gamma radiation, internal swelling reactions, liner corrosion, etc.) as well as physical phenomena (drying, creep, shrinkage, etc.), and their influence on macroscopic mechanical properties and structural/functional integrity of the components.

The ACES project aims to have a significant impact on the safety of operational Gen II and III NPPs and impact the design of next-generation plants. ACES will improve the understanding of ageing/deterioration of concrete and will demonstrate and quantify inherent safety margins introduced by the conservative approaches used during design and defined by codes and standards employed throughout the life of the NPP. The outcomes from ACES will therefore support the LTO of NPPs. This will be achieved by using more advanced and realistic scientific methods to assess the integrity of NPP concrete infrastructure. The project will provide evidence to support the methods by carrying out various tests, including large scale tests based on replicated scenarios of NPPs.

ACES engages 11 partners from five EU Member States (BE, CZ, FI, FR, SI) and two non-EU countries (UA and USA).

2. Partners

VTT ENGIE CVŘ CTU CZECH TECHNICAL UNIVERSITY IN PRAGUE cea

EDF IRSN INSTITUT DE RADIOPROTECTION ET DE SÛRETÉ NUCLÉAIRE ENERGORISK sck cen ZAG

3. International Partners

OAK RIDGE National Laboratory

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N. & Name	**D1.1 Overview of state-of-the-art knowledge for the quantitative assessment of the ageing/deterioration of concrete in nuclear power plant systems, structures, and components**				
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Summary

The ageing management of nuclear power plants (NPPs) is a requirement based on national regulatory guidelines, and the recommendations of the International Atomic Energy Agency. The report focusses on the ageing processes and management of concrete systems, structures or components in NPPs with two main objectives:

(i) a critical interpretation of ageing management guidance and practice, with special focus on reinforced concrete structures, and

(ii) an overview of the main concrete degradation processes

Based on existing international and national guidelines on ageing management approaches, a generic approach for ageing management and guidelines are given specifically for reinforced concrete structures (RCSs). The report gives information on the generic approach for ageing management, guidelines, and through-life management of concrete structures.

The environmental loading conditions of NPP RCSs are described, in particular in relation to the potential ageing and deterioration-related mechanisms that might affect RCSs.

Another main part of this report reviews the most common and likely ageing/deterioration mechanisms to affect NPP RCSs: acid attack, leaching, external sulphate attack, carbonation, alkali-aggregate reactions, delayed ettringite formation, bacterial processes, freeze-thaw, elevated and high temperatures, irradiation, abrasion-erosion-cavitation, creep-relaxation, settlement-movements, vibration, thermal stresses, pitting corrosion, general corrosion, crack corrosion, and crevice corrosion.

For each of the ageing processes, a short process description is given, as well as influential factors and effects of other ageing processes. Some general guidelines for deterioration rates are given, if information is available. Furthermore, assessment methods, performance indicators & acceptance criteria are discussed whenever possible, as well as an indication of current modelling capabilities.

In a final chapter, high level knowledge gaps and challenges are listed.

List of abbreviations

AAR	Alkali-aggregate reactions
ACR	Alkali-carbonate reactions
AE	Acoustic emission
AMP	Ageing management programme
AMRs	Ageing management reviews
ASR	Alkali-silica reactions
BFS	Blast furnace slag
BIM	Building information modelling
BWR	Boiling water reactor
CANDU	Canada Deuterium Uranium (Canadian PHWR)
CBS	Concrete biologically shield
CMEA	Coupled multi-electrode array
C-S-H	Calcium silicate hydrates
DEF	Delayed ettringite formation
EIS	Electrochemical impedance spectroscopy
ER	Electrical resistivity
FA	Fly-ash
IAASR	Irradiated-assisted alkali-silica reaction
IAEA	International Atomic Energy Agency
IGALL	International Generic Ageing Lessons Learned
ITZ	Interfacial transition zone
LOCA	Loss-of-coolant accidents
LRAs	License renewal applications
LTO	Long term operation
LVDT	Linear variable differential transformer
LWR	Light water reactors
MIC	Microbiological induced corrosion
MICP	Microbiologically-induced calcite precipitation
NDT	Non-destructive testing
NPP	Nuclear power plant
OpEx	Operational Experience
PCCV	Pre-stressed concrete containment vessel
PHWR	Pressurized heavy-water reactor
PIM	Plant information model
PR	Potentially reactive
PRP	Potentially reactive with pessimum effect
PWR	Pressure water reactor
R&D	Research and development
RCSs	Reinforced concrete structures
RH	Relative humidity
RIVE	Radiation-induced volumetric expansion
RPV	Reactor pressure vessel
SALTO	Safety aspects of long-term operation
SEM	Scanning electron microscope
SF	Silica fume
SSCs	Systems, structures and components
SSS	Stress strain state
TLAA	Time-limited ageing analysis
VVER	Vodo-Vodyanoi energetichesky reaktor (water-water power reactor)
w/b	Water/binder ratio
w/c	Water/cement ratio

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1. Introduction

The ageing management of nuclear power plants (NPPs) is a requirement based on national regulatory guidelines, and the recommendations of the International Atomic Energy Agency (IAEA) [1, 2]. Well-conceived, executed, and documented ageing management is critical to successfully maintaining NPP structures, especially when considering life-extension beyond 40 years, as is the new reality for many European NPPs.

Ageing management is part of a NPP's own safety management system. The focus of an AMP is to continuously acquire knowledge and information concerning the ageing mechanisms acting on systems, structures, and components (SSCs), and to provide assistance in implementing ageing management activities at different phases throughout the lifetime of a NPP renewal [3, 4]. Sufficiently accurate and relevant information on the long-term condition of SSCs should be readily available to be able to demonstrate their durability and estimate their future lifetime. This provides a solid basis for the decision-making process.

The division of an NPP into SSCs naturally includes civil engineering infrastructure, and of specific interest in this report are reinforced concrete structures (RCSs). The RCSs of NPPs provide foundation, structural support, shielding, and containment functions. Examples of RCSs important to NPP safety include the containment building, spent fuel pool, and cooling channels/towers/pools.

AMPs must provide assurances that the NPP RCS will continue to meet their functional and performance requirements and maintain adequate structural margins during the current licensing period, as well as for continued service periods beyond the initial operating license period that may extend plant operation to 60 or more years [5]. Ageing management activities include planning and monitoring activities during each of the phases in the life of an NPP: design, construction, operation (maintenance, inspection and intervention), and decommissioning/demolition.

Age-related degradation of RCS may affect engineering properties, structural resistance/ capacity, failure mode, and location of failure initiation that in turn may affect the ability of a structure to withstand challenges in service [6]. In order to ensure the safe operation of NPPs, it is essential that the effects of potential degradation of SSCs be assessed and managed during both the current operating license period as well as subsequent license renewal periods. In contrast to many mechanical and electrical components, replacement of many concrete structures is impractical or economically unviable. Therefore, it is necessary for safety issues related to NPP ageing and continued service of RCSs to be resolved through sound scientific and engineering understanding [5].

Through appropriate AMP, service life can be guaranteed proactively for NPP concrete structures, based on access to the structural, material, and environmental information. A comprehensive approach provides systematic methods for planning, surveillance (performance estimation), inspection, monitoring, condition assessment, maintenance, and repair of structures. Success is based on regular and continuous evaluation of ageing aspects, timely implementation of the necessary measures, and ongoing monitoring to ensure their effectiveness. The compilation of RCS SSC-relevant knowledge/information helps demonstrate the efficiency of ageing management, which in turn provides the underlying support for NPP management.

How well each RCS is designed, constructed, inspected, and maintained all directly influence its long-term performance. Information obtained from operational experience (e.g., inspections) helps determine the rate of structure deterioration. Incorporating information or knowledge gained from the earlier phases in the life of the RCS can assist in the decision-making process for future phases. Identification of potentially weak areas (e.g., high permeability and/or shallow cover) and estimation of the deterioration rate in advance of actual damage both allow for more accurate planning of funding requirements for future conservation, intervention, and, if desired, replacement activities. For RCSs, an AMP provides a framework for establishing and documenting these design decisions, the quality of workmanship, and the materials used during construction, as well as scheduled (and unscheduled) maintenance and performance and condition assessment activities that determine the remaining service life of structures [7].

Objective of report

This report has been written with the intention of providing a critical interpretation of ageing management guidance and practice, with special focus on reinforced concrete structures (RCS).

Furthermore, the report provides an updated review of main deterioration mechanism of reinforced concrete that might affect NPPs, and how they impact the ageing process of SSCs. For each of these, key knowledge gaps, especially when considering operations beyond the design life of current plants are identified, in addition to recommendations.

The report is written with the owners of NPPs in mind.

1.1 Scope of the document

This document provides a state-of-the-art report on ageing management of NPP RCSs. The scope of the document covers RCS that comprise the infrastructure of NPPs, with special emphasis on the safety-related concrete SSCs.

This document is not meant as a technical guideline as the report complements current ageing management guidelines and practices, highlighting the relevance for RCSs, from design and construction through to (long-term) operation and finally decommissioning.

The environmental loading conditions of NPP RCSs are described, in particular in relation to the potential ageing and deterioration related mechanism that might affect RCSs. Accident related loading conditions such as earthquakes, tornado, fires, etc. are not covered here.

Finally, this report reviews the most common and likely ageing/deterioration mechanism to affect NPP RCSs. A brief process description is provided, as well as influential environmental factors. For each ageing/deterioration mechanism, assessment methods, performance indicators & acceptance criteria are discussed whenever possible, as well as an indication of current modelling capabilities, and finally of current challenges and knowledge gaps.

2. Ageing management of concrete SSCs in NPP

2.1 Background on ageing management for NPPs

The tendency of NPP components to undergo changes in their properties during the course of the plant lifetime is generally termed ageing. Ageing is, therefore, a complex combination of factors concerned with materials, environments, and their time dependant interactions. Ageing Management is defined in the IAEA Safety Glossary [8] as “Engineering, operations and maintenance actions to control within acceptable limits the ageing degradation of structures, systems and components”, so that required safety functions of systems, structures and components are fulfilled over the entire operating lifetime of the plant [9].

A NPP has a large number and variety of concrete SSCs, and some of these have important safety functions. Among the SSCs of interest are the concrete containment, the concrete biological shield, the reactor building, spent fuel pools, cooling towers, water intake/outflow structures, etc. (varying according to NPP design). Table 1 lists typical safety-related concrete structures in light-water reactor plants and their accessibility for examination [10].

RCSs in NPPs are unique, characterized by large wall thickness (typically >1.0 m), dense and complex reinforcement detailing, existence of penetrations or cast-in-place items, limited accessibility (i.e. due to liners or other components), and adverse operating environments (e.g. radiation) [11]. The safety-related RCSs[1] perform multiple functions, e.g. load-bearing, radiation shielding, and leak prevention. They also play a key role in mitigating the impact of extreme environmental and loading events such as earthquakes, winds, fire, floods, aircraft impacts, etc., or internal hazards such as fires, explosions, floods, etc. on NPP safety.

RCSs in NPPs are composed of several constituents, including concrete, conventional steel reinforcement, pre-stressed steel, steel liner plates, and structural steel. While unique in application, they share many physical characteristics with conventional concrete structures. Experience shows that ageing degradation[2] of RCSs can be a result of exposure to aggressive environments, excessive structural loads, accidental conditions, use of unsuitable materials, poor material and construction quality, and the lack of or inadequate maintenance.

As concrete ages, changes in its properties will occur naturally as a result of continuous microstructural changes (e.g., due to cement hydration, crystallization of amorphous constituents, reactions between cement pore solution and aggregates, etc.), as well as environmental interaction leading to adverse performance of the cement paste matrix and aggregates under physical or chemical attack (e.g., internal expansion, cracking, leaching, etc.). The effect of age-related degradation often leads to a reduction in mechanical and durability properties of RCSs, which could result in their inability to meet functional or performance requirements.

The safety of NPPs during long term operation (LTO) has become more important given the current economic situation and energy production needs, where NPP owners are prioritising extended operation beyond the initial design life of NPPs [12]. Many existing NPPs, having entered operation in the 1970s often with an originally operating license of 30-40 years, have

[1] Concrete structures include both simple structures and complex structures that consist of structural components. The terms RCS used in this document refers to all safety-related structures and structural components made of reinforced and/or pre-stressed concrete in NPPs.

[2] Ageing degradation, sometimes simplified as ageing or degradation in this document, is a general process in which the performance characteristics of the structure gradually deteriorate with time or use owing to physical, chemical, or biological processes.

already renewed their operating licences once, extending their operation to 50-60 years.

Table 1. Typical safety-related concrete structures in LWR plants and their accessibility for visual examination [10].

Concrete Structure	Accessibility
Primary containment	
• Containment dome/roof	• Internal liner/complete external
• Containment foundation/basemat	• Internal liner (not embedded) or top surface
• Slabs and walls	• Internal liner/external above grade
Containment internal structures	
• Slabs and walls	• Generally accessible
• Reactor vessel support structure (or pedestal)	• Typically lined or hard to access
• Crane support structures	• Generally accessible
• Reactor shield wall (biological)	• Typically lined
• Ice condenser dividing wall (ice condenser plants)	• Lined or hard to access
• NSSS equipment supports/vault structures	• Generally accessible
• Weir and vent walls (Mark III)	• Lined with limited access
• Pool structures (Mark III)	• Lined
• Diaphragm floor (Mark II)	• Lined with limited access
• Drywell/wetwell slabs and walls (Mark III)	• Internal liner/partial external access
Secondary Containment/Reactor Buildings	
• Slabs, columns, and walls	• Accessible on multiple surfaces
• Foundation	• Top surface
• Sacrificial shield wall (metallic containments)	• Internal lined/external accessible
Fuel/Equipment Storage Pools	
• Walls, slabs, and canals	• Internal lined/partial external
• Auxiliary building	• Generally accessible
• Fuel storage building	• Generally accessible
• Control room (or building)	• Generally accessible
• Diesel generator building	• Generally accessible
• Piping or electrical cable ducts or tunnels	• Limited accessibility
• Radioactive waste storage building	• Generally accessible
• Stacks	• Partial internal/external above grade
• Intake structures (inc. concrete water intake piping and canal embankments)	• Internal accessible/external above grade and waterline
• Pumping stations	• Partially accessible
• Cooling towers	• Accessible above grade
• Plant discharge structures	• Internal accessible/external above grade and waterline
• Emergency cooling water structures	• Limited accessibility
• Dams	• External surfaces above waterline
• Water wells	• Limited accessibility
• Turbine building	• Generally accessible

Further renewal could extend an NPP's operation from 60 to 80 years [13]. To obtain licence renewal, an NPP must provide an assessment of the technical aspects of plant ageing and show how ageing issues are being managed to maintain the structural performance for the intended functions.

All the challenges RCSs face in NPPs require a systematic approach for through-life management that covers all activities that aim to prevent or control ageing effects of RCS, within acceptable limits. The primary objective of ageing management for RCSs in NPPs is to ensure timely detection and mitigation of degradation that could have an impact on safety functions. The focus should be both on:

- Ensuring that the RCS have sufficient structural margins to continue to perform in a reliable and safe manner, and
- Identifying environmental stressors or ageing factors before they reach sufficient intensity to potentially degrade structural components.

The IAEA Safety Guide NS-G-2.12 [1], as well as a more recent IAEA specific safety guide SSG-48 [12], provide a generic approach for effective ageing management of NPP SSCs. Following the same methodology, this chapter highlights those key elements of effective ageing management and seeks to contextualize them from a perspective related to RCSs.

In section 2.2, the generic approach for ageing management in NPPs is described, which provides a common basis for the remaining sections in this chapter. Section 2.3 focuses on the guidelines for effective ageing management, while in section 2.4 ageing management related activities from RCSs perspective at each stage in the lifetime of a NPP are elaborated on further.

2.2 Generic approach for ageing management

Effective ageing management requires the use of a systematic approach to managing the ageing effects and provides a framework for coordinating all activities relating to the understanding, prevention, detection, monitoring, and mitigation of ageing effects on the concrete structures in NPPs [12]. These activities are reformulated in the generic approach for ageing management of NPP SSCs recommended in [1, 12], and presented as five activities.

Of the five activities, 'UNDERSTAND' is at the heart, essential to ensuring effective ageing management. The remaining four activities, 'PLAN', 'DO', 'CHECK', and 'ACT', coordinate with the key activity to form a feedback loop. Figure 1 illustrates the relationships among the five activities as in [1, 12], but has been adapted slightly to fit an RCS perspective.

The 'UNDERSTAND' activity implies a comprehensive understanding of the RCS, including the degradation mechanisms and ageing effects, and how these affect the capability of the structure to perform its function(s). This is a prerequisite for the systematic ageing management process shown in Figure 1. Developing appropriate understanding is a continuous process based on review of a large body of knowledge, including historical information related to the specific concrete structure, related R&D results, and operational experience (OpEx) from a variety of sources, both domestic and international.

The level of understanding depends to a large extent on the degree of technical and/or scientific knowledge development for relevant ageing mechanisms, and on the quality and quantity of relevant historical data. Understanding ageing enables the prediction of future ageing degradation. This predictability in turn enables the optimization and coordination of activities aimed at detecting, assessing, and mitigating ageing.

The 'PLAN' activity involves coordinating, integrating, and modifying existing programmes and activities relating to the ageing management of RCSs, and developing new programmes if necessary.

The 'DO' activity involves preventing and mitigating expected ageing effects and degradation mechanisms by developing utilization and environmental control programmes, and/or by means of other prevention and mitigation actions.

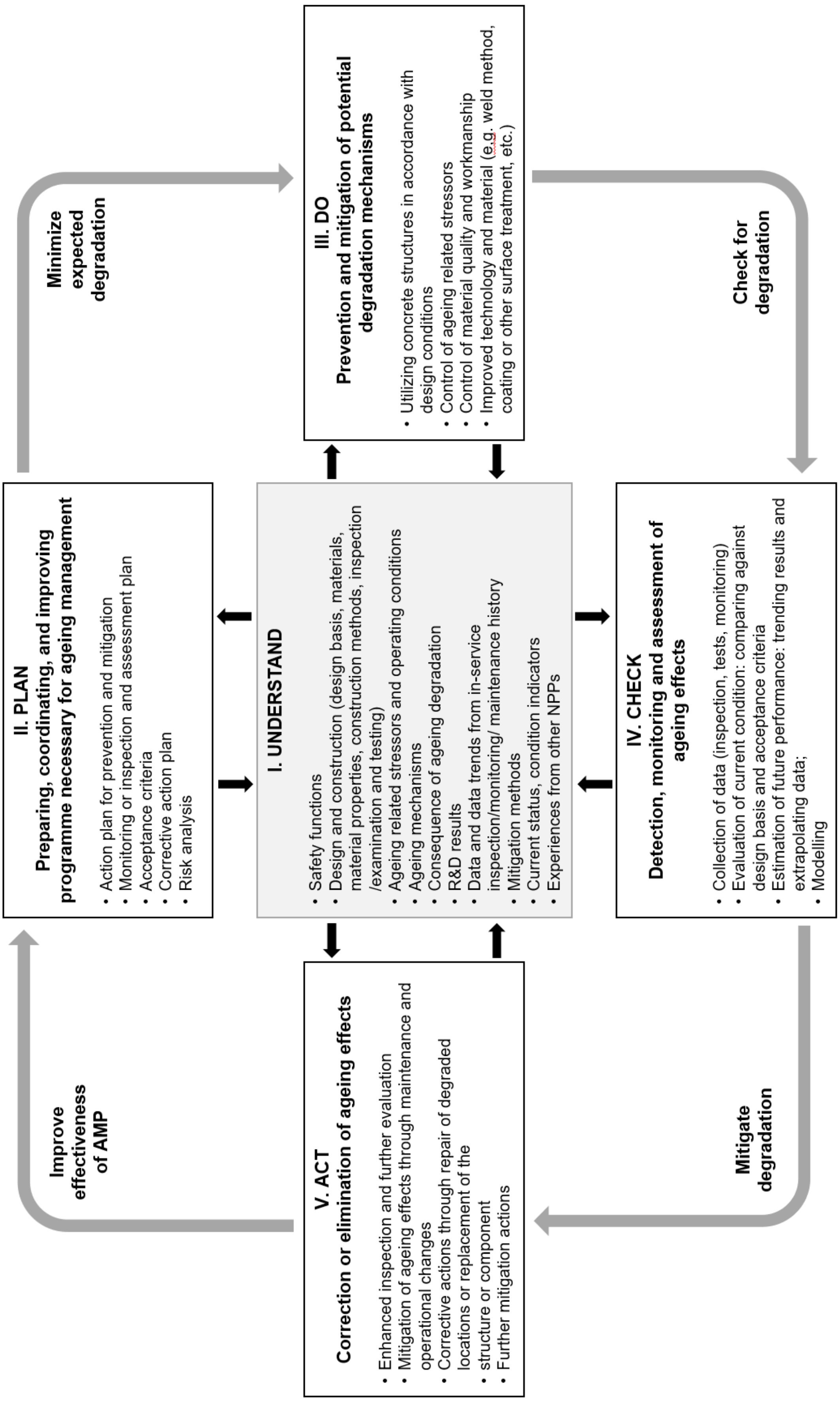


Figure 1. Systematic approach for ageing management of concrete SSCs (adapted from [1, 12, 14]).

The ‘CHECK’ activity involves the timely detection and characterization of significant ageing effects and degradation mechanisms through inspection and monitoring of RCSs, and the assessment of observed ageing effects to determine the type and timing of any corrective actions required.

The ‘ACT’ activity involves the timely correction of ageing effects on RCSs, and the introduction of further prevention or mitigation actions through appropriate maintenance and design modifications, including the repair and replacement of RCSs.

The closed loop formed in Figure 1 is indicative of the continuous improvement required for effective ageing management on the basis of feedback, wherein results from research and development (R&D), self-assessment, and peer reviews help to ensure that ageing will be detected and appropriately addressed in a timely manner.

2.3 Guidelines for effective ageing management of NPP RCSs

Effective ageing management of safety-related RCSs is an important element of the safe and reliable operation of NPPs. Based on chapter 5 in [12], which provides details guidelines for effective ageing management of NPPs, this chapter highlights those relevant to concrete structures.

2.3.1 Organizational Arrangements (qualified personnel)

The IAEA Safety Guide NS-G-2.12 [5] and the more recent IAEA Safety Guide SSG-48 [12] provide recommendations for organizational arrangements, including designation of an ageing management coordinator and documentation of team responsibilities and communication practices.

Ageing issues pertinent to concrete structures are complex, and the participants of ageing management teams should include experts from interdisciplinary fields. It is important that setting up and implementing the ageing programme is performed under the direction of personnel with appropriate engineering knowledge and experience. In addition, external organizations may be requested to provide expert services on specific topics, such as condition assessments and R&D. Section 7 of [15] gives guidelines for the qualifications of the evaluation team, including those who perform the inspections as well as those who perform an evaluation. Information regarding inspections and evaluations for the safety-related concrete structures in NPPs is covered in section 2.4.3.2 and section 2.4.3.3.

2.3.2 Information management system

Integrity and validity of information over time are essential to understanding ageing trends and providing vital information to assist in controlling and mitigating ageing effects. The information management system aims to collect, record, and classify all technical data produced throughout the service life of the NPP from design to decommissioning, making it easier to trace the historical information.

For NPPs commissioned before 1990, much of this information was initially provided on paper, or through multiple information systems and databases from different vendors with different purposes. Most of these systems are not integrated with one another and cannot easily share data during different phases of the NPP life cycle. With the exponential growth in computer technologies combined with development of standards for exchange of information, it is now possible to apply information technologies to form a virtual plant information environment, which can support effective, sustainable information interoperability, and thus streamline historically fragmented information systems [16].

The building information modelling (BIM) concept in the construction industry provides an example related to construction, in particular regarding architecture, civil, and structural engineering. Its effectiveness supports the implementation of a plant information model (PIM) and a knowledge-centric plant information model (K-PIM) to provide the nuclear industry with a practical, standardized, and portable information and knowledge repository. By connecting relevant associations of information objects with knowledge-centric named relationships, new knowledge-based services such as K-PIM shift the human oriented search-filter-extract methodology to a more automated, reliable computer assisted approach [16]. With design, construction, inspection, monitoring, maintenance, and repair information specific to the components of concrete structures continuously being entered into the information management system, this method allows for a high level of traceability of all information as it is consistently created, validated, modified, and reported.

Agreement on the scope and format of information to be shared, exchanged, and transferred over the NPP service life should be established in the pre-design stage[3]. This process can be facilitated by the introduction of the K-PIM concepts mentioned above. It is important to carefully select enabling IT technologies, methods, and processes for optimal knowledge capture and storage, to ensure that key information from formal documents as well as unstructured sources such as memoranda, discussions, e-mails, and research is captured and remains accessible for later retrieval and use.

A key element of ageing management is the systematic and rigorous condition assessment of SSCs, which generally involves a review of historical information together with collection of new inspection and monitoring data, to assess the effect of age-related degradation on structures, establish their current condition, and predict future performance. In the case of ageing management of concrete structures, K-PIM should be able to provide trackable information on such relevant aspects as listed in section 20.

2.3.3 Screening of safety-related NPP RCSs

An NPP has a large number and variety of RCSs. Those RCSs that are susceptible to ageing degradation and have a critical impact on the safety of the plant should be screened for ageing management.

A generic methodology for identifying SSCs subject to ageing management (also called ‘scoping’) is given in [1] and Fig. 3 in [12] (see Appendix in section 7.1.1). Ensuing the same approach, the following procedure is recommended for NPP RCSs:

- From a list of all SSCs of a NPP, identify those that are important to safety, on the basis of whether its malfunction or failure could lead to the loss or impairment of a safety function. Typical reinforced concrete safety-related SSCs in NPPs include, for example, the concrete biological shield, containment structure, spent fuel pools, cooling towers, water intake structures, concrete piping, etc.
- For each of safety related SSCs, list all concrete structures/components. Identify those that play key roles in maintaining safety or other important functions (for example, Table 1 in [11] (see Appendix in section 7.1.2) lists the typical safety-related concrete structures in pressurized water reactor and boiling water reactor plants).
- From the list of the identified safety-related concrete structures/components, list their functions, materials of construction (including mix design, cement characteristics, aggregates characteristics, concrete cover depth, type of steel, etc…), environmental

[3] The pre-design phase normally spans the period from the NPP construction decision point until the selection and engagement of the engineering, procurement, and construction (EPC) contractor and/or reactor designer. It is estimated that up to 70% or more of the basic knowledge that will guide the NPP through its life cycle is acquired during this period.

exposure and applied loads. Identify those for which ageing degradation has the potential to cause component failure.

• Finally, the safety critical concrete structures/components should be arranged into groups of components and structures that have similar characteristics, such as similar functions, similar materials, and/or similar environments (e.g. a group of foundations, piles, and underground structures).

2.3.4 Collation of background data

A typical procedure for setting up an AMP, when one does not previously exist, includes gathering background and historical documentation, performing a baseline condition assessment, and developing the AMP based on the condition assessment results [2].

Details of concrete structure design, construction, and operational history including inspections performed and corrective actions and/or repairs made are required for condition assessment and ageing management review. Table 11 in [2] (see Appendix in section 7.1.3) provides a list of data sources for such details, including plant records and architect/engineer or technical service organization/consultant files. In a modern NPP, this data should be continuously inserted into an information management system (section 2.3.2), which allows for easier recording and retrieval of information specific to the applicable structures.

Historical data may be grouped into four types: baseline, construction, operational history, and inspection/surveillance [2]:

• Baseline data identify specific safety and structural functions, type and property of materials used, and any assumed operating conditions. This feeds into a preliminary assessment of potential degradation and locations of degradation. Design documentation may also include details of provisions made for ensuring long term RCS integrity (e.g. dealing with creep effects) or of design limits (e.g. minimum pre-stressing loads and maximum crack widths). Laboratory study results may also be relevant for design review purposes. These may range from material tests that support concrete mix design, to large scale model tests validating design methods and assumptions.

• Construction data enable the review of concrete material quality and workmanship. Experience has shown that the most frequent cause of failure is poor quality construction, or design errors including improper material selection, with symptoms often evident at the earliest stages of structure life. It is important to identify locations where there was non-compliance with design or construction criteria, and where repairs or modifications were necessary.

• Operational experience data provide information on concrete loads and actual environmental and operating conditions that need to be compared to the original design basis to check for non-compliance. Operational experience data are particularly valuable for detailed assessments of potential future impacts of degradation mechanisms.

• Inspection and monitoring data provide historical information on actual structural and environmental conditions and these data are a baseline against which ongoing performance can be evaluated. This is valuable in tracking degradation progress (trending), and in determining whether design assumptions for environmental conditions are being achieved in practice. Data need to be reviewed to confirm that any changes in structure condition are stable and predictable, and to monitor effectiveness of mitigation measures.

At the pre-operational stage of a new NPP, the objective of an AMP for RCSs is to ensure the design and construction of durable RCSs with required instrumentation for acquiring baseline data and monitoring structural performance during operation, as well as to provide adequate guidelines for inspection, maintenance, and repair/rehabilitation of the SSCs.

An example of a comprehensive and standardized document, called a Birth Certificate [6], can provide most baseline data. It contains details on parameters important to the durability and service life of the structure concerned, and clearly defines the baseline condition, allowing for prediction of its specified design and/or remaining service life. Table 2 illustrates the content of such a birth certificate.

Table 2. Birth certificate document contents [6].

N.	Content
1	Structure identification
2	Durability specifications & standards
3	Environmental exposure conditions
4	Deterioration mechanisms & models
5	Construction materials
6	Durability test methods
7	Structural component durability data
8	In-service conservation plan
9	Dismantling plan

2.3.5 Development of AMPs

An AMP for RCSs in NPPs is a set of integrated engineering, operation, and maintenance actions to control ageing degradation of RCS within acceptable limits. It documents relevant programmes and activities, and the guidelines for coordinating such programmes and activities affecting RCS conditions and the exposure environment.

The programme generally includes the following types of activities to ensure structural integrity and the capability to satisfy design requirements through timely detection and mitigation of ageing effects until the end of lifetime [12]:

- Prevention activities, which preclude the ageing effect from occurring
- Mitigation activities, which attempt to slow the ageing effects
- Inspection and monitoring activities, including inspection and testing for the presence and extent of ageing effects, or tracking evolution of ageing effects
- Performance monitoring activities, which test the capability of a structure or component to perform its intended function(s)
- Validation of durability models based on the results obtained from monitoring or inspections.

The adequacy of an AMP in managing certain ageing effects for particular SSCs is based on a review of nine attributes credited for embodying an effective AMP in Table 3.

Table 3. The Nine attributes of an effective AMP - SSG-48 [12].

N.	Attribute	Content
1	Scope of the ageing management programme	- Concrete structures subject to ageing management - Understanding of ageing phenomena: • Structure materials, service condition, stressors, degradation sites, degradation mechanisms and ageing effects • Condition indicators and acceptance criteria • Quantitative or qualitative predictive models of relevant ageing phenomena
2	Preventive actions to minimize and control ageing effects	- Specification of preventive actions - Determination of service conditions (i.e. environmental conditions and operating conditions) to be maintained - Determination of operating practices aimed at precluding potential degradation of the structure
3	Detection of ageing effects	- Specification of parameters to be monitored or inspected - Specification of the type(s) of inspection method(s) (visual inspection, non-destructive testing, invasive testing, analytical methods) for detecting ageing effects before failure of structure - Determination of desired accuracy, and quantity and location of inspections and tests
4	Monitoring and trending of ageing effects	- Specification of condition indicators and parameters monitored - Data collected to facilitate evaluation of structure ageing - Assessment methods (including data analysis and trending)
5	Evaluation of ageing effects	- Specification of acceptance criteria against which the need for further monitoring or corrective actions is evaluated - Evaluation of current condition and estimation of future performance
6	Mitigation of ageing effects	- Actions (e.g. operations, maintenance) to mitigate degradation of the structure or component
7	Corrective actions	- Corrective actions (e.g. repair and replacement) if a structure or component fails to meet the acceptance criteria
8	Feedback of OpEx and R&D results	- Mechanism that ensures timely feedback of OpEx and R&D results - Provides objective evidence that they are taken into account in the AMP
9	Quality management	- Administrative controls to guarantee the implementation of the ageing management programme - Indicators to facilitate evaluation and improvement of the ageing management programme - Confirmation (verification) process for ensuring that preventive actions are adequate and appropriate and that all corrective actions have been completed and are effective - Knowledge management

Such a review should be integrated with the overall management system for the NPP. As an indispensable yet independent part of the general AMP for RCS SSCs in a NPP, an AMP for concrete structures should clearly identify the effective actions and measures for managing ageing in a timely manner. It should provide performance indicators that measure the effectiveness of the current practices, based on the results of ageing and condition assessments of the concrete structures [17]. Examples of such performance indicators include [12]:

- Material condition with respect to acceptance criteria (see section 2.3.6)
- Trends of data relating to failure and degradation
- Percentage of recurrent ageing-driven failures and instances of degradation

• Status of compliance with inspection programmes

• Newly discovered ageing effects and degradation mechanisms

An important constituent of an AMP is the schedule of periodic inspection and monitoring activities for concrete structures to promptly detect age-related degradation and assess the structure's performance. Periodic inspection and evaluation can be scheduled by prioritizing the structures in terms of economic aspects, safety significance, environmental exposure conditions, and anticipated tolerance to degradation. Among these factors, exposure conditions are related to the aggressiveness of the environment to which the structural component is exposed, and the susceptibility of the concrete structures thereof to ageing. One example for prioritizing safety-related SSCs would be evaluating their susceptibility to each age-related degradation mechanism as high, medium, and low based on the following criteria [2]:

• High: The degradation mechanism is most likely to occur or has occurred, either in this structure or in a similar structure under similar conditions. Steps will, or have not yet been taken, or it is unclear if any such steps are adequate to mitigate degradation for the target life or to prevent forced outages.

• Medium: The degradation mechanism is known for this structure or component, either at this facility or a similar one, and is being managed or mitigated accordingly

• Low: The degradation mechanism is possible for the structure but is easily managed with current ageing management programmes or has no impact on achieving the structure's/component's safety function.

Based on the priority listing of the safety-related RCSs, those most critical to the structural integrity and safety of the plant and those most likely to have experienced degradation should be given the highest priority. The inspection and evaluation procedures are applied to each selected structure, following prioritization and determination of an inspection schedule.

If a decision is taken to pursue LTO, justification of the adequacy of ageing management for the planned period of long term operation should be provided, based on the results of the periodic safety reviews or the results of an adequate evaluation process (that includes in its scope ageing management review and revalidation of time-limited ageing analysis (TLAA), as described in [18]), and this justification should be evaluated for adequacy by the regulatory body.

2.3.6 Condition assessment

Systematic and rigorous condition assessment plays an important role in ageing management. It helps to assess the effect of age-related degradation on structures, to establish structures' current condition, and to provide indications of structures' future performance. Generally, condition assessment needs to be performed in the following circumstances:

• prior to setting up an AMP

• in preparation for life extensions beyond the assumed service life of the plant

• periodically as a part of the integrated AMP framework

The primary objective of condition assessment prior to setting up an AMP is to establish structure baseline conditions, against which results of periodic inspections, testing, and monitoring will be compared. Typically, condition assessment involves a thorough evaluation of the collated baseline data (in section Collation of background data) at a component level via comparison of condition indicators (e.g. cracking, spalling, delamination and corrosion products, leakage rate, depth of concrete carbonation, ingress of chlorides, pre-stressing force, structural deformation, etc.) against design requirements and other predefined acceptance criteria. Condition assessment results should identify the potential ageing mechanisms that may cause degradation, the effect of age-related degradation on structures, the critical structural components, and the areas that are most susceptible to degradation, etc.

Acceptance criteria indicate whether a specific performance limit-state related to a specific deterioration mechanism or an anomaly for RCS has been reached. For example, the acceptance criteria for carbonation of concrete could be defined as the depth of the carbonation front having reached the level of reinforcement. In this example, this represents a durability/serviceability limit state.

Appropriate acceptance criteria should be based on the design basis or on the technical requirements for the structure or component, and the relevant regulatory requirements, codes and standards. Although some acceptance criteria may be found in design codes (criteria used in the mechanical and durability models), these criteria tend to be limited and wide-ranging. Determination of common acceptance criteria is difficult because of differing materials, functional requirements, behaviour characteristics, exposure conditions and other conditions. The need for sufficient margins should be taken into account in these acceptance criteria. A preliminary study into the acceptance criteria that could trigger maintenance actions of reinforced concrete infrastructure of nuclear facilities is covered in [11].

Based on the condition assessment results, the AMP can be developed, updated, optimized, and/or adjusted as appropriate. Condition assessment results enable the schedule of ageing management activities to be optimized to concentrate efforts on critical areas and guide the choice of inspection and monitoring techniques that are appropriate for the suspected degradation. Condition assessment results can also be used to identify changes necessary to address issues related to ageing effects. Recommendations are provided to ensure structural integrity and the capability to satisfy design requirements until the end of lifetime. This includes any planned life extensions and requirements during decommissioning.

2.3.7 Implementation and improvement of AMPs

An AMP is not a static programme. Once the AMP is developed, it should be periodically reviewed as described in Fig. 5 of [12] (see Appendix in section 7.1.4), where the continuous improvement of the AMP provide adequate assurances of the effectiveness of ageing management. Condition assessment plays an essential role in this process, and assessment results enable the current AMP to be updated and adjusted as appropriate. The effectiveness of current practices (e.g. inspecting, monitoring, and maintenance activities) should be confirmed through applicable ageing evaluations, condition assessments, and/or improvements to current practices as needed. For example, depending on structural performance, this may lead to either increased or reduced inspection frequency.

Periodic reporting on the performance indicators for evaluation of the effectiveness of the ageing management programme is an essential part of implementation of an AMP. AMPs can be improved and optimized on a regular basis with the assistance of feedback from OpEx, continuous improvements in the understanding and predictability of ageing mechanisms, developments in current knowledge and technology relating to both concrete durability and assessment techniques, improvements in safety thinking, and results from self-assessments and peer reviews.

Consideration should be given to arranging for peer reviews of ageing management programmes, such as SALTO peer review, to obtain an independent assessment. The review can be used to verify that existing and proposed programmes are in compliance with the nine attributes in Table 3 to verify/review specific samples of existing and newly created AMPs for consistency with IGALL AMPs, to confirm that operation, inspection/monitoring, and maintenance programmes are well-coordinated, and to identify areas for improvement

2.3.8 Operational Experience

NPP RCSs may be subjected to stressors and lifetime expectations that are not typical to the structures from which the majority of the available information on concrete ageing and deterioration has been gathered [8]. The understanding of concrete ageing for NPP RCSs needs to be improved on a continuous basis through access to R&D results related to concrete ageing in relevant environments, and evaluation of OpEx from a variety of sources. There is a need for routine re-evaluation of OpEx and R&D results related to concrete ageing, and adjustment of AMPs accordingly.

OpEx enables further advancements in understanding of the ageing phenomena. This includes understanding of degradation mechanisms and their consequences, locations and areas that are most affected, and appropriate methods and strategies for inspection, testing, monitoring, and mitigation. Adverse OpEx, which identifies ageing-related degradation, can be used to identify new ageing phenomena that require ageing management during LTO, either from the perspective of new ageing mechanisms or new locations of known mechanisms. Significant ageing events related to concrete structures in NPPs that have occurred since 1998 have been evaluated, and results are compiled in [8], which also details the mitigation measures implemented.

The International Generic Ageing Lessons Learned (IGALL) database is an international platform for discussion between regulators and utilities regarding implementation of acceptable NPP AMPs. The objective of the IAEA IGALL programme is to establish state-of-the-art reports on accepted practices for AMPs and time-limited ageing analysis (TLAAs). These serve as the basis for implementation of recommended AMPs and TLAAs for NPPs with diverse NPP technologies (PWR, BWR, VVER, CANDU, PHWR). IGALL reports are updated periodically at least every 5 years. For concrete and related civil engineering structures, IGALL [19] produces many AMPs that can be used as the basis for the development of an NPP-specific AMP.

TLAA is an important step in ageing management that is used to demonstrate that the analysed ageing effects will not adversely affect the capability of an SSC to perform its intended function(s) throughout an assumed period of operation. TLAA involves two types of parameters. The first is the time dependent variable used in the analysis (e.g. thermal cycles, moisture content, cyclic loadings that an SSC undergoes), the second is the ageing effect associated with the first parameter. Both should be evaluated and compared with a regulatory limit or acceptance criteria to determine the acceptability of the SSC for continued service [12]. Whether the existing TLAA is properly performed in the current safety analyses report or other licensing basis documents can be determined from requirements for revalidation of TLAAs as relevant to LTO found in [17].

IAEA Safety Aspects of Long-Term Operation (SALTO) peer review is a useful tool for Member States to exchange experiences, learn from each other, and apply good practices in the LTO of NPPs. The SALTO peer review is a comprehensive safety review directly addressing strategy and key elements for the safe LTO of NPPs. It is designed to assist operating organizations in adopting a proper approach to LTO including implementing appropriate activities to ensure that plant safety will be maintained during the LTO period. LTO peer review guidelines in [17] provide useful information to the operating organizations of NPPs (or technical support organizations) for carrying out their own self-assessments or comprehensive programme reviews, including guidelines for ageing management reviews and revalidation of TLAAs for civil structures in NPPs. A Member State can request that the IAEA provide a SALTO peer review service. The SALTO peer review can be carried out at any time during the lifetime of an NPP. However, the most suitable time is within the last ten years of the originally assumed lifetime of the plant

2.3.9 Ageing management review

Ageing management review is a process to identify relevant ageing effects and degradation mechanisms for SSCs, and provides a basis for effective ageing management implementation over the intended period of operation [19]. Ageing management review is performed to ensure that ageing will be effectively managed through systematically assessing the related degradation mechanisms that have been experienced or are anticipated, and the impact of their ageing effect on the SSCs' capability to perform their intended functions, taking into consideration the SSC current condition [12]. An in-depth review of ageing management needs to be performed periodically, for example as part of a periodic safety review [20] or as part of the safety review for long term operation [12].

The flowchart of ageing management review is illustrated in Fig. 4 of [1] (see Appendix section 7.1.5). It involves a general review of historical data and analysing the collated data for each selected structure or component or commodity group.

A process to identify relevant ageing effects and degradation mechanisms for each structure or component should be established, and the programmes to manage the identified ageing effects and degradation mechanisms should be in place [12]. This process should cover the following steps [12]:

- TLAAs associated with these SSCs should be evaluated to determine the continued validity of the analyses for the intended period of operation. Results of the evaluation of the TLAAs should be taken into account in the ageing management review.
- All relevant ageing effects and degradation mechanisms should be identified.
- If the ageing of SSCs is managed by existing ageing management programmes, it should be verified that the AMP is consistent with the nine attributes shown in Table 3.
- If the ageing of SSCs is managed by other NPP programmes, e.g., regarding maintenance, it should be verified that these programmes are consistent with the nine attributes shown in Table 3.
- If the ageing of SSCs is not managed by any existing programme, a new programme should be established or existing programmes should be modified or improved (e.g., by extending the scope of an ageing management programme) or a specific action (e.g., a new TLAA, replacement of the SSC, or further analysis) should be taken.
- If the qualified lifetime of equipment (e.g. structural monitoring) important to safety expires, such equipment should be requalified or replaced at the expiration of its present qualification.

Ageing management review involves, but is not limited to, the following essential elements [12, 19]:

- Assessment of the current condition of the RCS and providing predictions of future performance:
- Identification of those potential ageing mechanisms that may cause degradation and ageing effects on RCSs on the basis of fundamental knowledge for understanding of ageing.
- Identification of the appropriate programme for ageing management (existing ageing management programmes, possibly with improvements or modifications, or newly developed programmes).
- Determination of mitigation actions to slow the degradation rate or appropriate corrective actions to recover RCS capability to fulfil its designed function.
- Reporting of the ageing management review to demonstrate that the ageing effects and degradation mechanisms are being managed effectively.

In the event of some major intervention in operations of the NPP, such as reactor power update or important modifications or equipment replacement, if undetected ageing mechanisms are discovered, an appropriate ageing management review should be performed to acquire information and knowledge about the current status of SSCs. In addition to current condition, necessary changes, issues related to ageing effects, and economic improvement opportunities need to be identified in ageing management review.

Furthermore, relevant applicable lessons learnt (IAEA IGALL Programme) relating to ageing provide a good reference basis for the ageing management review [1]. These should not, however, be used in place of an NPP-specific ageing management review [12].

2.4 Through-life management of concrete structures

The ageing of safety-related RCSs should be managed with foresight throughout the entire lifetime of the plant. Through-life management of a RCS includes planning and monitoring activities during the following phases in the life of the structure [6]: design, construction, (long-term) operation, and demolition

2.4.1 Design

At the plant design stage and for licensing review, it should be demonstrated that ageing has been adequately taken into account, and proper measures taking into account the latest knowledge of ageing effects and degradation mechanisms, the latest technology in concrete and reinforcing steel have been introduced in the RCS design, to mitigate or eliminate the ageing causes.

NPPs RCS are designed, constructed, operated, and inspected in accordance with national consensus codes and standards. Normally these standards and codes have been carefully reviewed from those that exist for civil structures to ensure their applicability to NPP environments. Experiences indicate that some deterioration of concrete structures can be attributed to original design errors, primarily those related to inadequate structural design and lack of attention to detail, and inadequate concrete quality or type. Prudent engineering practices during concrete mixture design and specification, structural design, and construction are necessary to minimize the potential for degradation during service.

As RCS are structurally designed based on established codes and standards including applicable loadings and load combinations, the common causes of deterioration are typically due to environmental/ageing conditions other than mechanical loading. The main deterioration mechanisms of RCS are divided into those affecting the concrete and those affecting the reinforcing steel materials, including both steel reinforcement and post-tensioning system.

When designing a RCS, to ensure the fulfilment of the design service life, accurate definition of the working environmental/loading condition is required to define the appropriate concrete performance parameters necessary for design (structural and durability). Care must be taken so that ageing issues of each concrete structure/component or commodity group have been adequately addressed in the design specifications. These condition the choice of the characteristics of the mix design and basic concrete properties. For a specific RCS, it should design conservatively for the scenario with the most severe condition combinations. In the case that some environmental/loading conditions are not explicitly defined in codes and standards, appropriate margins need to be provided in the design to take into account relevant ageing mechanisms and potential age related degradation, to ensure its capability to perform the necessary safety function throughout its design life.

Designers need to consider that RCS may need to function longer than the planned service life. Materials need to be properly selected considering anticipated exposure conditions as well as longevity. Materials specified for use in NPP RCS that are designed to be excluded from replacement as part of normal maintenance programmes, need to be proven by long term satisfactory field performance or by tests. It is important to ensure that tests are representative of actual field applications and ambient conditions.

Instrumentation for monitoring RCS behaviour or environmental conditions needs to be considered during design of the structure to allow for its installation and maintenance. It is useful to install instrumentation during construction to provide information regarding initial conditions which can be used to verify design assumptions and to establish baseline data for the subsequent measurements.

The design phase plays a critical role in knowledge management in both the quantity and the importance of the information that is produced. During the design phase, the PIM provides the information infrastructure and framework to support obtaining and organizing necessary information. Since the PIM is a set of knowledge and data about the as-designed objects stored in electronic format by established rules, it contains the initial baseline information for engineering calculations, as well as supporting mathematical, computational, and simulation models. The design information contains both the tacit and explicit justification of the satisfaction of a variety of requirements coming from several different stakeholders, such as the reactor vendor, EPC companies, suppliers, and regulators. These requirements cover several domains that feed into specific design, procurement, construction, and project management processes such as: licence requirements; environmental requirements; operation and maintenance requirements (e.g. accessibility to SSCs); and, cost and planning [16].

2.4.2 Construction

Experience has shown that over 50% of ageing problems in RCS are associated with construction defects. Poor workmanship can lead to voids due to inadequate compaction, plastic shrinkage cracks due to uncontrolled curing and inaccurate positioning of rebar and joints tend to facilitate ageing related degradation. This signifies that quality control and quality assurance programmes at NPPs are necessary to ensure that the quality of concrete SSCs is in compliance with the design. Based on the guidelines in IAEA specific safety guide SSG-48 [12] and Safety Guide NS-G-2.12 [1], the following needs to take place during the construction stage for concrete structures [2]:

- Known factors affecting ageing management of RCS provided to manufacturers and constructors
- Current knowledge about relevant ageing related degradation mechanisms, their effects, degradation and possible mitigation measures addressed during construction
- A complete data collection and documentation of baseline data as defined in birth certificate [6]
- Manufacture of control specimens if necessary and surveillance specimens for specific ageing monitoring programmes made available and installed (e.g. construction samples where applicable).

Careful material selection and material quality is important for long term structure durability. The properties used during the design of the project, as well as those achieved during construction testing are to be recorded on the birth certificate. For concrete, each of the actual mix designs used in the structures should be identified. This includes the proportions of cement (brand and type and origin), water, coarse and fine aggregates, supplementary cementing materials (additions) such as silica fume, fly ash or blast furnace slag, and all other admixtures. A sample of concrete mix design can be found in [6]. A summary table containing durability test results for all concrete mixes is also provided in [6], which includes concrete used for pre-construction trial batches during design, as well as actual test monitored during construction. Processes need to be implemented to monitor the quality of fresh and hardened concrete, concrete placement, vibration processes, curing and thermal monitoring during setting and hardening.

Various structural tests (certified material test, performance test for prestressing tendons, liner acceptance test, jacking data for prestressing tendons, etc.), and pre-operational tests (structural integrity test, leakage test, Polar crane test, etc.), can help control the quality of concrete material and workmanship.

Besides the methods mentioned above for controlling structural strength of the material, there are a number of standardized tests that have been developed to quantify the durability performance. Several of the more common standardized test specifications are given in Table 2-7 and Table 2-8 in [6] (see Appendix section 7.1.6 and 7.1.7). Durability tests should be specified based on the type of deterioration mechanism anticipated for the structure and consistent with the design methodology used. The specification of the durability test and the frequency of the test need to be clearly identified, as well as the numerical models used for durability assessment.

Manufacturers and constructors should adequately address factors affecting ageing management and sufficient information on construction and material are provided to the operating organization, so that the authorized professional staff responsible for ageing management in RCS can take this information into account in developing ageing management programmes, including maintenance procedures.

After the structure is placed in-service, periodic monitoring of the deterioration process is essential to evaluating the performance over time and compare to the results of modelling which can be updated. Technical specifications need to include requirements for durability parameters that are measurable within a reasonable time frame (a few months) after construction. Depending on the exposure environment, type of deterioration mechanism, structure geometry, and loading, some parameters such as, for example, heat of hydration development, permeability of concrete cover and crack size may be specified to ensure long term durability of the structure. All the monitoring data and test results should be documented in the birth certificate to aid ageing management activities, enable condition assessments, and facilitate modifications and repairs.

2.4.3 (Long-term) Operation

The programmes and documentation relevant to ageing management need to be implemented effectively during the operation stage. The attributes of an effective ageing management programme, listed in Table 3, need to be continuously updated and improved following the generic method for ageing management in section 2.2.

2.4.3.1 Preventive actions

The primary strategy for effective ageing management is proactive ageing management, taking ageing into account with foresight and anticipation throughout the plant’s lifetime, and taking preventive actions to mitigate ageing effects. With sufficient understanding and predictability of ageing mechanisms and ageing effects, preventive actions aiming to control ageing related stressors and delay concrete ageing could be implemented as early as in the design and construction stages. The plant operator and plant supplier should specify requirements relating to ageing management in their bidding and tendering documents, and finally in their procurement contracts. Examples of typical preventive actions include

• Control of material quality and workmanship during construction of the concrete structures

• Monitoring and controlling service conditions (i.e. environmental and operating conditions) to ensure utilizing concrete structures in accordance with design conditions

• Improved technology and material (e.g. weld method, coating or other surface treatment, etc.) to prevent fluid penetration

• Routinely performed preventive maintenance of concrete structures during operation stage

Preventive maintenance may be seen as proactive intervention where some form of activity is applied prior to damage becoming visible. For safety-related concrete structures, some proactive actions at locations where ageing is likely to occur include inhibitors (cathodic protection), chloride extraction, re-alkalisation, electrochemical dehumidification, surface coating, and overlays. Possible time-based preventive actions include periodic replacement of sealants, gaskets used for joints, structural holes, and gaps irrespective of condition. More preventive maintenance activities are described in section 8.2 of [2].

2.4.3.2 Inspection and monitoring of degradation

Almost from the time of construction, RCS start to degrade under service and environmental conditions. It is very important to detect ageing degradation as early as possible, to address associated reductions in safety margins to take corrective actions before loss of integrity or functional capability occurs. The operating organization should ensure the timely detection and characterization of significant ageing effects through the inspection and monitoring of concrete structures, and the assessment of observed ageing effects to determine the type and timing of any actions required [12].

Through proper inspection techniques, the most likely locations for degradation and its causes within the concrete structures can be identified. A thorough survey of these critical locations will provide data to describe the current physical condition of the concrete structures, evaluate past structural performance, and form a basis for comparison during future inspections and decisions regarding the type and timing of actions to correct detected ageing effects.

Sound inspection and evaluation programs, in which the performance and condition of concrete structures are periodically evaluated, can be used to ensure that the structures continue to serve their intended function. A general methodology for developing a programme of inspection and evaluation that recommends the effective practices for inspection and evaluation of safety-related concrete structures is provided in [15]. It is an essential concept that the long-term soundness of concrete structures is ensured by both confirming their current soundness and predicting their future performance. Even if the structures are check by visual inspection and/or non-destructive evaluation, it just provides a current situation and will not ensure the future performance. Therefore, both the evaluations of current soundness of the structures and predicting their future performance are indispensable.

Because of the many similarities between nuclear and non-nuclear concrete structures, practices and procedures used for their inspection and maintenance are also similar. A vast variety of in-service inspection techniques is available to indicate the occurrence and extent of age or environmental stressor related deterioration. Details about various inspection methods can be found in [2, 10]. Inspection methods should be previously validated by means of tests on NPP RCS representative mock-ups, as their application strongly depends on the specific characteristics of the structure.

The selection and use of inspection and testing methods should be well established in the AMP and carefully planned and implemented. The document should contain the specification of periodic inspections for concrete structures, the methods of inspection, the criteria of evaluating the results of inspection, and knowledge management of the outcomes of inspection. Determination of the type(s) of test method(s), desired accuracy, and quantity and location of inspections and tests should also occur on each structure basis. In general [2, 15],

• The inspection methods used are able to inspect and assess reinforced concrete structure performance, which is undertaken within the in-service inspection (ISI) programs.

• It is important to know the accuracy, sensitivity, reliability and adequacy of the examination methods used to identify and evaluate the particular type of suspected degradation. The performance of examination methods must be evaluated in order to rely on their results.

• Inspection and evaluation frequencies are based on the aggressiveness of environmental conditions and physical conditions of the concrete structures or determined through the use of a time-dependent reliability study. The frequencies are also subject to change depending upon the likely rate of degradation and its potential consequence.

• Selected locations may include known defects which are monitored to check the evolution, or critical locations identified during the condition assessment.

In-service inspections for concrete structures in NPPs are divided into visual on-site inspections and monitoring. Continuous monitoring and measurements are addressed to structures which are especially important for the performance of the unit [14]. A single inspection gives a snapshot of structure condition at a given point in time, while estimates of future performance are generally based on monitoring data and extrapolation of results from earlier surveys. Comparisons with historical data are needed to calculate deterioration rates (to establish trends) and provide input parameters for models used to estimate the service life of the structure.

Inspection techniques for concrete structures in NPPs can be grouped in the following four categories and a brief description for each category (including uses and limitations) is available in section 3.5 of [15].

• Visual inspection: As the various concrete degradation mechanisms often produce visible indications, patterns, or features on exposed surfaces during initial manifestation and propagation, the general condition of concrete is often assessed using visual inspection by knowledgeable and experienced personnel. Visual inspection of accessible surfaces of the concrete structures are expected to detect and define areas of ageing-related distress that result in visible effects on the surface of the structures, e.g. cracking, spalling, volume change, cement-aggregate separation, mechanical degradation, or moisture movement. Visual inspection can be supplemented by other non-destructive testing methods such as rebound hammer, radar (GPR), audio (passive acoustic monitoring), infrared thermography, ultrasonic pulse velocity, tomography, electrochemical measurements, leakage rate test, and instrumentation, etc. [17]. A well conducted visual inspection constitutes a cost-effective method of assessing ageing symptoms, particularly for concrete structures having large exposed surface areas, such as the containment building.

• Non-destructive testing (NDT): Internally initiated degradation and inaccessible structures requires the use of in-place in-depth examination techniques, such as non-contact NDT, embedded wireless sensors and NDT techniques with higher penetration. NDT techniques are continually evolving and research continues to enhance existing methods as well as develop new methods. [15] provides an overview of techniques commonly used to concrete as well as several that are under development.

• Invasive testing: Invasive or partially invasive testing can be used to determine concrete strength, density and quality on samples removed from the structure, to locate voids or cracks in concrete, to confirm depth of concrete cover and to visual detect steel reinforcing material corrosion. Invasive testing also includes the testing of soil, groundwater, and other material samples that represent the environment to which a structure is subjected.

• Analytical methods: Such methods use supplemental calculations or analysis techniques to evaluate the structural behaviour and resistance of the structure. This kind of methods may be required in the event that any potentially significant degradation is found during the inspection and testing phase.

For structural parts, where detection of degradation would be difficult, or where repair of any degradation would be costly, it is appropriate to monitor and, if necessary, control the environment or potential stressors that could lead to degradation. Through monitoring against the baseline condition, tracing inspection data (routine inspection, monitoring results, and historical documents) and assessing collated survey data, it allows timely evaluation of age-related deterioration to confirm structural fitness for service or in case of deviations from the expected condition to determine corrective actions required based on the severity of the deviation. The monitoring and measurements programme contains the following [14]

• Deformation measurements

• Temperature measurements

• Surveillance of crack growth

• Surveillance of possible leakages

• Tightness of containment

• Surveillance of properties of concrete, reinforcing steel (corrosion), prestressing steel and expansion joint materials

• Monitoring and measurements in seawater channels

• Monitoring of groundwater parameters

• Surveillance of coatings

• Use of control or reference specimens

Instrumentation to facilitate ongoing concrete monitoring needs to be incorporated as part of structure design, and is typically embedded into the concrete at time of construction and includes such as vibrating wire strain gauges, thermocouples, pendulums, extensometers, liquid level gauges, humidity gauges, dynamometers, and benchmarks. In structural health monitoring different NDT methods are used for continuously monitoring the status of structures. In NPP containments where grouted tendon systems have been utilized, the instrumentation also provides important data for assessing the condition of the post-tensioning system. Additional information on performance of the grouted tendon systems is provided in some cases by leaving a limited number of tendons un-grouted and installing load cells to monitor the pre-stress level [21].

2.4.3.3 Evaluation of concrete structures

Degradation symptoms require a tiered approach which uses evaluation findings to trigger levels of response [2]. As an example, the following presents a three-tiered method defined in [15], where the detailed criteria defined for each tier are listed in section 5 of [15].

• Acceptance without further evaluation: The first-tier criteria represent acceptable conditions that are considered acceptable without requiring any further evaluation.

• Acceptance after review: In the event that the first-tier criteria are exceeded or the observed conditions need further evaluation, the second-tier criteria represent acceptable conditions for observed degradation that have been determined to be inactive. Inactive degradation can be determined by the quantitative comparison of current observed conditions with that of prior inspections. If there is a high potential for progressive degradation or propagation to occur at its present or accelerated rate, the disposition should consider enhanced surveillance of the specific structure or initial repair planning. Enhanced surveillance may also involve routine use of non-destructive techniques to provide quantitative data for trending behaviour of critical areas.

• Additional evaluation required: When the second-tier criteria are exceeded or conditions found to be detrimental to the structural or functional integrity, condition should be considered unacceptable and in need of further technical evaluation. At this stage of the evaluation process, re-analysis of structural capacity and behaviour under degraded physical conditions is often necessary. Should it be determined that the original design requirements and licensing commitments can no longer be achieved, repair or replacement options should be examined for the affected structure.

The quality and value of results obtained from an evaluation of an existing concrete structure depend, to a great extent, upon the qualifications and capabilities of the evaluation team. The personnel involved need to be knowledgeable and experienced in evaluating concrete structures (material degradation and structural integrity) and be familiar with design and functional requirements of nuclear concrete structures.

If degradation is present, an understanding of the processes involved is fundamental because degradation rates vary according to the mechanisms involved. If the process is not adequately understood, it may be necessary to commission material test programmes (e.g. mock-ups or accelerated ageing studies) and structural simulations.

2.4.3.4 Mitigation and corrective actions

In reference [12], it clearly defines that ageing management should ensure the timely detection and characterization of significant ageing effects through the inspection and monitoring of structures or components, and the assessment of observed ageing effects to determine the type and timing of any actions required. Associated actions include identification and evaluation of techniques for mitigation of any environmental stressors or ageing factors that act on concrete structures, and assessment of techniques for repair, replacement, or retrofitting of concrete components that have experienced an unacceptable degree of deterioration [22].

The action programme should document occurrences of identified ageing related degradation and the methods used to address the degradation. Corrective actions include repair of damaged concrete and mitigation of deterioration causes. Typical actions considered in response to detected concrete degradation include:

• Enhanced surveillance to trend progress of deterioration. This is the initial approach during early stages of degradation.

• Maintenance/operational changes to prevent deterioration from getting worse (if safety margins are acceptable).

• Local repairs or replacement to restore parts of a structure to a satisfactory condition or mitigate further degradation (e.g. waterproofing and coating, among other things).

All efforts should consider the degradation mechanisms and eliminate (if possible) and mitigate degradation causes as much as possible, to avoid future degradation from the same phenomenon. Table 8 in [2] summarizes available mitigation methods for various ageing degradation mechanisms.

Chapter 8 in [15] and Fig. 156 in [2] (see Appendix section 7.1.8) provide guidelines for repair in case that the structures identified in the evaluation procedure be restored to their desired strength. The responsible-in-charge engineer should ensure that the necessary research to assess available options, validate a specific repair and demonstrate feasibility on representative mock-ups is performed. Following the selection of a repair material and completion of necessary supportive documents and calculations, the responsible-in-charge engineer for repair should prepare a summary report identifying the source or root cause of degradation and reason for the evaluation, the decision-making for the repair method selection, repair material qualification data, summary of structural calculations, repair drawings, and all documents related to the evaluation. During the repair process, it is necessary to provide quality control in the form of inspections—possibly including NDT—or other reviews to verify the adequacy of the repair process and materials. During and following completion of the repair, the evaluation process should include several checkpoints to validate the material condition and structural performance of the repaired structure. After a repair action, follow-up inspections need to be conducted periodically to confirm repair effectiveness.

If it is determined, through the evaluation of the mitigatory and corrective action programme, that the ageing management programmes do not adequately manage the effects of ageing, modifications to the existing ageing management programmes should be specified and implemented or new ageing management programmes should be developed.

2.4.4 Decommissioning/Demolition

Decommissioning/demolition actions include the decontamination, dismantling and removal of SSCs, including management of the resulting radioactive waste and radiation protection of workers carrying out the decommissioning, as well as the conduct of characterization surveys to support decommissioning [23]. Decommissioning is nowadays being recognised as an important aspect of plant management, deserving consideration from the outset of design and planning activities. Design provisions specific to decommissioning include designing structures for long-term stability minimising infiltration, containing spills and releases, and retarding contaminant transport [24].

Appropriate ageing management arrangements should be put in place to ensure that required concrete structures remain available and functional until the completion of decommissioning, e.g. ensuring the long term integrity of SSCs to prevent their deterioration and to allow the safe dismantling, handling and transport of components; monitoring SSCs to ensure the integrity of the containment and subsurface infrastructure components such that there is no significant release of radioactive particles that may concern public health and environment [12].

It is essential that information management system introduced in section 2.3.2 incorporates requirements for records management, including:

- material specifications and records from the siting, design, construction, operation and shut down.
- The physical configuration of the plant on an ongoing basis,
- Leaks and other contamination incidents. Plant operators need to give particular attention to recording this information – as such contamination may otherwise only be identified during demolition of the concrete structure.

The decommissioning of a NPP involves a wide range of actions that are carried out in the presence of a variety of radiological and non-radiological hazards and associated risks. The level of detail of planning necessary for meeting the decommissioning requirements differs depending on the type and complexity of the facility, its radioactive inventory and the potential hazards expected during decommissioning. Despite this, the general approach to decommissioning remains the same, and is adapted to the specific facility's situation by the application of a graded approach, which can affect the selection of the decommissioning strategy, the planning details, the conduct of decommissioning actions and the end state chosen. Guidance can be found in the IAEA Specific Safety Guide No. SSG-47 *Decommissioning of Nuclear Power Plants, Research Reactors and Other Nuclear Fuel Cycle Facilities [23]*.

3. Environmental loading of NPP RCS

3.1 RCS and their surrounding environmental conditions

Even though some NPPs might have similar designs, their geographic locations define the variety of environmental loading conditions they are subjected too. For example, the conditions that RCS is subjected to if it is located in northern Europe by the seaside are quite different than those if it is located along a river in southern Europe. One key factor in the longevity of RCS is whether the primary containment structure is housed within the reactor building, or directly exposed to the environment. In Nordic environments, it is common for the primary containment structure is protected directly from the aggressive environment, where winter temperatures can often descend below - 20 °C. However, for many other European NPPs the containment structure is in direct contact with the environment.

When designing RCSs, to ensure the fulfilment of the design service life, accurate definitions of the working environmental/loading condition is paramount so as to define the appropriate concrete performance requirements necessary for design (both structural and durability related). These influence the choice of the characteristics of the mix design and basic concrete properties[11].

For the main safety-related concrete structures of a typical NPP, the most common internal and external loading conditions expected are [11]:

- E1: High/low temperature (environments where temperature is constantly at high /low values in relation to ambient temperature, or where high temperature gradients can occur)
- E2: High humidity (environments where humidity levels are constantly high, i.e. > 90 %)
- E3: Atmospheric conditions (environmental exposure to all elements of weather, such as rain, wind, freeze/thaw, solar radiation, etc.)
- E4: Submerged or in contact with sea/fresh water
- E5: Underground (structural elements that are in contact with the ground, typically foundation structures, tunnels, etc.)
- E6: Radiation (environments where concrete is subject to a source of radiation)
- E7: Internal building conditions (environments occurring in office type buildings), or low humidity conditions
- E8: Mechanical/Structural loading (dead weight, service load, pressure, vibration, and other types of mechanical loads that are present)

Any combination of exposure environments can result in unique loading conditions that the RCSs are expected to withstand (if designed accordingly). While in theory this seems relatively straight forward, in reality the RCSs are subjected to local climatic conditions, and microclimates, which can have synergetic effects on the deterioration process of concrete [25].

For example, a specific surface of a concrete beam or column subject to the atmosphere, can be loaded differently due to the existence of dominant winds/driving rain directions. The beams/column can become subject to differences in moisture content which can influence the rate of deterioration mechanisms or influence the ingress rate of an aggressive species. For a specific scenario, common practice is to design for the most severe conditions. This ultimately can result in un-optimised or incompatible design solutions.

Table 4. Typical exposure environments for safety-related concrete structures in LWR plants [11].

Concrete structure	Environments
Primary containment	
Containment dome/roof	E1, (E3), E7
Containment foundation/basemat	E5
Slabs and walls	E1, E7
Containment internal structures	
Slabs and walls	E1, E7
Reactor vessel support structure (or pedestal)	E1, E6, E7
Crane support structures	E1, E7
Reactor shield wall (biological)	E1, E6, E7
Ice condenser dividing wall (ice condenser plants)	E1, (E6) , E7
NSSS equipment supports/vault structures	E1, (E6), E7
Weir and vent walls	E1, E7
Pool structures (Reactor, fuel, condensation)	E1, E4, E7
Diaphragm floor	E1 , E7
Drywell/wetwell slabs and walls	E1, , E7
Secondary containment/Reactor buildings	
Slabs, columns, and walls	E3, E5
Foundation	E4, E5, E7
Fuel/Equipment storage pools	
Walls, slabs, and canals	E1, E2, E4, E5
Auxiliary building	E3, E7
Fuel storage building	E3, (E2), E6, E7
Control room (or building)	E7
Diesel generator building	E1, E3, E7
Piping or electrical cable ducts or tunnels	E1, E2, E7
Radioactive waste storage building	E2, E6, E7
Intake structures (including concrete water intake piping and canal embankments)	E1, E3, E4, E5
Pumping stations	E2, E3, E7
Plant discharge structures	E1, E3, E4, E5
Emergency cooling water structures	
Cooling water channels	E3, E4, E5
Water wells/pools	E3, E4, E5
Turbine building	E2, E3, E7

The environmental conditions expected for different SSCs are shown in Table 4. Note that, while some environmental conditions are common and sustained, others can occur periodically or as a result of unforeseen events.

Furthermore, experience has shown that during the normal construction process, the resulting concrete structure can be comprised of material with heterogeneous quality.

RCSs are constantly under mechanical loading, i.e., under gravitational loading and due to the pre-stressing of the concrete. The latter is by far greater than the dead weight of the structures. The main purpose of the RCS is to carry the dead weight of the structures and all types of service/live loads. The codes and standards provide guidance to design the structures including the loads and load combinations for normal operating, severe environment, and design basis accident (LOCA or LBB) scenarios.

Since some of the RCSs also serve as a biological shield against radiation and as protective structures against external and internal accidental loads, they are unusually heavy. The containment also has to withstand high internal pressure and temperature in case of loss-of-coolant accidents (LOCA). The accidental loads are not considered in this report, since they are not related to the normal maintenance of the NPPs.

3.2 Ageing and deterioration mechanisms of reinforced concrete

The ageing of an RCS starts immediately following construction and continues through its service life. For NPP RCSs, it is also necessary to consider the effects of ageing up to the end of any decommissioning process.

In general, NPP RCSs are subject to the same types of environmental loading as many common types of RCS infrastructure, with the exception of exposure to radiation. As a result of the safety requirements for radiation containment, NPP infrastructure has unique features and design requirements. These requirements are stricter for performance and fall outside the scope of conventional RCS design codes.

Reinforced concrete is a strong and durable building material. However, RCSs are affected by a number of factors that can cause deterioration, which can result in reductions of their performance and unsafe conditions. The common causes of deterioration are typically due to environmental conditions other than mechanical loading or are directly linked with errors in the construction phase.

Reinforced concrete deterioration typically occurs when the material is exposed to environmental atmospheric conditions, water, or chemicals over an extended period of time. In general, concrete structures can deteriorate for a variety of reasons, and often damage results due to a combination of factors. The main deterioration mechanisms of RCSs may be divided into those affecting the concrete and those affecting the reinforcing steel materials (i.e., steel reinforcement or post-tensioning system).

In many NPP types, the containment wall is post-tensioned with high-strength steel tendons This kind of structure is called a pre-stressed concrete containment vessel (PCCV). In PCCVs, the concrete is stressed (compression, tension and shear) which enhances creep. High temperatures due to the operation of the reactor further enhance creep. Near the anchors, high shear forces are also present, which demands careful design of reinforcement. Furthermore, the tendons undergo pre-stress losses in time, which decreases the compression in concrete and leads to deformations of the structure. The behaviour of PCCVs is complex, as the state of the structure is slowly but constantly changing. Together with environmental loadings such as temperature and moisture gradients through the thick walls, this can lead to different deterioration mechanisms of the concrete material.

Table 5. Causes of concrete/reinforced concrete deterioration as a function of the exposure environment [11].

Mechanism/Environments	E1	E2	E3	E4	E5	E6	E7	E8
Chemical & biological								
Aggressive water			•	•	•			
Acid attack (Boric acid *)		(•)	•	•	•		(•)†	
Leaching		(•)	•	•	•			
Alkali aggregate reaction (AAR)		•	•	•	•			
Carbonation		•	•				•†	
Chloride ingress			•	•††	•			
Sulphate and thaumasite			•	•	•			
Bacterial		(•)	•	•	•		(•)†	
Physical								
Freeze-thaw	(•)		•	(•)	(•)			
Elevated and high temperature	•					•		
Shrinkage (dry/plastic)	•					(•)	•	
Mechanical /Structural								
Abrasion/Erosion			•	•	(•)		(•)†	
Cavitation				•				
Creep and relaxation	(•)							•
Fatigue								•
Settlements and movements								•
Vibration (and seismic)								•
Thermal stresses	•					(•)		
Radiation *								
Radiolysis (in concrete)						•		
Radiation induced volumetric expansion (RIVE)						•		
Mechanical property change						•		
Corrosion (embedded metals, liners, etc.)								
Chloride ions - pitting		•	•	•††	•			
Carbonation - general			•				•†	
Other corrosion mechanisms	(•)	•	•	•	•			

** Unique to NPP concrete; (•) Possible indirect occurrence; •† Internal building conditions; •†† Does not occur in contact with fresh water.*

The deterioration of concrete can be caused by changes to the cement-paste matrix or aggregates under chemical or physical attack. Deterioration by chemical attack may occur due to: leaching, sulphate attack, acid/base attack, salt crystallization, and alkali-aggregate reactions, among others. Deterioration by physical attack may include freeze/thaw cycling, thermal exposure/thermal cycling, and abrasion due to mechanical action [10]. A quick glance at Table 5 reveals that the majority of the chemical, biological, and corrosion related problems of concrete are connected to the presence of water.

The deterioration of steel reinforcement due to electrochemical attack can occur due to the carbonation of the concrete cover layer, or due to the ingress of chlorides. Post-tensioning systems are susceptible to the same degradation mechanisms as mild steel reinforcement, and also to stress corrosion cracking or hydrogen embrittlement, as well as loss of pre-stressing force, primarily due to tendon relaxation and concrete creep and shrinkage [10].

3.3 Implications of climate change

The climate change induced by an increase in concentrations of greenhouse gases will affect the geographical and seasonal distribution of precipitation, wind conditions, cloudiness, air humidity, and solar radiation. NPP RCSs will be subject to changes of these climatic conditions, which can significantly affect rates of deterioration of concrete. For example, while corrosion of reinforcement is greatly dependent on the moisture and time of wetness of the structure, risk for freeze-thaw deterioration of concrete depends on the number of freezing-thaw cycles (when temperatures drop below freezing point and return) and a large amount of liquid precipitation before freezing.

Climate change associated with global warming will therefore influence the conservation conditions for NPP RCS. In addition to the gradual changes that occur over long periods, climate change will also mean more extreme and frequent weather events.

Climate model simulations are necessary to study the geographical and seasonal changes to weather. Modelling of the future climate is based on alternative scenarios of greenhouse gas and aerosol particle emissions using different assumptions about the future development of population growth, economic development, energy production modes, etc.

Infrastructure adaptation to climate change has been under scientific research only in very narrow areas. International research related to the effect of climate change on RCSs has concentrated mainly on building physics-related problems such as changes in energy needs in cooling and heating of buildings, and energy efficient performance of buildings. In addition, almost all of these studies have considered only new construction, not the existing infrastructure.

Hrabovszky-Horváth [26] studied a vulnerability of refurbished reinforced concrete buildings to the effects of changing climate in Hungary based on the increased number of windstorms and extreme rainfall events. The effect of climate change on chloride-induced reinforcement corrosion has also been studied extensively (e.g. [27]). However, the bulk of such studies have mainly considered new construction.

Research that studies the sufficiency of durability demands presented in concrete codes with regards to changing climate is needed.

4. Ageing/Deterioration mechanisms

4.1 Acid Attack

4.1.1 Process Definition

Chemical attack of concrete can be broken down into six categories: (1) acid attack, (2) alkali attack, (3) carbonation, (4) chloride attack, (5) leaching, and (6) sulfate attack. The first two, acid and alkali attack, constitute attack by aggressive water and are the focus of the current section. This ageing process can be further differentiated by which of the concrete's constituent materials is subject to degradation: (1) the cement paste, (2) the aggregate, or (3) the steel reinforcement. Because alkali-silica reaction (alkali attack of silicate aggregates) and reinforcement corrosion (chemical attack of the steel reinforcement) are separately classified ageing mechanisms in their own right, attack by aggressive water is defined herein as chemical attack of the cement paste matrix, within which these other materials are embedded. The focus will be further limited to acid attack, i.e., concerning the dissolution of cement paste due to changes in the speciation of water (H^+, H_2O, OH^-), as distinct from other mechanisms of chemical attack that rely on subsequent precipitation (sulphate attack), and reliant on concurrent leaching to provide driving force for ongoing acid-catalyzed dissolution.

While in a technical sense aggressive water encompasses both acidic and alkaline solutions, in practice effectively all documented instances of this ageing process concern acid attack alone. This is due to inherently alkaline cement chemistry, which already results in a pore solution of about pH 13 in most mature concretes. As such, alkaline conditions already represent the baseline from which stability of cement paste phases is assessed, with degradation during ageing largely attributable to neutral or acidic pH when concerning attack by aggressive water. This degradation manifests as a shift away from the equilibrium pH of the solid phases making up the cement paste matrix, largely constituted of hydrates and hydroxides, causing them to dissolve in the presence of excess H^+ ions. Specifically, the calcium hydroxide, calcium-silicate-hydrate (CSH), and calcium carbonate phases of the cement paste dissolve in the presence of acidic solution as shown below:

$$\begin{aligned} &Ca(OH)_2 + 2H^+ \longrightarrow Ca^{2+} + H_2O \\ &3CaO \cdot 2SiO_2 \cdot 3H_2O + 6H^+ \xrightarrow{H_2O} Ca^{2+} + 2(SiO_2 \cdot nH_2O) + 6H_2O \\ &CaCO_3 + 2H^+ \longrightarrow Ca^{2+} + CO_2 + H_2O \end{aligned} \tag{1}$$

References: More information is given in [28, 29].

4.1.2 Influential factors

Reference: A synthesis of the general influence of environmental factors is found in[29].

4.1.2.1 General conditions

Effect of moisture content and relative humidity Although acid attack depends on contact of the concrete with a reservoir of bulk water, it may take place between multiple wetting and drying cycles. While the process is ongoing, the concrete is saturated. However, capillary suction forces during drying cycles may cause intermittent mechanical degradation of the corroded layer.

Effect of temperature Due to the reliance of acid attack on the dissolution of cement paste phases, and the diffusion of acids and dissolved calcium through the corroded layer, the kinetics of both processes will be accelerated by higher temperatures.

Effect of irradiation No direct observation of the influence of irradiation on acid attack is available. Although irradiation accelerates dissolution of crystalline minerals by inducing damage to their crystal structures, this effect is minimal for calcium carbonate because its interatomic bonds are primarily ionic in character [30]. It remains ambiguous to what extent irradiation during acid attack may influence dissolution of crystalline but chemically similar cement paste phases such as calcium hydroxide, or disordered phases such as CSH. In both cases, radiolysis of water may also promote dissolution [31].

Effect of microbiological reactions The influence of microbiological reactions, mainly a concern in the case of sulphuric acid attack, is to: (1) create organic acids that further exacerbate acid attack beyond that of sulphuric acid of purely chemical origin, and (2) create biofilms on concrete surfaces that hinder transport of dissolved constituents out of the concrete [32].

Effect of exposure condition (free, sheltered, buried, submerged) Similar to the effects of moisture content or relative humidity, exposure condition may influence acid attack (beyond the obvious requirement for water contact to initiate the process) if it is such that it induces cyclic wetting and drying of the concrete. Typically, acid attack occurs in submerged concretes, i.e., those with continuous exposure to a bulk reservoir of water. This could, for example, also include any buried structures wherein the depth and environment would result in groundwater exposure.

Effect of external water composition (submerged or buried structures) By definition, acid attack is wholly dependent on external water composition, specifically its content of acids.

From a technical standpoint, there are no environmental conditions unique for their influence on acid attack. However, the specific *type* of acid involved in this ageing mechanism is highly relevant to the rate of progression (according to the solubility of the calcium salt of the acid's anion), and is likely dependent on local features of the natural or built environment that produce specific, known acid types. For example, the presence of hydrochloric acid (chemical industry), nitric acid (fertilizer, agriculture), acetic acid (fermentation), formic acid (food processing, dyeing), lactic acid (dairy), tannic acid (tanning, peat water), and sulphuric acid (sewers, microbial activity) are concerns, all of which may in theory be considered to derive from specific local "environmental" conditions. Of particular relevance to nuclear environments, attack by boric acid (e.g., due to boron's use in shielding applications), is noted to result in significant deterioration, though only at elevated (70°C) temperatures [33].

Effect of atmospheric conditions (rain, wind, freeze/thaw) Similar to the effects of moisture content or relative humidity, atmospheric conditions will only indirectly influence acid attack if they are such that they induce cyclic wetting and drying of the corroded concrete, e.g., rainfall/drought or freeze/thaw cycles, the drying stage of which would temporarily halt progression of acid attack, retarding the overall process.

Effect of mechanical and structural loading No direct observation of the influence of mechanical loading on acid attack is available. However, due to the emerging significance of such loading on dissolution of cement paste phases over long periods, such loading may accelerate acid attack [34].

4.1.2.2 Mixed design properties

References: A synthesis of the general influence of mix design properties is found in [28, 29].

Cement type and supplementary cementitious materials (SCM) Concrete with higher binder content of ordinary Portland cement (OPC) experiences a greater degree of acid damage, because it is the cement paste matrix that is the subject of this ageing process. For the second most (though still far less) common type of cement, calcium aluminate cement, this trend is reversed in part due to the higher aluminium content [35]. This is likely due to the intermediate products of acid attack (aluminium hydroxide) requiring relatively lower pH to continue subsequent dissolution [29]. Likewise, concrete mix designs containing higher silica and alumina contents, whether from cement type or addition of SCMs, result in lower degrees of acid damage. An additional feature of cement chemistries high in aluminate phases may also allow limited ion-exchange of inter-layer hydroxyl or sulphate anions to temporarily consume (e.g., buffer) the acid's anions [36], which would otherwise form more soluble calcium salts and exacerbate calcium dissolution.

Cement-water binder ratio Transport properties of cement paste are heavily influenced by water-binder ratio, which is closely linked with the porosity developed upon consumption of the water to form the hydrated cement paste phases that provide concrete its cohesion. Because acid attack is exacerbated in more permeable concretes, those with high water-binder ratio are more susceptible. Likewise, increased diffusion rate of dissolved ions through such concretes also contributes to faster progression of acid attack.

Concrete composition (aggregate content, types of aggregates) As outlined in the section on cement type, concrete with higher binder content of OPC experiences a greater degree of acid damage, because it is the cement paste matrix that is the subject of this ageing process, with the exception of calcium aluminate cements. As such, concrete with a higher aggregate content would logically be less susceptible to acid damage at a fixed water-binder ratio. Likewise, concretes whose aggregates are calcareous (e.g., calcium-rich) would also result in a less susceptible concrete due to a lower proportion of total calcium dissolved originating from the paste matrix.

Curing (temperature, moisture, time) conditions Stability of the corroded layer plays a large role in limiting ongoing acid attack. Because stability of this layer rests heavily on the mechanical integrity of the cement paste matrix, which gains strength with improved degree of reaction (i.e., dependent on curing conditions), more effective curing results in greater resistance to acid attack. This is especially true in the case where high-silica and alumina SCMs are added to the mix design, which undergo pozzolanic reactions to consume calcium hydroxide and form C(A)SH phases that are relatively more stable.

4.1.3 Influence of other ageing processes

4.1.3.1 Chemical/biological processes

Leaching (submerged or buried structures) Acid attack is a specific instance of leaching. If leaching occurs separately prior to acid attack, the associated changes to pore structure will accelerate the process upon introduction of acid.

Alkali-aggregate reactions Acid attack reduces pore water pH, precluding ASR. In the case of ASR that has occurred prior to acid attack, its influence would likely depend on how far this process has progressed: whereas the expansive siliceous gels formed during the early stages of ASR may reduce porosity and thus hinder transport of dissolved ions (and thus acid attack), ASR at a stage advanced enough to have caused appreciable cracking and mechanical damage would instead accelerate leaching of calcium via the cracks and enhance subsequent acid attack.

Carbonation Acid attack results in preferential dissolution of calcium carbonate, and thus concretes which have carbonated may be more susceptible. This may be counterbalanced by the porosity reduction implicit in the greater molar volume of calcium carbonate relative to calcium hydroxide, wherein transport (leaching) and thus the driving force for acid attack would be reduced.

Ettringite and thaumasite reactions (DEF/sulphate attack) Acid attack by sulphuric acid results in sulphate attack concurrently. Auxiliary effects relating to transport (leaching) are also likely, similar to other processes that result in precipitation of new phases in the pore structure, which would thus retard subsequent acid attack

Bacterial processes Acid attack may be induced by bacterial processes.

4.1.3.2 Physical-Mechanical

Freeze-thaw Acid attack will occur only when the pore water remains in the liquid state, e.g., being interrupted during freeze-thaw cycles. Auxiliary effects relating to transport (leaching) are also likely, similar to other processes that result in cracking and permeability increase, which would thus accelerate subsequent acid attack.

Elevated and high temperature (<150°C, no fire) High temperatures exacerbate acid attack by accelerating dissolution and transport kinetics. Auxiliary effects relating to transport (leaching) are also likely, similar to other processes that result in cracking and permeability increase, which would thus accelerate subsequent acid attack

Irradiation Irradiation may exacerbate acid attack by facilitating dissolution of calcium-bearing cement phases, though this has not been directly confirmed

4.1.3.3 Mechanical

Effects of (surface) cracks If cracks exist from other damage mechanisms, they effectively increase the permeability of the concrete in line with what is described in the section on leaching.

Abrasion/Erosion/Cavitation These may exacerbate acid attack whenever they render calcium-bearing cement phases more soluble, e.g., due to surface area increase. These are especially significant when they result in the full removal of weakened surface material.

Creep and relaxation The conditions conducive to significant creep may exacerbate acid attack, though the dissolution and subsequent effects induced by acid attack will likely obscure any creep that may otherwise have occurred in the absence of an external acid source.

Settlements and movements Other than exposing a greater surface area of concrete to aggressive water, such movements are unlikely to interact with acid attack.

Vibration (and seismic) These may exacerbate acid attack whenever they render calcium-bearing cement phases more accessible, e.g., due to cracking and subsequent permeability increase

Thermal stresses (gradients) These may exacerbate acid attack whenever they render calcium-bearing cement phases more accessible, e.g., due to cracking and subsequent permeability increase.

4.1.3.4 Electro-chemical

Pitting corrosion (ingress of chlorides or other aggressive species) This may exacerbate acid attack whenever it renders calcium-bearing cement phases more soluble, in particular due to the higher solubility of calcium chloride relative to other calcium salts.

General corrosion (carbonation or general pH decreasing processes) Other than carbonation, these already fall within the definition of acid attack. Those which produce sufficient changes to pore structure or phase balance prior to acute acid attack may initially accelerate the process.

Other corrosion mechanisms (e.g. crevice) This may exacerbate acid attack whenever it renders calcium-bearing cement phases more accessible, e.g., due to cracking and permeability increase.

4.1.4 Rates of deterioration

Deterioration rates vary depending on the specific acid, mix design, and quality of the concrete. In the broadest sense, kinetics are initially rapid, following a square-root of time dependence typical of diffusion controlled processes. However, it would be effectively meaningless to provide an estimation of a single order of magnitude for rate without specifying the acid-concrete combination of concern.

4.1.5 Impact on concrete properties

Chemical – Pore water chemistry – By definition, acid attack occurs *via* the pore water of the cement paste. It is the very change to the pore water chemistry (pH reduction) induced by the ingress of an external source of acid that goes on to induce the characteristic dissolution for this process, and subsequent leaching of calcium that maintains the driving force for ongoing dissolution.

Chemical – Solid phase composition – Solid phase composition is significantly altered upon acid attack, becoming calcium-deficient and silica- and alumina-rich. In specific cases, calcium-bearing phases may reform when the salt formed by the acid anion is relatively less soluble (e.g., gypsum).

Structural – Microstructure – Due to the dissolution of nearly all primary cement paste phases, the microstructure becomes significantly more porous. In cases where beneficial re-precipitation occurs, a semi-passivated state may be reached wherein the corrosion front ceases to progress, or at least slows substantially, after a certain degree of initial deterioration occurs.

Structural – Cracking – Although acid attack is not expected to directly cause cracking, subsequent mechanical weakening of the cement paste matrix will of course make the concrete more prone to cracking caused by other ageing processes.

Transport properties – Porosity, permeability, diffusion coefficients – As outlined in the effect of acid attack on microstructure, porosity, permeability, and diffusion coefficients within the corroded layer all increase due to acid attack, with the exception of specific acids that induce re-precipitation (e.g., sulphuric acid, during sulphate attack).

Mechanical properties – Due to inclusion of CSH, concrete's main load-bearing phase, in the set of phases that dissolve during acid attack, mechanical properties are severely compromised.

4.1.6 Assessment methods

Acid attack is most often assessed by weight loss and porosity change of the concrete [29]. It may also be assessed by microstructural analysis, and chemical characterization of the cement pore water and leachate solutions, particularly when other concurrent damage mechanisms are in effect (e.g., sulphate attack in the case of sulphuric acid [35].

4.1.6.1 Visual inspection

Visual inspection is often the first method by which attack by acid attack is observed, due to the often unexpected nature of this ageing process

4.1.6.2 Continuous monitoring

Subsequently, however, it is more common to use quantitative metrics such as weight loss to track its progress. In cases where acid attack is expected, samples that can be removed periodically for non-destructive or destructive testing are included in the design.

4.1.6.3 Destructive testing of sampling

Due to the importance of reducing permeability in mitigating acid attack, destructive testing of field structures is avoided whenever possible in favour of testing of analogous laboratory samples. In cases where it is merited, all common techniques for sample extraction may be employed. Typically, removable pieces or the accommodation in design for removable test pieces that are not part of the main structure are designed

4.1.6.4 Non-destructive techniques

The most common non-destructive technique for monitoring acid attack is weight loss (i.e., wherein pieces of the structure are designed for temporary removal for evaluation). Other in-situ non-destructive tests exist that would be suitable to monitor this process (e.g., embedded pH sensing), but are not commonly used with this process in mind.

4.1.7 Performance indicators & acceptance criteria

The extent (%) of weight loss over time is most frequently taken as the main performance metric, to account for cases where sufficiently corroded layers have been fully removed from the subject concrete (and are thus no longer present for other forms of testing). The limit state then, is the point at which recorded weight loss would result in an unacceptable reduction in element thickness, as relevant to its load bearing or other intended functionality being considered compromised. A formalized design code was not found, likely due to the case-specific nature of acid attack, and potentially also because acid attack and associated material removal can be adequately addressed using performance metrics for the other design features that are subsequently impacted. There is however, a method for evaluation of chemical resistance more broadly (ASTM 267 [37]) that may be taken as a model for such evaluations (e.g., weight change, dimensional change, splitting stress) in cases where the process of concern is acid attack specifically [38].

4.1.8 Model approaches

4.1.8.1 Phenomenological models

Existing phenomenological models have concentrated on coupling the effect of dissolution, precipitation, and transport resulting from the cement chemistry and composition of the contacting acidic solution with the influences of aggregate gradation and distribution within the corroded surface layer and periodic abrasion, e.g., as a result of fluid flow across the concrete surface. These models adequately modelled interaction of cement paste and mortar with acetic, nitric, hydrochloric, and sulphuric acid solutions [39, 40].

4.2 Leaching (submerged/buried structures)

4.2.1 Process Definition

The term “concrete leaching” refers to the process of extraction of different chemical elements from the porous space and solid phases of concrete due to the contact with an aggressive (generally acid) solution.

It is triggered by concentration differences of different ions between the concrete interstitial solution (present in the concrete porous space) and the solution in contact with concrete (river water, transported water in case of pipes or drain channels, …).

The concentration gradient induces a diffusion of ions inwards and outwards of the concrete, modifying the interstitial solution composition and hence, disturbing the chemical equilibrium between the interstitial solution and the concrete solid phases, mainly the hydrated cement phases. As a result, some solid phases will dissolve depending on their solubility, feeding the process of extraction of chemicals from concrete to the outside solution.

Leaching is the result of simultaneous diffusion and chemical reactions (dissolution/precipitation). It is generally accepted that diffusion is by far the slowest mechanism and hence, its kinetics drives the overall kinetics.

The pH of Portland cement concrete is between 13 and 13.3. If such a concrete is put into contact with a lower pH (for example water with pH 7), first alkalis will be leached, and then, the equilibrium will be dictated by dissolution of portlandite ($Ca(OH)_2$ or CH) which occurs at pH 12.5 (T = 25°C), following the equation:

$$Ca(OH)_2 \Leftrightarrow Ca^{2+} + 2OH^- \tag{2}$$

The pH will remain at this value until CH is depleted, and then, will be free to decrease and promote the dissolution or decalcification of other hydrates [41]. Therefore, multiple dissolution fronts progress towards the core of the concrete member, as shown in Figure 2.

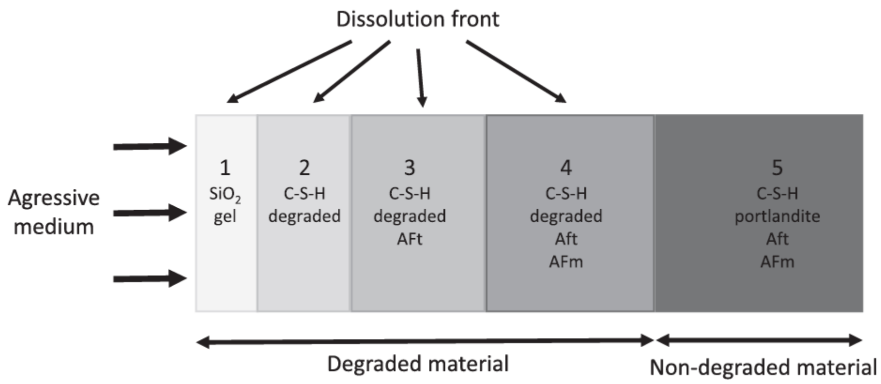


Figure 2. Schematic diagram showing the degradation of a cement paste subjected to leaching by pure deionized water stabilized at pH 7, according to Adenot, reproduced from L'Hostis and Naus [42].

This document addresses the case of structures in permanent contact with water.

References: For further references, the reader can consult Phung [43].

4.2.2 Influential factors

Reference: Syntheses on influential factors are given in Kamali [44], Kamali *et al.* [45] and Glasser *et al.* [46]

4.2.2.1 General conditions

Effect of moisture content and relative humidity High moisture environments promote alkali leaching since the diffusion of ions is faster when the liquid phase is connected [47].

Effect of temperature The kinetics of degradation by hydrolysis or decalcification of cementitious materials is accelerated by the increase of temperature [48], due to the acceleration of diffusive transport. Even though the solubility of portlandite decreases with temperature, this effect is not sufficient to stop the acceleration related to the transport kinetics.

Effect of irradiation Not relevant because the heating due to irradiation dries the concrete.

Effect of microbiological reactions The growth of micro-organisms on the surface of concrete can lead to deterioration, which is observed for example in sewage systems, and also in potable water systems [49]. These bacteria can create an acid film, which can reach 3 to 5 mm at the surface of the concrete element, promoting concrete leaching.

Effect of exposure condition (free, sheltered, buried, submerged) Leaching is promoted in submerged conditions, which are favourable for the diffusion of leached ions. Leaching is less a problem under atmospheric conditions. In buried conditions, saturation is high, and different chemical species can be present, see below.

Effect of external water composition (submerged or buried structures) The chemical composition of water is very influential. Some chemicals have the ability to enhance the pure leaching behaviour (e.g. presence of solubility increasing acids as nitric acids) while other products promote precipitation combined with leaching (when sulphate, magnesium, chloride etc. are present).

In case of presence of sulphates, decalcification and external sulphate attack can occur simultaneously, which means that portlandite will dissolve and ettringite and gypsum will precipitate. In case of presence of chlorides, chloro-aluminates precipitates can be formed inside concrete [41, 50, 51]. Specifically, chloride ions are preferred thermodynamically to displace counter-ions in AFm phases [52].

Effect of atmospheric conditions (rain, wind, freeze/thaw) Not relevant since in this document leaching of concrete in permanent contact with water is addressed. On the other hand, precipitation might temporally saturated the concrete (limited period of larger leaching rates) or during freezing, leaching may temporally being halted.

Effect of mechanical and structural loading A degradation of concrete by calcium leaching due to on-site water is possible. Experimental investigation on accelerated calcium leaching shows that leaching generates tertiary creep and rupture of the concrete [34, 53]. Similarly, mechanical loading's potential effect on dissolution during leaching may be to induce dissolution at points of stress (e.g., particle-particle contact points where the electrical double layers of dissimilar surfaces overlap [54].

4.2.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) In CEM V cement paste, much less portlandite is present, and generally the porosity (and hence diffusivity of ions) is lower, which has an influence on the development of leaching. The overall effect is generally that the dissolution front propagation is slowed down. Thus, the lower buffer capacity is less important compared to the effect on porosity [55, 56].

Cement-water binder ratio Two different effects can be considered: the effect on the porosity (a lower porosity leads to a lower permeability and diffusion kinetics), and the effect on the presence of residual anhydrous phases (these phases can be leached, leaving behind large pores) [57].As a high w/c ratio generally leads to higher porosity, leaching rate is directly influenced by the w/c ratio.

Concrete composition (aggregate content, types of aggregates) Limestone aggregates can also be dissolved by leaching [58]. Other types of aggregates are also influential since their porosity influences the overall concrete diffusivity and hence, the ions migration kinetics.

Curing (temperature, moisture, time) conditions Curing would largely be limited to its impact on porosity; greater degree of hydration for the cement, associated with optimum temperature, greater moisture, and longer time, would lead to lower porosity and thus a lower degree of leaching.

Specific conditions The leaching of concrete at the wall of spent fuel storage pools in NPPs can result in a cement attack by boric acid in the cooling water [59].

4.2.3 Influence of other ageing processes

4.2.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes)

Alkali-aggregate reactions AAR gels would block pores and reduce diffusion rates, if this damage mechanism has progressed significantly before the onset of leaching.

Carbonation Not practically relevant sin and ce carbonation is promoted by humidities around 60% while leaching is promoted by saturated conditions. On the other hand, if carbonation occurred before the leaching process, the porosity decrease the transformation of portlandite, and potentially C-S-H into calcite will influence the leaching process.

Ettringite and thaumasite reactions (DEF/sulphate attack) Changes in porosity and solid phase composition may influence the leaching process compared to pristine concrete.

Bacterial processes Bacteria can be at the origin of the acidity at the surface of concrete and hence, can contribute to leaching.

4.2.3.2 Physical-Mechanical

Freeze-thaw If microcracking occurs due to freeze-thaw, it might accelerate ion diffusion into concrete and promote leaching.

Elevated and high temperature (<150°C, no fire) Not relevant because very high temperatures lead to drying of the concrete

Irradiation Not relevant because irradiation dries the concrete

4.2.3.1 Mechanical

Not relevant (except in the case of thin pipes where the decrease of mechanical properties of the leached concrete surface induced a significant strength reduction and hence can increase the risk of failure).

Effects of (surface) cracks – Leaching is modified by the presence of cracks because they modify water transport and also because leaching can take place at the lips of the crack [60].

Abrasion/Erosion/Cavitation Not relevant in the absence of dynamical loads

Creep and relaxation Not relevant

Settlements and movements Can promote leaching if cracking occurs and ions exchange with the external solution is promoted.

Vibration (and seismic) Can promote leaching if cracking occurs and ions exchange with the external solution is promoted

Thermal stresses (gradients) Can promote leaching if cracking occurs and ions exchange with the external solution is promoted

4.2.3.2 Electro-chemical

Pitting corrosion Not relevant except in the context of pipes exposed to marine water.

General corrosion General corrosion and leaching can have coupled effects in the case of reinforced concrete [61].

Other corrosion mechanisms including crevice corrosion Not relevant except for chloride water.

4.2.4 Rates of deterioration

The kinetics depends mostly on the type of concrete (the larger the amount of portlandite, the faster the deterioration), the temperature, the pH of the solution and the chemistry of the external water, see [62].

4.2.5 Impact on concrete properties

Chemical – Pore water chemistry – The contact with external water changes the pore solution properties, depending on the composition of external water[41]. he contact with external water changes the pore solution properties, generating concentration and pH gradients both via diffusion and dissolution/precipitation according to Le Chatelier’s principle. This is the origin of leaching for example, wherein the contact of external water with a lower calcium concentration results in outward diffusion of pore water calcium, and the concurrent dissolution of calcium-containing solid phases results in re-elevation of pore water calcium concentration. Effectively, all induced chemical changes to the concrete by a given composition of external water amount to processes that promote re-establishment of chemical equilibrium by reducing the difference in composition of the pore water relative to this external water.

Chemical – Solid phase composition – The solid phase composition evolves according to Figure 2 if the external water is simply clear water, and can evolve in a more specific way if other products (sulphates, chlorides …) are present in the external water.

Structural – Microstructure – The pore space changes due to dissolution of different hydrates (in the first place portlandite). The porosity increases in the case of pure water where no precipitates appear[63]. When the water properties induce precipitation in the pore space, the porosity does not necessarily decrease[64].

Structural – Cracking – Experimental investigation on accelerated calcium leaching shows that leaching generates tertiary creep and rupture of the concrete[65].

Transport properties – Porosity, permeability, diffusion coefficients – In relation with the porosity increase, ion diffusivity increases. In lab experiments increases between 1 and 2 orders of magnitude have been recorded[66].

Mechanical properties – The dissolution of part of the solid skeleton of cement paste induces a loss of mechanical properties[67]. The magnitude of the property loss depends on the fraction of dissolved hydrates.

4.2.6 Assessment methods

4.2.6.1 Visual inspection

In the case of cooling towers, concrete attack by the acid treatment have been observed. In the case of spent fuel pool, there are automated inspection of the state of corrosion of the liner and of the possibility of leaching of the concrete wall by boric acid water.

4.2.6.2 Continuous monitoring

Not applicable.

4.2.6.3 Destructive testing of sampling

Cores can be taken to determine the depth of progression of leaching by different techniques:

- pH assessment with phenolphtaleine
- Measurement of presence of portlandite with TGA
- Microstructure assessment through SEM images

4.2.6.4 Non-destructive techniques

Ultra sound wave velocity tests might in some case reveal the degradation of the concrete by leaching [68].

4.2.7 Performance indicators & acceptance criteria

The acceptance criterion is the same as for carbonation: if the portlandite dissolution front reaches the steel reinforcement special care is needed to assess the corrosion kinetics and mitigation measures must be taken.

For thins structures such as pipes, the structure strength can be affected by leaching. No direct acceptance criteria can be readily proposed in this case: an evaluation of the bearing capacity must be done.

4.2.8 Model approaches

4.2.8.1 Empirical models

Some empirical models have been developed to link the leached depth to various factors (environmental and concrete properties) [45].

4.2.8.2 Phenomenological models

Phenomenological and simplified approach of the decalcification and the consequences of ionic attack have been developed in [69].

More recently a model using transport equations combined with Ca solubility have been developed [70].

4.2.8.3 Complex coupled models

Different complex models can be used to predict leaching. These models are reactive transport models, which model:

- Chemical species diffusion in the porous space
- Chemical equilibrium (liquid and solid)
- Back-coupling change in mineralogy – porosity- transport properties

Such models are implemented in the software HYTEC for example [71] or PHREEQC [72] and continue to be improved [73].

These models are fed with thermodynamic data to solve for the local chemical equilibrium. For instance, the solubilities of all considered precipitates must be known (portlandite, different kinds of C-S-H, ettringite, monosulfoaluminates, gypsum, calcite …). Some crucial data for leaching modelling can be accessed through the CEMDATA18 database [74].

The transport kinetics for all ions must also be known.

The coupling between transport and chemistry is obtained by an iteration loop seeking equilibrium for both phenomena.

4.3 External sulphate-attack

4.3.1 Process definition

External sulphate-attack is a process by which ingress of sulphate ions, typically from a contacting body of water (e.g., the sea, sulphuric acid, etc.), induces conversion of the lower-volume mono-substituent aluminoferrite phase (AFm) to its higher-volume tri-substituent counterpart (AFt, or ettringite) [28]. While this process is quite similar in nature and manifestation to delayed ettringite formation (DEF) it is not limited by the initial phase (im)balance of the cement paste (e.g., dependent on curing temperature). Rather, external-sulphate attack, when allowed to run its course, is limited only by the aluminate content of the cement binder; for this reason “sulphate-resistant” binders incorporate low or no reactive calcium-aluminate phases. Effectively, the extent of damage from this mechanism can be far greater than that caused by DEF due to its ability to progress using any reactive aluminium source (e.g., CASH), but due to transport limitations constraining ingress of external sulphates (the interplay with available porosity as pore-filling and expansive phases like ettringite form), likely progresses at a different rate [75]. While it should be noted that the mechanism of expansion following ettringite formation remains controversial, broad agreement exists regarding chemical origin of this process and appropriate mitigation measures (focused on the cement chemistry).

4.3.2 Influential factors and consequences

4.3.2.1 General conditions

The primary factor of the environment contributing to external sulphate-attack is, of course an external sulphate source, i.e., either via intermittent our continuous exposure to water.

References: Syntheses on influential factors are given in [76].

Effect of moisture content and relative humidity Moisture plays a critical role, as the balance of cement hydrates is dictated also by the chemical activity of water; due to the high hydration state of ettringite, moist conditions favour its formation. While thermodynamic data suggest a relative humidity (RH) in excess of 62% is required for ettringite formation to be possible [77], typically higher humidities (RH > 80%) are reported for being necessary to induce sulphate attack progression [78].

Effect of temperature Generally, temperature accelerates the rate of the chemical reactions (e.g., dissolution, diffusion, precipitation) involved in external sulphate-attack. However, due to the dependence different phase stabilities and solubilities on temperature, in practice an optimum intermediate environmental temperature (e.g., 35°C) has been found to favour progression of this deterioration mechanism [78].

Effect of irradiation Though direct results on the interplay between irradiation and external sulphate attack are not available, two potential impacts consist of (1) retardation of sulphate attack due to either temperature-induced or radiolysis-induced drying, and/or (2) acceleration of sulphate attack due to disordering of cement paste phases containing aluminium, which will subsequently dissolve faster during sulphate-attack.

Effect of microbiological reactions Any microbiological reactions in proximity to the concrete that produce sulphate can act as instigators for sulphate attack. Those which produce biofilms at the concrete surface, however, may inhibit transport and slow the rate [32].

Effect of exposure condition (free, sheltered, buried, submerged) Exposure conditions relevant to sulphate attack are those such that contact (intermittent or continuous) with an external body of water (containing sulphate ions) is maintained, e.g., submerged structures, or those not sheltered from rain/runoff.

Effect of external water composition (submerged or buried structures) External water composition must contain sulphates to induce sulphate attack. Additionally, presence of chloride ions also mitigates sulphate attack due to the stabilization of chloride-AFm phases rather than sulphate-AFm phases (as the inter-layer charge balancing anion), inhibiting conversion of these aluminate phases into ettringite [79]. Due to the diverse array of interlayer anions these phases can accommodate (e.g., carbonate, nitrate, hydroxyl, others), there may also be other compositions that inhibit external sulphate-attack.

Effect of atmospheric conditions (rain, wind, freeze/thaw) Rain and freeze-thaw conditions will impact sulphate attack to the extent that they result in appropriate moisture environments for the deterioration to progress (e.g., not frozen, not dry).

Effect of mechanical and structural loading Mechanical and structural loading are not well-established to influence sulphate attack, but may influence its manifestation in the form of oriented cracking parallel to the direction of maximum applied load (due to the Poisson effect preferring ettringite-induced expansion transverse to this direction).

4.3.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) – Cement type and supplementary cementitious materials are both extremely influential in dictating the course of external sulphate attack. Low-aluminate (sulphate resistant) cements are far less susceptible because there is less available material to form ettringite, while the under- or over-sulfation caused by an improperly proportioned aluminate/sulphate-bearing SCM (e.g., high-calcium fly ash, slag, etc.) can exacerbate the process. Typically, however, SCMs such as fly ash, slag, silica fume, and metakaolin are reported to both "dilute" the aluminate content and result in more stable cement hydrate phases (higher silica content) forming a more compact pore structure (due to particle gradation), all of which inhibit external sulphate-attack [76].

Cement water-binder ratio – Higher water-binder ratios increase porosity and decrease mechanical strength of concrete. The net effect serves to result in accelerated sulphate attack with higher water-binder ratio, mainly due to the increased permeability for sulphate ingress caused by higher porosity [80].

Mix design properties (aggregate content, type of aggregates, …) – Typically, it is reported that diffusion of species is enhanced at the interfacial transition zone (ITZ) between cement paste and aggregates. As such, any aggregate characteristics (gradation, surface chemistry) that would promote formation of a more porous ITZ would promote sulphate attack [81].

Curing (temperature, moisture, time) conditions – Because improved curing results in higher degree of hydration of the cement paste, and commensurate reductions in porosity and permeability, curing conditions that are engineered as such inhibit later sulphate attack, while inadequate or improper curing can leave concrete more susceptible [82]. In worst-case scenarios, DEF and external sulphate attack may occur concurrently if initial curing temperature is too high.

4.3.3 Influences of other ageing processes

4.3.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) This process typically proceeds more rapidly than sulphate attack, but may be further exacerbated when the acid in question is sulphuric acid.

L**eaching (submerged or buried structures)** This process typically promotes sulphate attack, as pore water alkalinity is reported to inhibit this process, and is reduced following leaching.

Alkali-aggregate reactions This process would most likely occur only in highly alkaline pore solutions, but prior such reactions sufficient to result in appreciable cracking to facilitate transport would exacerbate subsequent sulphate attack.

Carbonation By reducing pore water pH, this would indirectly promote sulphate attack if the processes occurred concurrently.

Ettringite and thaumasite reactions (DEF/sulphate attack) This process is effectively equivalent to external sulphate-attack, but is induced specifically by improper curing conditions resulting in premature formation of AFm phases before gypsum is depleted.

Bacterial processes Bacterial processes that create a sulphate source could drive sulphate attack.

4.3.3.2 Physical/mechanical

Freeze-thaw Freezing would temporarily halt sulphate attack. Induced cracking could then subsequently accelerate it following thaw.

Elevated and high temperature (<150°C, no fire) Sufficient drying due to temperature extremes would permanently alter the pore structure of CSH, possibly limiting transport and inhibiting sulphate attack. However, thermal stresses and associated cracking may compensate and rather exacerbate sulphate attack.

Irradiation Irradiation would likely impact sulphate attack by either (1) halting progress in the case of sufficient radiation-induced drying or radiolysis of water, or (2) accelerating the process by enhancing the dissolution rate of AFm phases following neutron collisions.

4.3.3.3 Mechanical

Effects of (surface) cracks – Surface cracks sufficient to facilitate transport of external sulphates into the concrete are sufficient to accelerate sulphate attack.

Abrasion/Erosion/Caviation Unlikely to impact sulphate attack unless associated with cracking.

Creep and relaxation In the case where the hypothesis for creep being caused by dissolution-precipitation processes of CSH induced in zones of increased stress, similar localized dissolution-precipitation reactions may be favoured during sulphate attack, accelerating the process concurrently with creep progression.

Settlements and movements Unlikely to impact sulphate attack unless they result in exposure to a new source of sulphate.

V**ibration (and seismic)** - Unlikely to impact sulphate attack.

Thermal stresses (gradients) Unlikely to impact sulphate attack unless associated with cracking, though **sustained** zones of high temperature would accelerate ongoing sulphate attack as well.

4.3.3.4 Electro-chemical

Pitting corrosion (ingress of chlorides or other aggressive species)

General corrosion (carbonation or general pH decreasing processes)

Other corrosion mechanisms (e.g. crevice)

4.3.4 Impact on concrete properties

Chemical – Pore water chemistry – Pore water chemistry, specifically equilibrium of aluminate and sulphate phases, is the driving force behind this ageing process. To maintain the effects of the mechanism, a higher concentration of sulphate must be maintained. Additionally, presence of specific other species, i.e., magnesium, is noted to also play a significant role in inducing or preventing additional damage during sulphate attack (e.g., via brucite formation) [83].

Chemical – Solid phase composition – Transformation of AFm to Aft (ettringite) is the primary alteration in solid phase composition. In high enough concentrations of sulphate in the pore water, gypsum may also form; likewise at low enough moisture content, other sulphate salts may crystallize.

Structural – Microstructure – More permeable microstructures facilitate sulphate attack.

Structural – Cracking – Cracking increases permeability, facilitating sulphate attack.

Transport properties – Porosity, permeability, diffusion coefficients – Transport properties regulate the diffusion of sulphate into the concrete, and are rate limiting in most instances of sulphate attack.

Mechanical properties – Mechanical properties dictate the concrete's response to ongoing sulphate attack, i.e., whether or not visible cracking and damage manifests for a given degree of reaction progress.

4.3.5 Rates of deterioration

Due to the primary dependence of external sulphate-attack on inward diffusion of sulphate species, the rate of deterioration can be highly dependent on the permeability of the concrete. Generally this mechanism would parallel the rate of other transport-controlled processes like leaching, chloride ingress, etc. with the caveat that the diffusion constant of sulphate ions specifically is likely higher than that of relatively smaller monatomic species like calcium or chloride (in the absence of other factors that would subsequently bind or immobilize these species and/or sulphate).

4.3.6 Assessment methods

Due to similarity to delayed ettringite formation (DEF), many assessment methods for external sulphate attack can mirror those used to evaluate this damage mechanism. However, one significant difference is the ability to monitor and quantify external concentrations of sulphate.

4.3.6.1 Visual inspection

While visual inspection by crack mapping is possible (as for DEF [84]), standards more frequently prefer classification of exposure condition for the assessment, given its predictive utility in addition to greater ease of continuous monitoring [83].

4.3.6.2 Continuous monitoring

The ability to externally monitor sulphate concentrations assists in continuous monitoring of external sulphate attack, without the need for embedded sensing as in the case of DEF. However, modelling efforts lag behind in their ability to extrapolate directly from the composition of external water to the induced deterioration.

4.3.6.3 Destructive testing of sampling

Destructive testing focuses on both strength evaluations (compressive, preferably) and microscopy, although there is ongoing debate regarding appropriate criteria in both cases. Due to the also still-debated mechanism of sulphate attack-induced damage (i.e., the mechanism of expansion), a broad array of assessment data that also include external monitoring and visual inspection are typically required to provide strong confirmation that the source of observed damage from destructive testing is indeed external sulphate attack [83].

4.3.6.4 Non-destructive techniques

Non-destructive testing techniques have shown success in tracking changes induced by sulphate attack in laboratory environments, but fill largely the same role as destructive tests, requiring further data from other assessment methods to confirm sulphate attack [85].

4.3.7 Performance indicators & acceptance criteria

Although acceptance criteria relating to understanding of the detailed mechanisms of sulphate attack are lacking, there are several empirical indicators that are frequently used. Most common is the basic specification of a maximum tricalcium aluminate content for the cement based on the expected exposure environment, supplemented by more broadly applicable criteria for design strength of concrete in a given application [83].

4.3.8 Model approaches

Modelling, whether empirical or theoretical in focus, tends to split between three general emphases: transport processes, chemical reactions, and the source of expansion pressure [86].

4.3.8.1 Chemistry and transport models

While the fundamentals behind transport of ions and stability/solubility of cement phases in controlled environments are well-established by empirical models, the applicability of these models is severely limited by their (in)ability to also account for the concurrent evolution in pore structure (and thus transport and solution composition) resulting from the resulting expansive forces and mechanical damage [86].

4.3.8.2 Micro-mechanical models

Models for mechanical manifestation of damage are split between the two prevailing hypotheses for how ettringite precipitation translates to expansion: volume increase approach, and crystallization pressure theory. The main distinction from a general standpoint is that the latter depends more on the location of ettriginte, whereas the former depends primarily on the volume that has formed. Neither has been conclusively validated, likely indicating a difference between locally generated expansion and that which manifests at the macroscopic scale that confounds efforts to experimentally isolate and confirm the underlying mechanism.

4.3.8.3 Mechanical models at the mesoscopic and macroscopic scale

Despite widespread efforts to extend micro-mechanical models to structural scale, accurate modelling of external sulphate attack has been hindered by the ambiguity regarding which hypothesis for micro-scale damage predominates. Most modelling approaches incorporate assumptions consistent with volume increase, whereas crystallization pressure theory has given more promising initial indications, but requires significant additional development justify the increased complexity of incorporating it into models (i.e., detailed knowledge of activity of each chemical species). Chemical damage due to dissolution of calcium-containing phases is also inadequately quantified, and may as well result in improved scalability of such models.

4.4 Carbonation

4.4.1 Process Definition

Carbonation is the process in which hardened cement paste hydrates are converted to calcium carbonate. The main sequence of processes are the diffusion of gaseous CO_2 through the unsaturated cement paste (or dissolved inorganic carbon via the aqueous phase), dissolution of gaseous CO_2 in the aqueous phase, dissolution of hydrated phases and precipitation of carbonates as calcite but also polymorphs as aragonite and vaterite can be formed. Dissolution of CO_2 in the aqueous phase, following Henry's law, initiates a number of aqueous dissociation reactions with the carbonate ion as the main abundant species at the high pH (above ~10 pH) in cement pore water (pH is above 12 in fresh ordinary Portland cement). Formation of carbonate ions produces water and lowers the pH. When the cement pore water is oversaturated with calcite, calcite will precipitate lowering the Ca concentration and triggering dissolution of cement hydrates, first portlandite and, once depleted, also calcium-silicate-hydrates and other hydrates. During carbonation, a so-called carbonation front develops from the surface to deeper in the cement, with a pH lower than 9-10 before the front (closer to the surface) and higher pHs beyond the front. In reality, a diffuse front will develop with pH and the solubility of Ca controlled by portlandite, C-S-H (changing Ca/Si ratio) or other hydrates [87].

References: For further references, the reader can consult Glasser *et al.* [46], Ashraf [88] and von Greve-Dierfeld *et al.* [89], the latter describing also the influence of supplementary cementitious materials.

4.4.2 Influential factors

Reference: Syntheses on influential factors are given in [90, 91].

4.4.2.1 General conditions

Effect of moisture content and relative humidity – The fastest carbonation rate is obtained at a relative humidity range of about 40-60% [92] or 60-80% [90] – typically it is set between 50-70%; the optimal value is strongly related to the microstructure of the cement mainly controlled by the water/cement ratio and the curing conditions. With lower values, water content is insufficient for CO_2 dissolution and the other aqueous reactions. Higher values decrease the gaseous phase content reducing the diffusion of CO_2 in the gaseous phase.

Effect of temperature – The temperature changes many factors that influence the carbonation rate [93-95]; temperature affects aqueous concentrations of calcium and carbonate ions, diffusion coefficients (increasing diffusivity with temperature), and water retention (higher temperature decreases the water retention at the same relative humidity). Experimental evidence shows an increase of carbonation rate with temperature at temperature below 60-80 °C [94, 95], but the temperature at which the fastest carbonation rate occurs depends on other factors (cement type, RH, …) as well. Some studies show also that above an optimal temperature, carbonation rates may decrease again (e.g. above 60 °C in the study of Liu *et al.* [96], above 90°C in Wang *et al.* [95]).

Effect of irradiation – Several studies [97, 98] mentioned that gamma-ray heating and water radiolysis influence the carbonation process and may increase the degree of carbonation [99-101]. Maruyama et al. [98] observed that the calcium carbonate polymorphs vaterite and aragonite are formed rather than calcite. It seems that strength is enhanced by irradiation via the formation of vaterite.

Effect of microbiological reactions – Some bacterial groups (heterotrophic bacteria) produce CO_2 during their metabolism while other consume it (autothropic bacteria, see section 4.7.1). This may change locally the CO_2 content and thus change potentially carbonation rates. The influence of bacteria on carbonation is also promoted in so-called self-healing cement types with microbiologically-induced calcite precipitation for sealing microcracks (MICP, [102]).

Effect of exposure condition (free, sheltered, buried, submerged) – Differences in carbonation rates between free or sheltered conditions are mostly related to differences in environmental conditions such as temperature, relative humidity and CO_2 partial pressure [90]. Under buried or submerged conditions, the conditions in the cement-based materials is near or at saturation which limit significantly the inwards diffusion of CO_2 and carbonation, even if in some buried conditions, the CO_2 partial pressure is larger than in the atmosphere (e.g. in soils). Simultaneously leaching reactions will influence the carbonation rate, because leaching typically increase porosity and diffusivity and consumes the same cement mineral phases (portlandite and C-S-H).

Effect of external water composition (submerged or buried structures) **–** The inorganic carbon concentration (and thus the CO_2 partial pressure) could have a direct impact on the carbonation rate, specifically a large microbial activity can lead to partial pressure more than one order of magnitude larger than in the atmosphere. However, in buried or submerged conditions, diffusion limitations to due to high water saturation in the concrete will have a larger impact on the carbonation rate. In addition, the consequences of the simultaneous leaching process (that also strongly depends on the aggressivity of the external water) will affect the carbonation rate.

Effect of atmospheric conditions (rain, wind, freeze/thaw) – Unsheltered (influenced by rain) or sheltered conditions is one of the environmental conditions that have a large effect on the carbonation rate [91] with sheltered condition leading to higher carbonation rates (e.g. [103]). The pores in concrete structures that are exposed to intermittent rainfall events may be blocked by full saturation during rain events or wet conditions. The study of [91] indicates that the ratio of carbonation rates between sheltered and unsheltered conditions is between 0.4 to 0.65 depending on the mechanical strength (a durability indicator for concrete quality) which in line with other recommendations.

Effect of mechanical and structural loading – Concrete under tension loading will increase the carbonation rate due to the formation of micro cracks that accelerate the diffusion of CO_2 [104] – see below for a somewhat more comprehensive discussion on the effect of cracks on carbonation. However, coupled compression loading and carbonation is a complicated process. It is generally assumed that up to a certain loading (e.g. 30-40% compressive strength), the compression loading reduces the carbonation rate due to the confinement of the pores resulting in a reduction in CO_2 transport. Nevertheless, if the loading increases more than such level (e.g. 50-60% compressive strength), the microcracking might happen leading to a faster carbonation rate. This work is now under investigation of RILEM TC281-CCC.

4.4.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) – Supplementary cementitious materials influence the carbonation resistance in several ways; with the amount and nature of the carbonatable hydrates and the developed microstructure being the most important ones. For example, concrete based on cement with fly-ash replacement shows a faster carbonation rate [88, 89, 94, 105]. Due to pozzolanic reactions, the amount of calcium to be carbonated is less with thus a decrease in carbonation resistance[4] [88, 106]. Different SCMs have other effects on carbonation rates [89] with a decreasing carbonation rate from replacement by fly ashes or silica fume, limestone, and blast furnace slags. Rules of thumbs given in [89] indicates that for a replacement level of 25%, carbonation is about 2.3, 1.9 and 1.3 faster for fly ashes/silica fume, limestone and blast furnace slags, respectively compared to plain Portland cement. Note that this has only a limited precision as the carbonation resistance depends on many additional factors as binder specific alteration to porosity, reaction products, degree of hydration, etc.

Cement-water binder ratio – The water/cement ratio is an important factor; carbonation rate increases with increasing water/cement ratio [89, 107, 108]. This is immediately linked to the more open microstructure in cements, mortars and concretes that are have a higher water/cement ratio. The more open microstructure leads to a larger porosity, diffusion coefficient and permeability. Furthermore, the portlandite content is also smaller for samples with higher water/cement ratios.

Concrete composition (aggregate content, types of aggregates) – There is a tendency that the carbonation rate increases with increasing aggregate/cement content, probable linked to a more porous microstructure and portlandite rich interfacial transition zone [92, 109]. However, also a decrease in aggregate content may lead to faster carbonation which has been linked to more autogenous and drying shrinkage and a larger porosity (the latter in case of hardened cement paste) [110]. In line with that, carbonation rate is typically higher for hardened cement paste compared to mortar and concrete, especially in blended systems [89].

Curing (temperature, moisture, time) conditions – As curing has a major effect on the microstructure of the hardened cement paste, curing conditions can have an effect on carbonation resistance. Curing conditions that densify the microstructure will increase the carbonation resistivity by limiting the inward diffusion of CO_2. Longer curing times [108] or curing under wet conditions (e.g. [111]) increases degree of hydration, strength or densification of the microstructure and may contribute to a slower carbonation rate. However, on the other hand, the study of [91] does not shown statistically significant differences at a confidence interval of 95% between different curing regimes.

4.4.3 Influence of other ageing processes

4.4.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) As attack by acid water already decreases the amount of the typical cement phases (e.g. portlandite, C-S-H),

[4] The inverse of the carbonation rate

the resistivity against carbonation is expected to decrease after interaction with aggressive water.

Leaching (submerged or buried structures) As leaching already decreases the amount of the typical cement phases (e.g. portlandite, C-S-H), the resistivity against carbonation is expected to decrease after leaching. Leaching typically leads also to higher porosity and will thus increase CO_2 diffusion as well. The degree of leaching depends on the water composition, sea water being more aggressive compared to typical soil, ground or river water.

Alkali-aggregate reactions ASR results in the formation of cracks which increases the CO_2 diffusion, thereby the carbonation rate.

Ettringite and thaumasite reactions (DEF/sulphate attack) The most important factor is that these degradation processes induces cracks in the concrete. These cracks will act as preferential paths for CO_2 diffusion and thus carbonation. There might be also some effect in the amount of carbonatable material.

Bacterial processes Microbes and their activities can alter the contact angle of solid surfaces and consequently have an effect on the moisture retention curve. As such, they may have an influence on the water saturation at a given relative humidity and thus influence the diffusion process. Note that this is a rather theoretical effect not studied for concrete structures to the best of our knowledge (some work on effect of microbes on the moisture retention curve in porous media is available). Microbiologically-induced deterioration may result in an increase in porosity [112], reducing strength and promoting of crack growth which in turn may accelerate carbonation. Note that this is observed for specific applications such as sewage collections systems in which sulphur oxidizing bacteria produces acidifying conditions with sulphate or sulphuric acid.

4.4.3.2 Physical-Mechanical

Freeze-thaw As freeze-thaw processes cause microstructural degradation which may influence the carbonation rate (similar to effects of surface crack). An increase in carbonation rates after ice-induced deterioration of the microstructure was experimentally observed in the study of [113].

Elevated and high temperature (<150°C, no fire) Elevated temperature decomposes some hydrates (e.g. C-S-H, AFt) [114] resulting in crack formation and accelerating the carbonation process.

Irradiation The study of [115] investigated the sequential process of irradiation – carbonation and observed that carbonation depth increased when hardened cement paste samples were pre-treated with gamma irradiation with a clear change in the carbonate polymorph. The effect of a simultaneous process was described above.

4.4.3.3 Mechanical

Presence of (surface) cracks Surface cracks have a significant effect on carbonation rates as they facilitate the diffusion of CO_2 inside the cement and concrete; there is a relation between diffusion rate and crack width in the 0.1-0.4 mm crack width range [116]. Microcracks below 0.3 mm might increase the carbonation depth significantly [117-119].

As discussed above, carbonation increases when the microstructure is more open and when crack and microcracks are present. Any of the mechanical degradation processes that cause an increase in porosity, microstructure or cracks will have a consequence on the carbonation rate.

Abrasion/Erosion/Cavitation see above

Creep and relaxation see above

Settlements and movements see above

Vibration (and seismic) see above

Thermal stresses (gradients) see above

4.4.3.4 Electro-chemical

Pitting corrosion

General corrosion

Other corrosion mechanisms including crevice corrosion

4.4.4 Rates of deterioration

Some general rates of deterioration are given based on the study of [90]. It has to be acknowledged that many other datasets exist on carbonation rates in concretes of different composition and in various environmental conditions. Although the synthesis was based on empirical data from a Nordic climate, it is chosen here as it give indications for different strength classes (as a proxy indicator for concrete durability) and environmental conditions that may be relevant for NPP environments as well. We report here only the strength class larger than 35 MPa.

Table 6. Carbonation rates (mm/year0.5, see also k in Eq. (3)*) for different environmental classes for the concrete strength class > 35 MPa as reported by [90]).*

Wet/submerged	Buried	Exposed	Sheltered	Indoor
Surroundings are completed saturated	For example, in soil which are not fully saturated	Exposed to atmospheric conditions, specifically rain	Outside, but protected from rain	
0.5	0.75	1	2.5	3.5

Note that a number of empirical models exists that make a more nuanced prediction of the carbonation rate accounting for different engineering and environmental factors. Some are referred to in section 4.4.8.1.

Andersson *et al.* [120] also list carbonation rates as a function for environmental classes and concrete strength class (see Table 7). Further details on calculation CO_2 uptake by existing structures are given in e.g. [120, 121].

Table 7. Carbonation rates (mm/year$^{0.5}$, see also k in Eq. (3)*) for calculation of depth of carbonation for different concrete strength class (cylinder) and exposure conditions and degree of carbonation (the amount of CO_2 uptake by carbonation in relation to the theoretical maximum CO_2 uptake by carbonation) for different exposure conditions for OPC (Table 2 in [120]).*

Concrete strength	≤ 15 MPa	15–20 MPa	25–35 MPa	≥ 35 MPa	Degree of carb
Parameters	Value of k-factor, in mm/year ^{0,5}				Percentage
Civil engin. structures					
Exposed to rain		2,7	1,6	1,1	85
Sheltered from rain		6,6	4,4	2,7	75
In ground [a]		1,1	0,8	0,5	85
Buildings					
Outdoor					
Exposed to rain	5,5	2,7	1,6	1,1	85
Sheltered from rain	11	6,6	4,4	2,7	75
Indoor in dry climate[c]					
With cover[b]	11,6	6,9	4,6	2,7	40
Without	16,5	9,9	6,6	3,8	40
In ground[a]		1,1	0,8	0,5	85

[a] Under groundwater level k = 0,2.
[b] Paint or wall paper. (Under tiles, parquet and laminate k is considered to be 0.)
[c] Indoor in dry climate means that the RH is normally between 45 and 65%.

4.4.5 Impact on concrete properties

Chemical – Pore water chemistry – In the carbonation process, cement hydrates are consumed and replaced by calcium carbonates. In a first phase, portlandite is replaced by calcium carbonate, but as long as it is present and in contact with the pore water, it will control the pore water chemistry. Only when it is disappeared or not in contact with the pore water, another hydrate will buffer the pore water chemistry. The sequence of hydrates that are carbonated results in a pH decrease with increasing degree of carbonation (see Fig. 1 in [89]), but will remain above 11-12 as long as portlandite and high Ca C-S-H is present (e.g. [122]). With increasing carbonate, we expect also a decrease in Ca concentration and an increase in Si concentration. Also the concentration of other elements as Al and S may change when AFm and AFt phases are carbonated.

Chemical – Solid phase composition – The primary polymorph of carbonation is calcite, although other metastable polymorphs (vaterite, aragonite) can be formed as well depending on cement type and carbonation kinetics. Carbonation of the anhydrous Ca-Si particles (C2S, C3S) can also lead to formation of amorphous calcite carbonates sometimes intermixing with decalcified C-S-H [88]. From a thermodynamic point of view, a well-defined sequence of hydrates will be carbonated which can be predicted with thermodynamic modelling tools (e.g., [122] see also section 4.4.8.2). However, from experimental work, it is observed that portlandite and C-S-H are carbonated simultaneously [123-125] which results in a silica-rich gel variant of C-S-H.

Structural – Microstructure – Carbonation has a distinct but complex effect on the microstructure of the hardened cement paste. Portlandite carbonation typically leads to a decrease in the larger capillary pores because the molar volume of the calcium carbonates is larger than that of portlandite. However, when C-S-H is carbonated as well, also the gel pore (radius < 10 nm) volume decreases. The evolution of the mesopore volume (radius > 10 nm) is more complex and depends also on the nature of the cement (e.g. differences between CEM I and CEM II as observed in [126]). Cements with lower Ca/Si ratios of the C-S-H (e.g. CEM II or blended cements) tend to have an increased mesopore volume after carbonation with decalcification of the C-S-H leading possibly to carbonation shrinkage [127].

Structural – Cracking – Carbonation of plain Portland cement systems results in a small porosity decrease which generally does not cause cracking. In blended systems, porosity will tend to increase rather than decrease. On the other hand, they are sensitive to carbonation shrinkage that may cause microcracks.

Transport properties – Porosity, permeability, diffusion coefficients – Again, carbonation of plain or blended cements give rise to different consequences of carbonation. In plain ordinary Portland cements, typically, a decrease in porosity is observed due to portlandite carbonation. However, in blended cements with blast furnace slag (BFS), or high fly-ash (FA) or silica fume (SF) replacement also a porosity increase or even cracking are observed in some studies (e.g. [128]). The consequences on permeability and diffusion coefficients were summarized in [89]. In plain ordinary Portland cements, permeability decreases in plain cement types and cements blended with limestone. Limited carbonation can already decrease the permeability by one-order of magnitude due to a decrease in both total porosity and shifting of the critical pore size to smaller pore radii [129]. On the other hand, in blended cement types with BFS, FA, and SF with moderate to high replacements, permeability decreases after carbonation. A similar consequence of carbonation on aqueous and gaseous diffusion coefficients is observed (e.g. [128]).

Mechanical properties Many researchers point out that the carbonation process offers several benefits including durability improvement, reduction of porosity [125], reduction of the average pore size [130] associated with the increased densification of the matrix through the precipitation of $CaCO_3$, which is a denser and more stable product than $Ca(OH)_2$ [131]. Such changes in microstructure and mineralogy are expected to result in an alternation in the mechanical properties. Unfortunately, not many works have covered this aspect. Typically there is a positive effect of carbonation on mechanical properties due to the denser microstructure. Several mechanical properties of concrete such as compressive strength and surface hardness may change due to carbonation. Chi *et al.* [132] reported that the 28 day compressive and splitting strengths of carbonated concrete were slightly higher than those of noncarbonated concrete[133]. Carbonation does not only increase strength, modulus of elasticity, but also the shape of the stress–strain curved as reported by Jerga [134]. The descending branch of the stress–strain curve indicates a more brittle behaviour of carbonated concrete. Shrinkage increments due to the carbonation up to 0.35‰ were reported [134].

Carbonation curing at early age is currently consider as a technology to reduce carbon dioxide emission and to improve the mechanical properties of cementitious materials. Almeida *et al.* [135] studied carbonated cementitious specimens reinforced with bleached eucalyptus pulp, with a climate chamber set to 60°C temperature, 90% RH, and atmospheric CO_2 concentration (15 vol%) for two days. The results showed that the mechanical properties were better for the composites subjected to accelerated carbonation at early stages of hydration and carbonated samples had higher values of mechanical properties for all aging tests. Pizzol *et al.* [136] studied the accelerated carbonation process in cementitious specimens reinforced with bleached cellulosic pulp and synthetic fibres, with the same carbonation conditions as [135] for ten hours. Even with such a short curing time, the carbonated products exhibited a good fibre–matrix bonding, improved mechanical properties with a reduction of composite degradation. Santos *et al.* [137] studied the supercritical carbonation of extruded fibre-cement reinforced with bleached eucalyptus pulp and residual sisal chopped fibres, with specimens placed in a chamber with complete saturation of supercritical CO_2 at 20 MPa with the chamber immersed in water at 45 °C. The results showed that the supercritical carbonation treatment for only 2 h could significantly improve the physical characteristics and mechanical performance. The improvement in mechanical properties during carbonation curing is attributed to the denser ITZ between cement paste and fibre [138] and aggregates. However, excessive carbonation could induce a negative effect on mechanical properties due to the formation of microcrack due to carbonation shrinkage

4.4.6 Assessment methods

4.4.6.1 Visual inspection

None – Consequences from carbonation-induced corrosion can be visually inspected.

4.4.6.2 Continuous monitoring

Witness samples in the same environmental condition can be very informative to assess carbonation rates for specific materials and conditions. A throughout characterization is possible for a good assessment of the carbonation process and rate. Combined with empirical or more physically-based modelling approaches (see section 4.4.8), this can give important information on the long-term ageing by carbonation and an assessment on the risk of carbonation-induced corrosion.

4.4.6.3 Destructive testing of sampling

Testing

The standard method to evaluate the carbonation depth is by phenolphthalein spraying on broken samples, in small boreholes or small core samples. Phenolphthalein colours pink when in contact with alkaline conditions (pH > 9) and is colourless at lower values. The test is described in different standards, e.g. RILEM Recommendation [139]. The method gives only an approximated depth of the carbonated zone, because zones with a pH larger than 9 can be carbonated as well [88].
Cores can be used for SEM(-EDX) analyses as well. These allows for a qualitative and quantitative information on the extent of carbonation.

Intact cores

None

Disturbed sampling (e.g. dust)
A more nuanced insight in the carbonation progress can be obtained with thermogravimetric analysis of e.g. dust that has been collected from drilling small holes (e.g. a hole with a diameter of 8 mm collecting dust every 3 mm; diameter is smaller than small coring events of a diameter of 20 mm). This allows for quantification of portlandite and calcite mass profiles and for deducing even the amount of carbonated Ca other than portlandite

4.4.6.4 Non-destructive techniques

Carbonation has a pronounced and well-documented effect on mechanical and transport properties. As it is an ageing process that progress inside the concrete from the exposed wall, non-destructive tests could include measurement of surface properties and their evolution in time. For example, air permeability devices measure the air permeability that is mainly influenced by the permeability of the surface/cover concrete layer (e.g. Torrent air permeability). Mechanical properties can be measured such as compressive strength with the Silver Schmidt hammer. However, these methods do not assess or quantify the carbonation process as such, because other ageing processes influence these properties as well (e.g. small cracks will influence the air permeability measures). Also, although it is shown that e.g. air permeability is correlated with other durability indicators [140-142], no information, as far as we know, that evaluates the evolution of air permeability with time as a consequence of evolving carbonation.

4.4.7 Performance indicators & acceptance criteria

In most cases, carbonation as such is not a detrimental ageing process. The critical consequence of carbonation is when the carbonation front reaches the reinforcements bars initiating their corrosion which leads to the carbonation-induced corrosion detrimental consequences.

Therefore, the performance indicator is the depth of the pH 9-10 front, i.e. the pH below which corrosion of reinforcement bars accelerates. This front is clearly indicated by the phenolphthalein test. As carbonation is one of the most relevant processes to decrease the pH, this front is often called the carbonation depth. Although this does not mean that the material deeper in the material is still completely uncarbonated and the material closer to the surface is fully carbonated, this definition of the carbonation depth (as indicated by the colour change following a phenolphthalein test) is used to calibrate empirical models (next section) and is linked to the acceptance criteria that the carbonation front (or the pH 9-10 front) does not reach the reinforcement bars.

4.4.8 Model approaches

A recent overview of different types of models for carbonation of concrete is given in Ekolu [143].

4.4.8.1 Empirical models

The most simple model, based on the assumption of a constant CO_2 diffusion rate and Fick's first law of diffusion, is the so-called square-root-time relation:

$$x_c = k\sqrt{t} \tag{3}$$

where x_c is the carbonation depth [L], t is time [T], and k is the carbonation coefficient [L $T^{\wedge 0.5}$]. k accounts for different environmental factors and concrete properties that is determined by the ratio between CO_2 (CO_2 boundary concentration and CO_2 diffusivity influenced by microstructural properties, temperature and relative humidity) and the amount of carbonatable material [89]. Typically, the value of k needs to be calibrated for given concrete types and environmental conditions. Ekolu [143] detailed the factors that influence k as:

$$k = e_{\mathrm{h}} e_{\mathrm{s}} e_{\mathrm{c}} \mathrm{cem}(F) \tag{4}$$

where e_h is a function accounting for the effect of relative humidity, e_s is a function accounting for sheltered or unsheltered (outdoor) conditions, e_c is a function accounting for varying CO_2 concentrations and cem(F) is a time-dependent strength growth function and a factor for cement type. Based on an extensive dataset, parameters for the different functions were fitted. A simpler variant was proposed by [144], where statistical relations between carbonation coefficients for a relative humidity smaller or larger than 70% were predicted using different environmental conditions and/or cement properties. Another example is given in Nilsson [145]. Ta *et al.* [146] presented a short literature review on semi-empirical models that accounted for some internal and external factors and proposed a metamodel that accounts for both internal and external factors. Alternatively, artificial neural networks can be used as well to predict either carbonation depth or carbonation coefficients (e.g. [147, 148]). Another variant is the model defined in the fib-Model Code 2010[5] accounting also for environmental, curing, CO_2 content and carbonation resistance of the concrete.

[5] fib-Model Code, Model Code 2010, vol. 2, International Federation for Structural Concrete (fib), Lausanne, Switzerland (2010)

Assuming such square-root models for carbonation rates and progression has a fast and direct link with the performance indicator of depth of carbonation. Beside the limitation that it is strongly empirical approach, the assumption of constant boundary conditions and material properties puts limit on its applicability if service life over long-time scale has to be estimated under changing environmental conditions.

4.4.8.2 Phenomenological and complex models

Instead of using the root-square time type of models, a large number of models based on conservation laws combined with chemical reactions has been proposed to numerically solve the carbonation problem. These models capture typically the most important processes, i.e.:

- Diffusion of CO_2 in the porous material (mostly only in the gaseous phase).
- A geochemical reaction that leads to dissolution of portlandite and possible other phases and the precipitation of a calcium carbonate

However, these models differ in the way of the details of the geochemical reaction network, and of additional processes. Additional processes that are taken into account are (e.g., [149-158]) :

- Heat production and transfer (e.g. [150, 151])
- Moisture production and transfer (e.g. [149-151])
- Diffusion of Ca-ions
- Advective transport of CO_2 (e.g. [152])
- Change in transport properties (e.g. [152])
- Multicomponent or multispecies reactive transport based on thermodynamic equilibrium (e.g. [159, 160])

Models that account for changing boundary conditions and effects of environmental variables on transport parameters (e.g. diffusion coefficients) give a more flexible way to evaluate the risk for carbonation-induced damage, including assessing the effects of climate change as was illustrated in [103, 161-163].

4.5 Akali-aggregate reactions

4.5.1 Process Definition

Aggregates are more or less chemically inert in most concrete compositions. However, some aggregates react with the alkalis present in the concrete pore solution, causing expansion and cracking over a period of many years. Alkali-aggregate reaction (AAR) has two forms: alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR).

The alkali-aggregate reaction is a chemical reaction between amorphous silica or poorly-crystallized silicates present in certain types of aggregates and alkali components that are dissolved in the concrete pore water solution usually present in the from cement clinkers. The occurrence of this type of reaction is conditional upon the simultaneous presence of three conditions: potentially reactive aggregates, sufficient moisture, and a high concentration of alkalis present in the pore solution [164]. Grattan-Bellew and Mitchell [165] explains that the phenomenon occurs when some minerals present in aggregates react chemically with alkali (Na, K), thus forming a hygroscopic gel. Hasparik [166] points out that the consequences of AAR in concrete include expansion due to the water absorption of the hygroscopic gel, cracking, and negative effects on the concrete properties: reduction in the modulus of elasticity due to the cracking produced by expansion, reduction in tensile, and compression strengths. Alkali-carbonate reactions are observed mainly with aggregates composed of clayey dolomite as well as fine-grained limestones with inclusions of clay minerals. Sanchez [167] states that AAR can decrease the load bearing capacity of a structure or its structural concrete elements.

It affects also its functionality and durability, and thus affecting its service life. However, the degree of structural degradation generally depends on factors such as the AAR expansion level and the reinforcement layout.

References: For further references, the reader can consult [166-170]

4.5.2 Influential factors

Three factors are essential for the development of the alkali reaction: alkalinity (high pH), reactive silica and water content. As a result, the main parameters playing a role on the AAR are:

- Aggregates (reactivity, size and pessimum effect, detailed in section 4.5.2.2)
- The composition of the concrete (alkali content, porosity of the paste, mineral additions)
- The environmental conditions (temperature, humidity)

Reference: In this section, we will briefly detail the influence of the environmental factors on the development of this pathology ([168] to [171]).

4.5.2.1 General conditions

Effect of moisture content and relative humidity – Water is an essential ingredient for the alkali-aggregate reaction because it is both the transport medium for ionic species, the reaction medium and is part of the reaction products. The relative humidity inside a concrete is very variable depending on the value of the initial water/cement ratio and the external water conditions to which it is subjected. AAR forms at a relative humidity of specimens between 80% and 100% [168]. There is a strong correlation between the amplitude of the swelling and the concrete's water content [169]. Poyet *et al.* [169] shows that below a threshold of approximately 70% relative humidity, expansion is negligible. Moreover, the final swelling generated by AAR seems less important at saturated conditions (100% RH) than at 96% relative humidity. This could be due to the leaching of alkalis which are then no longer available for AAR.

At the structural scale, expansions vary spatially: important swelling occurs a wet area while little or no swelling is observed in drier areas. Below a relative humidity of 70-80%, the reaction is unlikely to occur. In their work, Louarn and Larive [170] also defines a threshold value in terms of water loss in concrete below which the reaction stops. If the concrete loses more than 0.15% by mass (i.e., more than 15 g of water per kg of concrete) then the chemical reaction stopped but it may resume with subsequent water ingress. This value should be adopted with caution because it greatly depends on the composition of the concrete.

Effect of temperature – Temperature influences the kinetics and magnitude of swelling. Its impact on kinetics is well established. Temperature activation is used to accelerate the reaction in laboratory. An increase in temperature results in an acceleration of the overall reaction kinetics. The rise in temperature increases the solubility of the silica, thus activating the reaction. However, several authors observe different swellings as a function of the storage temperature of the specimens: Louarn and Larive [170] found comparable final swelling whatever the temperature (20, 38 or 60°C), whereas Diamond [172] found that long-term expansion is lower at 40°C than at room temperature (20°C). As a result, thermal activation of AAR in the laboratory would therefore not necessarily lead to the same final swelling as those on structures [170].

Effect of irradiation - The synergetic effects between irradiation and ASR are still poorly understood. What is clearly identified is that neutron irradiation increases the reactivity of aggregates, i.e., the first stage of irradiated-assisted alkali-silica reaction (IAASR) formation [173, 174] (negligible effect on reactivity of carbonate aggregates [175]). It could increase the reactivity of alkali-silica reaction (ASR)-susceptible aggregates potentially used for radiation shielding concrete [176]. The stability of hydrophilic ASR-gel under gamma-induced radiolysis is not established. Radiolysis-induced drying may reduce the likelihood of ongoing AAR in absence of another moisture source.

Effect of microbiological reactions

Effect of exposure condition (free, sheltered, buried, submerged) Strongly related to the moisture condition, see above.

Effect of external water composition (submerged or buried structures) Strongly related to the moisture condition, see above.

Effect of atmospheric conditions (rain, wind, freeze/thaw) –

Effect of mechanical and structural loading - ASR expansion in concrete is influenced by the stress state of the material. Dunant and Scrivener [177] showed, based on studies on concrete developing AAR and subject to uniaxial loading, that load influences the micro-crack propagation, which changes the shape of the expansion curve. The expansion is not redistributed, but the applied load forces the orientation of the micro-cracks at the micro-structural level [177]. Under tri-axial loading, Liaudat *et al.* [178] showed that the volumetric ASR expansion rate is reduced as the applied volumetric compressive stress is increased. There seems to be also an increase of the expansion rate in the less compressed direction in detriment of the expansion rates in the most compressed ones [178].
However, in previous works in which measurements were performed on concrete specimens subjected to several states of stresses along the three directions (due to applied stresses and to passive restraint), Multon and Toutlemonde [179] showed that the volumetric expansion imposed by ASR is constant whatever the stresses conditions and that an “expansion transfer” occurs along the directions which are less compressed.

4.5.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM)
There is a general consensus that the use of SCMs has a positive effect to control the expansion and ASR-induced damage when mixed with highly reactive aggregates [180]. Some SCMs contribute to inhibiting the alkali-aggregate reaction and reducing the expansion. Siliceous additions (silica fumes, fly ash, etc.) and pozzolanic additions modify the ionic balance of the interstitial solution by consuming portlandite and alkalis which reduces the concentration of hydroxyl ions and limits the alkali-aggregate reaction [181]. The extent of its effectiveness depends on the amount and type of SCM, the alkali content and the CaO/SiO_2 ratio of the SCM, reactivity of the aggregate, quantity of alkalis supplied by the Portland cement, and the presence of aluminium species in the pore solution.

Moreover, with certain reactive aggregates, the addition of lithium salts allows the lithium ions to react preferentially with the reactive silica to form a non-expanding crystalline product [181].

Alkali content

The equivalent content of alkali is an important parameter with respect to the swelling rate and amplitude. The higher the alkalis content in the pore solution, the higher the concentration of hydroxyl ions, and the higher the kinetics and amplitude of the reaction [182]. The main source of alkali is from the cement (typically reported as Na_2O and K_2O in the clinker composition). In addition, certain aggregate-forming minerals such as clays, feldspars or even micas can potentially contain alkalis [183]. The leaching of alkalis from concrete is an important phenomenon to take into account. Indeed, an outward diffusion of alkalis creates an important gradient of alkali content over relatively small depths (a few cm) [184, 185].

Cement-water binder ratio

Porosity

Porosity plays an important role in the development of the reaction. On one hand, it controls the diffusion and transport of chemical species in the interstitial solution, including alkalis, and on the other hand, it plays a role in the expansion volume thus reducing the pressure caused by the gel formation. These two mechanisms are competitive. Low porosity (directly related to a low water-cement ratio) reduces the diffusion and transport of the reactants which will delay the onset of the reaction and results in better initial mechanical performance. Conversely, low porosity will lead to higher gel pressure causing greater swelling.

Concrete composition (aggregate content, types of aggregates)

Type of aggregates

Aggregates have a significant influence on the kinetics and magnitude of swelling generated by the reaction. The reactions differ depending on the type of siliceous aggregate. A classification vis-à-vis the alkali-reaction makes it possible to qualify the aggregates as: non-reactive, potentially reactive (PR), potentially reactive with pessimum effect (PRP) [detailed below] [186]. The reactivity of aggregates depends on the type and quantity of reactive silica contained therein and on their texture. It should also be emphasized that the more chemically disorganized and unstable the structure of the minerals making up an aggregate, the more reactive the aggregate is likely to be. As stated by Godart *et al.* [186], reactive minerals that may be present in aggregates include opal, cristobalite, tridymite, silicates, and intermediate volcanic glass, chert, glassy cryptocrystalline volcanic rock, artificial glasses, some argilites, phyllites, schists, gneisses, gneissic granites, vein quartz, quartzite, impure sandstone and limestone, and chalcedony.

Texture of aggregates

The texture of an aggregate is, for its part, linked to its granularity, its microporosity and its micro-cracking. A fine texture (aphanitic) increases the potential reactive surface area. The pre-existing integrity of the aggregate also plays an important role: microcracking and strained silicates increase the dissolution reaction [187].

Influence of aggregate size – Pessimum effect

The size of the reactive aggregates has an important influence on the reaction. A particular reactive grain size (diameter) may result in greater swelling than other sizes [188].This phenomenon is called the pessimum effect in aggregate size. It also depends on the type of silica in question. For siliceous aggregates such as opal and recycled glass aggregates, it appears that the expansion increases as the diameter of the aggregates decreases [172, 189]. However, for a very small aggregate size (diameter <0.15 mm) and reactive fines, the expansion is much less or even non-existent [190].

However, for siliceous limestone aggregates, the expansion obtained increases with the diameter of the reactive aggregates. Thus, Poyet *et al.* [169] showed a swelling inversely proportional to the specific surface of the aggregates and Multon [11] showed that the size effect is linked to the alkali concentration and the duration of the experiment. Likewise, Moundoungoun [191] demonstrated a pessimum effect for siliceous limestone aggregates by adding, via chippings, reactive free silica to concrete based on reactive sands. This effect, for aggregates of the same mineralogy, can be attributed to several reasons:

1- Small aggregates have a high specific surface area relative to their volume. The chemical species in the pore solution diffuse rapidly into the aggregate and accelerate the dissolution of the silica. As a result, the kinetics of alkali consumption are larger when the aggregate size is smaller, which causes the alkali concentration of the solution to drop rapidly, subsequently reducing the overall swelling [169] .

2- The interfacial transition zone (ITZ) around each aggregate is of approximately constant thickness, and of greater porosity than the rest of the cement paste [185]. This volume acts as an expansion vessel for gels produced near the grain surface. If the aggregate is small, then it is most likely that the volume of the halo is relatively larger than that of the aggregate. As a result, more gel can migrate into the connected porosity for a small size aggregate than for a larger one thus reducing swelling [188].

3- An effect linked to the cracking of the cement paste which promotes microcracking around large grains. The resulting gel will only cause a small increase in pressure due to the additional space created by the cracks opening [188].

Curing (temperature, moisture, time) conditions

4.5.3 Influence of other ageing processes

The question of the concomitance of AAR with other pathologies in reinforced concrete is poorly studied to date to the exception of combined AAR and DEF (Delayed Ettringite Formation).

4.5.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes)

Leaching (submerged or buried structures)

Carbonation

Ettringite and thaumasite reactions (DEF/sulphate attack) The use of reactive aggregates in the construction of a structure whose thermal history has exceeded 65 ° C during young age is plausible. As a result, DEF and AAR can occur concomitantly in certain structures, which is confirmed by several researchers [171, 192, 193]. Coupling mechanisms between the two reactions have been identified from chemical and structural perspectives. From a chemical point of view, the stability of ettringite relies partly on the presence of alkalis. The decrease in pH, caused by the leaching of alkalis by diffusion outside the concrete or by auxiliary phenomena such as the creation of the AAR gel, promotes the desorption of the sulphates trapped in the C-S-H. This phenomenon accelerates the delayed ettringite formation [171]. The alkali-aggregate reaction can therefore initiate the internal sulphate reaction. From a mechanical point of view, the creation of the AAR gel weakens the concrete by creating micro-cracks. This increases the water ingress which accelerates the onset of DEF. In his work, Martin [194] carried out experiments on concrete formulations subject to both pathologies. The onset of expansion occurs earlier and the rate of expansion are higher in specimens combining DEF and AAR when cured at 38 °C. However, the long-term swelling amplitudes are comparable to those of DEF in laboratory experiments. Other works in the literature suggest the hypothesis that the thermo-activation of AAR during thermal cure at an early age may increase the effect the volume of gel created [193].

Bacterial processes

4.5.3.2 Physical-Mechanical

Freeze-thaw

Elevated and high temperature (<150°C, no fire)

Irradiation The was discussed in the part irradiation in section 4.5.2

4.5.3.3 Mechanical

Presence of (surface) cracks

Abrasion/Erosion/Cavitation

Creep and relaxation

Settlements and movements

Vibration (and seismic)

Thermal stresses (gradients)

4.5.3.4 Electro-chemical

Pitting corrosion

General corrosion

Other corrosion mechanisms including crevice corrosion

4.5.4 Rates of deterioration

Jensen [195] proposed a new classification of all alkali-aggregate reactions according to the rate of damage: very fast, fast and slow. Very rapid AAR corresponds to damage in a few days in the laboratory and a few months in-situ. It includes rocks containing microscopic inclusions of opal, quartz, chalcedony or volcanic silica glass. The rapid AAR (damage observed from at least 1 year in-situ) groups together minerals such as opal, chalcedony, in various types of rocks. Slow AAR (damage observed from at least 10 years in-situ) contains minerals such as micro-crystalline quartz, recrystallized quartz in various rocks.

4.5.5 Impact on concrete properties

The effects of AAR on concrete and structures start at the microstructural level, that is, at molecular and microscopic levels, such as the formation of reaction products in concrete and associated microcracking, which ultimately leads to macro-level effects such as visual cracking, differential deformations, and displacement. The consequences of these effects vary considerably, and depend on the severity of the reaction, the configuration and functions of the affected element and the exposure conditions.

The formation of alkali–silica gel can have detrimental effects on concrete such as the initiation and propagation of cracks in the paste matrix, exudation of the gel from the cracks, volume expansion, stress development in the concrete, and degradation of the mechanical properties of concrete. The extent of damage caused by the reaction depends on several factors including the type of aggregate and its characteristics, the rate of formation of the gel, its volume concentration within the concrete, as well as the incorporation of SCMs as discussed previously.

Chemical – Pore water chemistry Around the reactive aggregates pore water chemistry will be modified due to formation of alkali-silica gel.

Chemical – Solid phases composition

Structural – Microstructure

Structural – Cracking The expansion of the gel from ASR generates pressure within the concrete and produces microcracking when the tensile strength of the concrete constituents is exceeded. Water may ingress into the concrete through the cracks and cause additional gel expansion, which may increase cracking and possibly lead to spalling. The continued development of cracks due to ASR undermines the structural integrity of the concrete. In extreme situations, the expansion due to ASR may lead to the yielding of the steel reinforcement and limit the performance of the structure [173, 176].

Transport properties – Porosity, permeability, diffusion coefficients The continued development of cracks due to AAR impact the transport properties of concrete hence leading to an increase in permeability and diffusion coefficients. Porosity for instance may decrease due to the AAR gel formation. Specifically, the cracks provide a pathway to aggressive chemicals toward the embedded steel reinforcement. This can lead to corrosion and strength loss of the concrete elements.

Mechanical properties The development of AAR leads to a decrease in the compressive strength, tensile strength and in the modulus of elasticity of concrete [169, 172].

4.5.6 Assessment methods

4.5.6.1 Visual inspection

The visual inspection of structures affected with AAR consists of mapping cracks apparent to the visual eye and measuring their width. Commonly used methods such as the LPC method n˚44, provide a cracking index which represents an average crack opening (in mm) per square meter of concrete surface [196]. Visual inspection also includes searching for signs of distress (large deformation impairing gate, efflorescence, pattern cracking)

4.5.6.2 Continuous monitoring

Some nuclear structures may be equipped with built-in extensometers, initially installed to monitor creep and shrinkage. Such devices allow the measurement of the local deformation in the concrete which can provide a continuous monitoring of the eventual swelling generated by the development of AAR.

4.5.6.3 Destructive testing of sampling

Extracting cores from structures affected with AAR serves three purposes:

- First, performing microscopic observation of the concrete; hence identifying the presence of AAR gel.
- Second, performance residual swelling tests (cf. see section 4.5.7).
- Third, measuring the mechanical properties (compressive strength, tensile strength…) of the concrete to assess the integrity of the structure.

4.5.6.4 Non-destructive techniques

Acoustic emission (AE) is the term used to define transient stress waves emitted from the sudden release of energy due to a deformation in the monitored material, such as crack formation or growth (ASTM E1316) [29] [197]. As a passive piezoelectric sensing technique, which does not need excitation or human intervention after the sensors are connected to the data acquisition system, AE is suitable for real-time long-term monitoring of structures. AE sensors are very sensitive (in the kHz), which enables them to detect active cracks long before they are visible (microcracks). However, concrete is susceptible to different material degradation mechanisms due to its heterogeneous nature, including corrosion of reinforcement, sulphate attack, alkali-aggregate reaction, freeze thaw cycling, leaching, radiation, elevated temperatures, salt crystallization, and microbiological attack. Additionally, under service loading conditions, concrete structures are usually designed as cracked sections due to the low tensile strength of concrete. Most of the aforementioned degradation mechanisms, such as corrosion and AAR, take years to progress and are associated with the formation of microcracks. The sensitivity of AE can be used to monitor the progression of these degradation mechanisms that will create AE activity with low amplitude and level, referred to here as low-level AE [197].

4.5.7 Performance indicators & acceptance criteria

Among the performance indicators that allow to assess the severity of a structure affected with AAR we note the accelerated test methods. Those tests provide indicators as to the reactivity of the aggregates and the potential final swelling and degradation of the concrete. Protocols for accelerating the alkali-aggregate reaction are standardized in some countries (such as France, Canada, the United States, Switzerland, etc.). These test methods apply to concretes and mortars and can be classified into four categories: ultra-accelerated and moderately accelerated tests on mortar bars, performance tests on concrete and residual swelling tests on concrete extracted by coring. Mortar bar tests do not exactly reproduce the behaviour of concrete, however they are a good indication of its potential reactivity and allow a rapid response as to the reactivity of a given aggregate. The performance tests on concrete and the residual swelling tests complete the tests on mortar and make it possible to judge the reactivity of a given concrete with more reliability. The mechanisms brought into play by these test methods in order to accelerate the alkali-aggregate reaction are relatively simple: a high storage temperature coupled with a humid environment (RH> 95%). However, the laboratory specimens deform freely and do not reproduce the actual conditions of the structure in terms of the effect of stresses or the effect of humidity gradients.

4.5.8 Model approaches

Modelling is critical to assess the remaining life of structures affected by AAR. Two modelling scales are distinguished: the mesoscopic scale (either at the scale of an aggregate or at the scale of several aggregates) and the scale of the structure.

4.5.8.1 Chemistry and transport models

The purpose of chemistry and transport models is to predict the amount of gel produced at a given reactive site depending on the availability of reactive chemical species in aggregates (silica), in concrete (alkali ions, hydroxides, calcium, water) and migrating from the outside (alkalis and water). Three models are mainly used: the Uomoto-Furusawa-Ohga’s model [198], Bazant and Steffens’ model [199] and Poyet’s model [196].

4.5.8.2 Micro-mechanical models

For a volume of gel created around a reactive site, mechanical models at the microscopic scale allow to determine the deformations and the variation of the mechanical properties. The approaches are elastic and take into account an evolution of the microstructure by damage or cracking. Micromechanical models can be classified into four families:

- Poromechanical elastic models: Dormieux *et al.* [200], Lemarchand *et al.* [201], and Esposito and Hendriks [202],
- models coupling chemistry and mechanics: the theoretical model of Suwito *et al.* [203],
- models with damage: Nielsen *et al.* [204], Qian *et al.* [205] with a Mohr-Coulomb criterion and Multon *et al.* [206], called LMDC's model [206] (Laboratory of Materials and Durability of Constructions of Toulouse),
- models with cracking: Ichikawa and Miura [207], Sanchez *et al.* [208] probabilistic model, Bazant *et al.* [209]], the poromechanical model of Charpin [210], and the lattice model of Copuroglu and Schlangen [211].

The model of Charpin [210] fits into the last category with the development of a poromechanical model based on microscopic damage capable of studying anisotropy considering a class of aggregate that is subject to rapid reaction rates.

4.5.8.3 Mechanical models at the mesoscopic scale

Mesoscopic models describe the mechanism of mechanical deterioration of a concrete affected by AAR at the material scale. The models address the interaction between the gel and the mortar matrix. The mesoscopic approach takes into account the particle size distribution and the location of reactive aggregates. Models are generally implemented in finite element codes. We can cite the works of Comby-Peyrot *et al.* [212], Naar [213], and Dunant and Scrivener [214].

To describe the degradation mechanisms, mesoscopic models consider multiple mineral phases in aggregates, the cement paste and the AAR gel, so that the material anisotropy can be explicitly represented in concrete. Swelling is simulated either by computing the equivalent expansion of aggregate particles [212] or by computing the expansion of distributed gels randomly in the aggregates [214].

In particular, the discrete model of Alnaggar *et al.* [215], based on a network of discrete particles and the model of Shin [216] [49], based on real microstructures (SEM) to generate numerically an equivalent finite element mesh, are developed on a mesoscopic scale.

4.5.8.4 Mechanical models at the macroscopic/structure scale

Concrete structures are typically in service for decades, so it is important to quantify the time during which the mechanical behaviour of structures affected by the AAR remains satisfactory. Macroscopic scale models allow the evaluation of this long-term behaviour. Mechanical and chemical mechanisms at the aggregate scale are taken into account using homogenized or phenomenological laws including complex phenomena such as the plasticity, viscosity, creep, shrinkage, cracking, thermal, etc.

Several macroscopic models have recently been developed to analyse the behaviour of the concretes affected by the AAR. Most models are formulated within the framework of the finite element method and couple the hydric, chemical and mechanical phenomena [217-230].

4.6 Delayed ettringite formation

4.6.1 Process Definition

The delayed ettringite formation (DEF) is an endogenous pathology of slow kinetics which affects certain concrete structures. The sulphates normally originate in cementitious materials with high sulphate contents. DEF has been linked with elevated temperatures of fresh concrete, either due to an internal cause such as hydration-induced heating of cement in mass concrete, or to an external cause such as steam-curing, a method often used by the precast

industry. Ettringite is normally one of the earliest phases formed during cement hydration. However, when the concrete temperature is higher than approximately 65 °C, ettringite is unstable and the formation of primary ettringite is halted. The constituents of ettringite are dispersed as monosulphate and calcium silicate hydrates. These constituents form secondary ettringite in already hardened concrete when thermodynamic conditions become favourable, i.e., at relatively lower temperature with available moisture, to result in expansion of the cement phases causing micro then macro-cracking of the concrete.
The elevation in temperature is a necessary parameter for the onset of this pathology, which causes the destabilization of the primary ettringite. Following this high temperature destabilization, it is the neoformation of ettringite after cooling that is responsible for the development of structural disorders. This cracking has an irregular appearance similar to map cracking caused by AAR. As a result, DEF and AAR may be confused with each other and misdiagnosed during field inspection. Petrographic analysis is generally required to distinguish these two types of chemical-reaction induced cracking.

References: For further references, the reader can consult [231-235]

4.6.2 Influential factors

Reference:

4.6.2.1 General conditions

Effect of moisture content and relative humidity – Relative humidity plays a key role in triggering this degradation mechanism. Water has two distinct and essential functions in the process of ettringite formation. On one hand, it is a necessary reactant for the formation of ettringite because water is directly involved in the composition of ettringite since 32 molecules of water are present in one molecule of ettringite. On the other hand, water is also the reaction medium, being the means of transporting ionic species to the ettringite precipitation sites. Thus, a minimum relative humidity is necessary for the development of this pathology. This value is 92% according to Graf-Noriega [236], 95% according to Martin [194] and 98% according to Al Shamaa *et al.* [237]. Exceeding this threshold, even later on in the structures service life, can trigger the DEF reactions.

Effect of temperature - Temperature influences the kinetics and amplitude of swelling. Flatt and Scherer [238] and Baghdadi [239] showed that a high curing temperature decreases the final amplitude of expansions and increases the kinetics of the reaction. In the same perspective, Pavoine [240] concluded that a storage temperature of 23°C generates a higher swelling amplitude than a temperature of 38°C. As a matter of fact, the solubility constant of ettringite depends directly on temperature: at low temperatures, the precipitation is facilitated but the kinetics of chemical interactions are reduced.

Effect of irradiation - While there is currently no evidence of combined DEF and irradiation, some concrete biological shields include a liner retaining the moisture content at a high level. These structures being relatively massive (~1.5 m in depth), the possibility of DEF pre-exiting irradiation effects cannot be ruled out at this stage.

Effect of microbiological reactions Not relevant

Effect of exposure condition (free, sheltered, buried, submerged)

Effect of external water composition (submerged or buried structures)

Effect of atmospheric conditions (rain, wind, freeze/thaw) –

Effect of mechanical and structural loading - There are many indications that the stress state of a material prone to DEF will impact the development of swelling. As a matter of fact, the development of microcracks during mechanical loading or the variation of the solubility of the ettringite crystal with the surrounding pressure according to thermodynamic principles clearly show the existence of a correlation between stress and expansion. In the case of reinforced concrete, the presence of reinforcements in a material can modify the homogeneous spatial swelling of the paste [241]. The stress field created by the reaction of the reinforcements to the swelling is likely to "orient" the directions of crystallization of ettringite. Anisotropic expansions can therefore be expected. Unfortunately, insufficient research work to date seems to be available in the open literature. According to several expert reports, the cracking pattern developed would depend on the direction of the pre-stressing or the reinforcement layout. The development of degradation could therefore depend on of the applied stress. Recently, Bouzabata *et al.* [242] investigated the effect of mechanical loading on DEF swellings. Their first conclusion is that DEF-induced swellings are isotropic under free swelling conditions but become anisotropic as soon as a loading is applied. The uniaxial compressive stress generates a reduction in deformations in the loaded direction and leads to the development of a cracking facies parallel to the stress lines. As the compressive stress increases, the developed expansions decrease in the loading direction.

4.6.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) There is a general consensus that the use of SCMs is one approach to control the expansion and consequent damage as a result of deleterious reactive aggregates. The use of certain mineral additions, such as blast furnace slags, fly ash, natural pozzolans, metakaolin and silica fumes, can improve the resistance of concrete against DEF. Several studies have shown that the introduction of these mineral additions in an appropriate amount in the concrete mix, replacing cement, reduces, or even inhibits, the expansion caused by DEF [243-245]. Al Shamaa *et al.* [237] have shown that the addition of limestone fillers influences the kinetics and amplitude of swelling due to DEF by impacting water movements in the studied mortars. However, these additions do not inhibit the pathology. Mineral additions affect the microstructure development of the cement paste. Densification limits the ionic transfers and tends to reduce the expansions linked to DEF.

Initial content of sulphates and aluminates Given the composition of ettringite, the initial content of SO_3 sulphates and Al_2O_3 aluminates greatly influences the delayed formation of ettringite. Sulphates are mainly found in hydration controlling agents such as gypsum. Aluminates are present in the unhydrated cement phases C_3A or C_4AF. The aluminate content is often associated with that of the sulphates in the form of the SO_3/Al_2O_3 molar ratio. Heinz *et al.* [232] found that a maximum value of 0.67 for the SO_3/Al_2O_3 ratio would prevent swelling, while Zhang *et al.* [246] estimated that the ratio should be close to 1. However, these conclusions are based on tests carried out at high temperatures and different heating durations.

Influence of alkali content The content of equivalent alkali is an important parameter with respect to swelling. Alkalis affect the stability of ettringite by varying its solubility. A high concentration of alkalis during the hydration phase promotes the dissolution of primary ettringite via the threshold temperature and the adsorption of sulphates on the C-S-H. Subsequently, alkalis leaching promotes the precipitation of ettringite [247-250].

Cement-water binder ratio

Porosity of the cement paste Porosity plays an important role in the development of the reaction: On one hand, it controls the diffusion and transport of chemical species in the interstitial solution, and on the other hand it plays the role of “expansion vessel”, thus reducing the stresses caused by the delayed ettringite. Two phenomena thus come into competition. A low porosity (directly linked to a low water-cement ratio) makes it possible to limit the diffusion and transport of the reactants which will delay the onset of the reaction, but, on the other hand, this porosity cannot play its role of expansion vessel which will lead to greater swelling. This parameter is directly influenced by the water to cement (w/c) ratio. According to Brunetaud *et al.* [251] and Taylor *et al.* [252], a decrease in the W/C ratio leads to greater swelling amplitudes. This is linked to the reduction in the porosity of the cement matrix; thus, the precipitation of a small amount of ettringite can generate crystallization pressures of greater magnitude. On the other hand, for a low W/C ratio, the expansion kinetics are slowed down because of the slowing down of ionic transfers in a porous medium.

Concrete composition (aggregate content, types of aggregates) Aggregates have a significant influence on the kinetics and magnitude of swelling generated by the reaction. The work of Yang *et al.* [253] has shown that the expansions attributed to DEF in concrete mixes using limestone aggregates are reduced compared to siliceous aggregates. Calcareous aggregates develop epitaxy properties (formation of hydrated calcium carbo-aluminates) with the cement matrix leading to a much ITZ than with siliceous aggregates and leading as well to the local reduction of the concentration of aluminates. The ITZ is therefore more porous and more fragile when using siliceous aggregates. This factor accelerates the diffusion of ionic species [251, 254]. In addition, ettringite migrates to precipitate in cracks at the cement matrix/aggregate interfaces (Ostwald ripening phenomenon [255]). This explains why the DEF-induced swelling in concrete formulated with siliceous aggregates are greater than with limestone aggregates for the same cement and heat treatment [255].

Curing (temperature, moisture, time) conditions The effect of temperature, and moisture was discussed above.

4.6.3 Influence of other ageing processes

4.6.3.1 Chemical/biological processes

The question of the concurrence of DEF with other pathologies in reinforced concrete has been poorly studied to the exception of combined AAR and DEF.

Attack by aggressive water, acid-base attack (industrial processes)

Leaching (submerged or buried structures) Alkali leaching impacts the thermodynamic equilibrium of ettringite. In the long term, the decrease in the alkali content (and therefore, of the pH) promotes the release of sulphates previously trapped by the C-S-H [256]. Thus, samples stored in water are subject to more rapid swelling than those stored at 100% relative humidity. It is often difficult to separate the effect of humidity from the effect of leaching. We may hypothesize that the alkalis play a role during the heating phase (a higher alkali concentration enhances the dissolution of primary ettringite) and a role during the life of the material (a drop in the concentration after heating amplifies the delayed ettringite formation).

Alkali-aggregate reactions The use of reactive aggregates in the construction of a structure whose thermal history has exceeded 65°C at early-age is plausible. As a result, DEF and AAR can occur concomitantly in certain structures, which is confirmed by several researchers [239, 251, 257]. Both chemical and mechanical synergies between AAR and DEF have been identified. From a chemical point of view, the stability of ettringite depends partly on the presence of alkalis. A decrease in pH may e induced by the leaching of alkalis by diffusion outside the concrete or by auxiliary phenomena such as the creation of the AAR gel. This decreasing alkalinity promotes the desorption of the sulphates trapped in the C-S-H, which accelerates the delayed ettringite formation [251]. The alkali-aggregate reaction can therefore initiate the internal sulphate reaction. From a mechanical point of view, the creation of the AAR gel weakens the concrete by creating micro-cracks which accelerates the development of DEF enhanced by water ingress. In his work, Martin [194] carried out experiments on different concrete formulations subject to both pathologies. The onset of expansion and its kinetics are faster in specimens combining both DEF and AAR when the temperature is kept constant at 38°C. However, in the laboratory, the long-term swellings when both pathologies occur concomitantly are close to those of DEF. Other works in the literature advance the hypothesis that the thermo-activation of AAR during thermal cure at an early age will increase the volume of created gel [239].

Carbonation

Bacterial processes

4.6.3.2 Physical-Mechanical

Freeze-thaw

Elevated and high temperature (<150°C, no fire)

Irradiation

4.6.3.3 Mechanical

Presence of (surface) cracks

Abrasion/Erosion/Cavitation

Creep and relaxation

Settlements and movements

Vibration (and seismic)

Thermal stresses (gradients)

4.6.3.4 Electro-chemical

Pitting corrosion

General corrosion

Other corrosion mechanisms including crevice corrosion

4.6.4 Rates of deterioration

The rates of deterioration due to DEF may vary greatly (from a few years to a few decades) depending on the severity of the reaction, the early-age thermal history of the structure and the exposure conditions (conservation temperature, alkali-leaching, moisture and relative humidity).

4.6.5 Impact on concrete properties

The effects of DEF on concrete and structures start at the microstructural level, that is, at molecular and microscopic levels, such as the formation of reaction products in concrete and associated microcracking, which ultimately leads to effects observable at the structural scale, such as cracking, differential deformation, and displacement. The range of distress varies greatly, and depends on the severity of the reaction, the configuration and functions of the affected element and the meteorological exposure conditions (temperature, moisture content and relative humidity). The delayed formation of ettringite can have detrimental effects on concrete such as the initiation and propagation of cracks in the cementitious matrix, volume expansion, stress development in the concrete, and degradation of the mechanical properties.

Chemical – Pore water chemistry and solid phase composition The pore water chemistry is modified due to precipitation of ettringite. At the early age of concrete, C-S-H gel will adsorb sulphate rapidly at high temperature resulting in the quick depletion of the gypsum in the Portland cement-water system. Sulphate absorbed at high temperature is desorbed more slowly than that adsorbed at ambient temperatures. Slower release of sulphate from an internal sulphate source is a critical condition for DEF in high temperature cured Portland cement paste. Crystal nucleation requires less surface energy in a crack than in the cement paste matrix. Sulphate ions, after release from the C-S-H gel, will diffuse into the nearest microcrack and react with the Al-bearing materials in the crack to nucleate and crystallize ettringite. The growth of ettringite crystals tends to open pre-existing cracks wider and contributes to damaging the cementitious products [258, 259].

Structural – Microstructure

Structural – Cracking The delayed ettringite formation generates pressure within the concrete and produces microcracking when the tensile strength of the concrete's constituents is exceeded. Water may ingress into the concrete through the cracks and cause additional expansion, which may increase cracking and possibly lead to spalling. The continued development of cracks due to DEF may undermine the structural integrity of the concrete.

Transport properties – Porosity, permeability, diffusion coefficients The continued development of cracks due to DEF impact the transport properties of concrete hence, increasing the permeability and the diffusion coefficients.

Mechanical properties

4.6.6 Assessment methods

4.6.6.1 Visual inspection

The visual inspection of structures affected with DEF consists of mapping cracks apparent to the visual eye and measuring their width. Commonly used methods such as the LPC method n°44, provide a cracking index which represents an average crack opening (in mm) per square meter of concrete surface [84].

4.6.6.2 Continuous monitoring

Some nuclear structures may be equipped with built-in or embedded extensometers, initially installed to monitor creep and shrinkage. Such devices allow the measurement of the local deformation in the concrete which can provide a continuous monitoring of the eventual swelling generated by the development of DEF.

4.6.6.3 Destructive testing of sampling

Extracting cores from structures affected with DEF serves three purposes:

- First, performing microscopic observation of the concrete; hence identifying the presence of delayed ettringite formation.
- Second, performing residual swelling tests (cf. see section 4.6.7).

- Third, measuring the mechanical properties (compressive strength, tensile strength…) of the concrete in order to assess the integrity of the structure [260, 261].

4.6.6.4 Non-destructive techniques

Acoustic emission (AE) is the term used to define transient stress waves emitted from the sudden release of energy due to a deformation in the monitored material, such as crack formation or growth (ASTM E1316) [197]. As a passive piezoelectric sensing technique, which does not need excitation or human intervention after the sensors are connected to the data acquisition system, AE is suitable for real-time long-term monitoring of structures. AE sensors are very sensitive (in the kHz), which enables them to detect active cracks long before they are visible (microcracks).

However, concrete is susceptible to different material degradation mechanisms due to its heterogeneous nature, including: delayed ettringite formation, corrosion of reinforcement, sulfate attack, alkali-aggregate reaction, freeze thaw cycling, leaching, radiation, elevated temperatures, salt crystallization, and microbiological attack. Additionally, under service loading conditions, concrete structures are usually designed as cracked sections due to the low tensile strength of concrete. Most of the aforementioned degradation mechanisms, such as corrosion and AAR, take years to progress and are associated with the formation of microcracks. The sensitivity of AE can be used to monitor the progression of these degradation mechanisms that will create AE activity with low amplitude and level, referred to here as low-level AE [197].

4.6.7 Performance indicators & acceptance criteria

Among the performance indicators that allow to assess the severity of a structure affected with DEF we note the accelerated test methods. Those tests provide indicators as to the potential final swelling and degradation of the concrete [262, 263].

Standard protocols for accelerating DEF are accepted in several countries such as France, Canada, the United States, Switzerland. These test methods are applied either to concrete and mortar specimens. They can be classified into four categories: ultra-accelerated and moderately accelerated tests on mortar bars, performance tests on concrete and residual swelling tests on concrete extracted by coring. While mortar bar tests are not representative of actual concrete performance, they provide a good indication of the potential reactivity and make it possible to obtain a rapid assessment of the reactivity of any given cement. The performance tests on concrete and the residual swelling tests complete the tests on mortar and make it possible to judge the reactivity of a given concrete with increased reliability. The mechanisms brought into play by these test methods in order to accelerate the DEF are relatively simple: early-age heat treatment, a humid storage environment (RH> 95%), alkali-leaching and artificial micro-cracking (generated by drying/wetting cycles). However, the laboratory specimens deform freely and do not reproduce the actual structural conditions in terms of the effect of stresses or the effect of humidity gradients.

4.6.7.1 Model approaches

Following the various experimental studies carried out, the parameters influencing the formation of delayed ettringite have been identified and their effects have been quantified both on cement pastes, mortars and even concrete. Thus, models are proposed in order to either predict the final free expansion or to estimate the risk of occurrence of the DEF. In the literature, several families of models can be distinguished: empirical models, statistical models, thermodynamic models and kinetic models.

4.6.7.2 Empirical and statistical models

In general, these models propose a calculation of the final expansion of the material as a combination of the chemical parameters of the material. We can cite the works of Kelham [249] and Lawrence [264] based on mortars, and models of Zhao *et al.* [265] and Zhang *et al.* [246]. For example, the latter allows you to calculate a DEF index. When it is greater than 1.2, swelling would be observable, while if it is less than 0.8 no swelling is observed. All of these empirical models operate from the composition of the mortar and/or concrete and can be considered as a preventive tool against DEF. They make it possible to evaluate the reactivity of the formulation according to the composition of the cement. However, these models partially take into account several parameters that are necessary for the initiation of the reaction, such as the thermal history at young age or the environment in which the concrete is stored. They do not give any information on the kinetics of the reaction, which limits their practical application.

Brunetaud *et al.* [251] proposed a predictive model for the amplitude of swelling based on extensive experimental data. This technique makes it possible to prioritize the importance of the various parameters and to assign them an importance coefficient. More recently, Kchakech [266]proposes an equation making it possible to calculate an ettringite indicator which depends on the initial quantities of sulphates and aluminates, on the useful energy (a parameter which includes the maximum temperature reached, the threshold temperature of concrete and the duration of heating at a young age), the equivalent alkali concentration and two setting parameters. However, in order to determine the setting parameters, several tests at different temperature stages are necessary for a given concrete.

4.6.7.3 Thermodynamic models

Thermodynamic models were developed in the 1990s; we can cite for instance the models proposed by Glasser *et al.* [267]. These models make it possible to determine the amount of ettringite that can be formed in a given system, using as input data the solubility products of the different phases considered to be stable. From the results obtained, it is possible to estimate the potential for chemical swelling. However, this potential does not allow an evaluation of the real expansion potential of a concrete because they do not consider the microstructure or the exposure conditions. Moreover, these models cannot represent the kinetics of DEF expansions and do not take into account the determining role of C-S-H on equilibria. They do, however, provide a deterministic basis for the formulation of swelling models. Nevertheless, the modelling of the physical-chemical process of DEF requires the simultaneous consideration of the thermodynamic equilibria and the phenomena of transport. Thus, Salgues *et al.* [268] proposes to complete the thermodynamic models by integrating the kinetic aspects depending on the thermal damage of the material.

More recently, Sellier and Multon [269] propose a semi-empirical chemical model pertaining to the early age of concrete. This model estimates the amount of aluminium and silica ions available for DEF while distinguishing between AFt ettringite and ettringite as a consequence of DEF. The model takes into account the pessimum effect and the dominant effect of alkalis on the threshold temperature at an early age and on the dissolution kinetics of mono-sulfoaluminates, ettringite, and hydrogenate formation. Partially based on empirical relations, the use of this model requires a large number of experimental results for its calibration.

4.6.7.4 Kinetic models

In order to model the amplitudes and kinetics of swelling, Brunetaud *et al.* [251] proposes a model in the form of a sigmoid to describe the curves of swelling. This law expresses the progress of the swelling reaction as a function of three main parameters to which are added two setting parameters for the final slope when there is one:

$$\varepsilon(t) = \varepsilon_\infty \xi(t) = \varepsilon_\infty \frac{1-\exp\left(-\frac{t}{\tau_c}\right)}{1+\exp\left(-\frac{t-\tau_l}{\tau_c}\right)} \left(1+\frac{\Phi}{\delta+t}\right)$$

- ε_∞ is the amplitude of the final swelling,
- τ_L is the latency time which corresponds to the abscissa of the inflection point of the curve. This latency time characterizes the moment when the reaction accelerates
- τ_C the characteristic time related to the attenuation phase of expansive phenomena. It characterizes the time to reach the deformation plateau.
- Ø and δ are two parameters making it possible to take into account the linear evolution of the expansion in the phase of gradual attainment of a possible plateau. This phase corresponds to the stopping or slowing down of the expansions.

According to Salgues *et al.* [268], this model is in fact only a configurable swelling equation which allows to describe a swelling curve but not to predict it. This type of model can however be used to quantify observations, in particular thanks to the kinetics parameters τL and τ_C which make it possible to describe the swelling kinetics of concrete.

The model of Brunetaud *et al.* [251] was then improved by Kchakech [266]. The latter proposes a first modification which eliminates the overestimation of the final expansion and a second modification by adding a parameter which takes into account the water swelling at the beginning of the dimensional monitoring of the samples.

4.6.7.5 Mechanical models at the macroscopic/structure scale

Concrete structures are typically in service for decades, so it is important to quantify the time during which the mechanical behaviour of structures affected by the DEF remains satisfactory. Macroscopic scale models allow the evaluation of this long-term behaviour. Mechanical and chemical mechanisms at the aggregate scale are taken into account using homogenized or phenomenological laws including complex phenomena such as the plasticity, viscosity, creep, shrinkage, cracking, thermal, etc.

Several macroscopic models have recently been developed to analyse the behaviour of the concrete affected by the DEF. Most models are formulated within the framework of the finite element method and couple hydric and chemical phenomena. Two main approaches have been proposed:

• the mechanical approach (e.g. Baghdadi [194, 239, 266, 270])

• the poromechanical approach (cf. [268, 271, 272])

4.7 Bacterial processes

4.7.1 Process Definition

Reinforced concrete structures may be subject to rapid corrosion and degradation processes induced by heterotrophic and autotrophic bacteria[6] transforming C, S and N compounds [273]. The role of micro-organisms on the ageing of concrete itself can be grouped by [274-276][7]:

- physical deterioration by microbial growth or movement inside the material itself (biophysical) by mechanical processes , and
- chemical deterioration (biochemical) by (i) corrosive metabolites as acids that may react with hardened cement paste phases leading to leaching of calcium and other alkaline-binding materials and formation of secondary minerals (biomineralization) and (ii) the use of the material itself as a source for energy and/or elements for growth [276].

In addition, microbes may cause the corrosion of the reinforcement itself by a process of microbiological induced corrosion (MIC) [277][8].

Chemical deterioration depends on the metabolic pathway defined by aerobic or anaerobic conditions.

- Aerobic conditions – Production of biogenic organic acids by heterotrophic bacteria or autotrophic algae and cyanobacteria (getting energy from sunlight), for example in biofilms at the interface with ambient environment [278])
    - Bacteria and microscopic fungi may produce organic acids (acetic, lactic, butyric, oxalic) and carbon dioxide, all being detrimental to concrete [276, 279, 280]. Organic acids have found to be aggressive in concrete degradation (e.g., [281, 282]). The organic acids listed above are all weak acids forming anions at lower pHs than encountered in cement pore water (depending on their pK_a). Consequently, Ca^{2+} will form aqueous complexation with their conjugated bases (acetate, lactate, butyrate, oxalate) and thus increase the solubility of Ca over almost the complete pH range ($Ca(OH)^+$ becomes only the predominant Ca species when pH > 12.5 [283]). An example, the complexation of Ca^{2+} with acetate (the conjugated base of acetic acid CH_3-COOH) can form calcium acetate as:

    $$\mathrm{Ca}^{2+} + 2\mathrm{C_2H_3O_2^-} \Leftrightarrow \mathrm{Ca}\left(\mathrm{C_2H_3O_2}\right)_2 \tag{5}$$

    The overall reaction with portlandite can be written as:

    $$\mathrm{Ca(OH)_2} + 2\mathrm{CH_3COOH} \Leftrightarrow \mathrm{Ca}\left(\mathrm{C_2H_3O_2}\right)_2 + 2\mathrm{H_2O} \tag{6}$$

    Note that also biogenic CO_2 can be formed what may lead to carbonation (see section 4.4).
- Aerobic conditions – Production of inorganic acids by autotropic bacteria (getting energy from oxidizing inorganic electrons) with metabolism producing sulphuric or nitric acid
    - Sulphur oxidizing bacteria in environments rich in reduced sulphur forms (sulphides, elemental sulphur, thiosulphates, e.g., in sewer systems in which hydrogen sulphide is produced by sulphur reducing bacteria from sulphuric acid

[6] Autotropic bacteria obtain energy from oxidizing inorganic electron donors or from sunlight and C from inorganic C.
Heterotrophic bacteria obtain C from oxidizing organic carbon (by oxygen (aerobe) or other oxidized inorganic species (anaerobe)) and energy from the organic carbons.
[7] A third group is aesthetic deterioration due to fouling (formation of a biofilm), but this is considered to be less relevant for reinforced concrete in nuclear power plants.
[8] Note that, given the wide variety of microorganism, also MIC-inhibition (MICI) can occur [277] Kip, N., and J.A. van Veen, 2015. The dual role of microbes in corrosion. **The ISME Journal** 9:542-551.

and organics. However, these types of bacteria and consequent concrete deterioration have been observed on other civil structures such as bridges as well, e.g. [284], or (geothermal) cooling towers [285]), typically acidophilic bacteria as Thiobacilus but a succession of other communities can develop depending on the pH [275, 284, 286] and others. The sulphuric acid reacts with free lime/portlandite to form gypsum. Further reaction with calcium aluminates may produce ettringite or thaumasite [287]. These expanding phases may cause an increase in internal pressure and to the formation of cracks.
- Ammonium oxidizing bacteria (nitrifying bacteria) leading to the production of nitric acid by e.g. Nitrosomonas and Nitrobacter sp. Ageing of concrete is increased by forming soluble calcium nitrate with thus increase the solubility of Ca-bearing hydrated phases such as portlandite [276]:

$$\begin{aligned} &2NH_4^+ + 3O_2 \rightarrow 2NO_2^- + 2H_2O + 4H^+ \\ &2NO_2^- + O_2 \rightarrow 2NO_3^- \\ &Ca\left(OH\right)_2 + 2NO_3^- + 2H^+ \Leftrightarrow Ca\left(NO_3\right)_2 + 2H_2O \end{aligned} \tag{7}$$

- Anaerobic conditions (heterotrophic bacteria getting energy and C from oxidizing organic carbon and reducing oxidized inorganic species other than oxygen)
  - Sulphate reducing bacteria that produce hydrogen sulphide during dissimilatory reduction (anaerobic respiration) of sulphate. Hydrogen sulphide is an aggressive compound with respect to metals and may lead to direct interaction with the metals in the reinforced concrete (pitting corrosion) and formation of galvanic pairs on the metal surface [276]. The process in which biogenic dissolved hydrogen sulphide reacts with metallic iron is sometimes referred to as chemical microbiocally influenced corrosion [288, 289].
  - Fermentation reactions produce many organic acids as by-products; as the physiological diversity of fermenters is very wide, they may likely also be involved in concrete biodegradation processes [280] leading to e.g. a chemical zonation (zone with more or less intense to decalcification of the matrix) [290].

References: Main references were mentioned before, see also Turick and Berry [280], Gu *et al.* [291], Cwalina [292], Small and Abrahamsen-Mills [293], or Hoffmann *et al.* [294].

4.7.2 Influential factors

References: Influence of some environmental factors is discussed in Turick and Berry [280]

The detrimental consequences of microbiologically reactions are influenced by at least two processes: (i) the production of metabolites (e.g. H_2S, sulphuric acid, nitric acid etc.) and (ii) the transport within and the reactions with the cement matrix of metabolites. The influential factors for the second group are similar to those of ageing processes such as leaching or attack by acids and will not be repeated here. The focus here is mainly on the influence on microbiological growth, i.e. which conditions lead to the most optimal growth.

4.7.2.1 General conditions

Microorganisms can grow under very wide conditions including extremely harsh environments. The environmental conditions will determine the type of organisms that flourish and growth rates can be similar under different conditions [280].

Effect of moisture content and relative humidity – Microorganisms typically have an optimal range for metabolisms, with too dry conditions forcing microorganisms to dormancy. Many bacteria become dormant at relative humidity levels below 90 % [295], but activity is still likely down to 60 % [280]. It is unlikely that inside concrete structures, this low relative humidity will be reached under service conditions. The surface of buried concrete structures will also have a relative humidity larger than 90% as such a high relative humidity is also expected in soils and other unsaturated systems. On the other hand, concrete surfaces in environments with very low relative humidity will likely be less subject to microbiological-induced deterioration, although in environments with cycles of low and high humidity, microorganisms may become metabolically inactive during dry periods (e.g. via sporulation), but become active again under higher humidity.

Given that many processes in concrete are diffusion driven, the higher the moisture content or relative humidity, the faster the aqueous diffusion processes and thus the migration of organic or inorganic acids in the porous matrix, except for those that are present in the gaseous phase as well.

Effect of temperature Many microbiologically-mediated redox reactions are temperature dependent as enzyme activity depends on temperature (Arrhenius type of dependence). Enzymes will grow within a certain characteristic temperature range (for most bacteria, a temperature range of 30°C is applicable, [296]), and will have an optimal growth rate at a specific temperature. Similar to pH (see below) and other environmental factors, different communities can develop for temperatures below zero up to ~120°C [296], called psychrophiles for the cold environments and thermophiles or hyperthermophiles for hot environments. In the range of 10-45°C, it is expected that the activity of the communities as a whole will increase [280].

From a chemical-physical point of view, a temperature increase will affect diffusion (increasing diffusion) and will change solubility depending on the thermodynamic properties of the cement phases.

Effect of irradiation It is well known that radiosensitivity of microorganisms depends strongly on various intrinsic (e.g. physiological and genetic features, state of the microbial cells) and extrinsic (environmental, e.g., water content, temperature) factors (see [297], and references therein). To our best knowledge, no information was found on the effect of the irradiated concrete itself on the microbiological processes in cementitious materials.

Effect of exposure condition (free, sheltered, buried, submerged) Free and sheltered conditions are more likely to harbour microorganisms that use light as an energy source (phototrophs, e.g. fungi). If light is not available, either organic or inorganic substances are used as an energy source (chemotrophs). In buried conditions in soils for example, organic sources are the most likely C and energy sources, with oxygen or other oxidized inorganics serving as electron acceptors. For example:

- The sulphate reducing bacteria uses organic substance (electron donor, energy, C source) and sulphate (electron acceptor); an example of heterotrophic and chemotrophic reactions.

- The sulphate oxidizing bacteria uses inorganic carbon (C source), H_2S (electron donor, energy source); an example of autotrophic and chemotropic reactions.

Some other types of micro-organisms that may be present on concrete exposed to sunlight, or under buried or submerged conditions are listed in Turick and Berry [280].

Effect of external water composition (submerged or buried structures) The external water in contact with the concrete again has a large effect on what kind of microbial communities will be present. But the environment must provide a carbon source (for growth – organic or inorganic carbon) and an energy source (metabolism – beside light, organic carbon, some reduced inorganic species). Oxidation of the energy source also requires a terminal electron acceptor being oxygen (aerobic conditions) or nitrate, ferric iron, sulphate and so one

(anaerobic conditions). However, as microorganisms can be present in biofilms – matrix-enclosed bacterial populations adherent to each other and/or surfaces or interfaces [298] - on concrete structures that can contain different microbial communities such that the bioreactions might differ from what would be expected from the bulk solution [280].
Rates of bacterial (enzymatic) reactions are often described with a Monod type of kinetics using a dual Monod expression (e.g. [293, 299]):

$$R = Q_{\max} B \frac{[E_{\mathrm{D}}]}{K_{\mathrm{D}} + [E_{\mathrm{D}}]} \frac{[E_{\mathrm{A}}]}{K_{\mathrm{A}} + [E_{\mathrm{A}}]} IT \tag{8}$$

Where R is the rate [M/T], $Q_{\max}$ is the maximum rate [M/gBiomass/T] at a reference temperature, B is the biomass [gBiomass] the third term is the Monod factor for the electron donor with K_D the half-saturation constant for the donor [M/L^3], the fourth term is the Monod factor for the electron accepter with K_A the half-saturation constant for the acceptor [M/L^3], [E_D] and [E_A] are, respectively, the concentrations of the electron donor and acceptor [M/L^3], I is a term to represent e.g. inhibition effects caused by toxic species, pH, or to suppress reaction when higher energy-yielding terminal electron acceptors are still available; the latter can be introduced by a thermodynamic potential factor as in [300-302]. Equation (8) shows that the composition of the external water may influence the rate of ageing by biological processes by the concentration of electron donor, acceptors or other species.

Effect of atmospheric conditions (rain, wind, freeze/thaw) As microbes go into metabolic inactivity during environmental unfavourable conditions (too low relative humidity, outside their optimal temperature range) but re-activate when conditions become more favourable, the effect of the atmospheric conditions on the activity of the bacterial community as a whole is not too large. Indeed, in the study of Silva [303] of biodeterioration of concrete under three different climate conditions concluded that the climate is not a very important factor for the development of microorganisms on concrete but has an effect on the specific microorganisms that occur.

Effect of mechanical and structural loading Unknown if this is a relevant factor.

4.7.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) Not many studies were found that investigate the effect of supplementary cementitious materials on protection of concrete against the detrimental effects of microbiologically processes. Partial replacement of cement by BFS or SF was found to improve resistance [285, 304], although other studies showed the reverse. De Belie et al. [305] found the reverse, possible linked to a more rapid colonisation of slag cement (as observed in [306]), although no cause for this observation is given. Fisher *et al.* [307] found that the addition of silica fume lead to an increase in weight loss under the activity of sulphur oxidizing bacteria; they stated that SF reduce the structure of the cement paste because of a more limited extent of pozzolanic reactions or possible alkali-aggregate reactions.

Cement-water binder ratio No relevant information was found, probable not an aspect with respect to production of metabolites by a biofilm attached to the concrete surface. Some aspects will be similar as for the ageing by leaching or acid attack.

Concrete composition (aggregate content, types of aggregates) A study of De Belie et al. [305] (again for sewer pipes) indicated that limestone aggregates limit the degradation depth of biogenic sulphuric acid (similar to chemical tests) because of buffering by limestone. No other information was found. A low lime content would provide better resistance to acid attack since the use of lime-containing granules results in a homogeneous surface attack. With non-lime granules, only the cement stone is affected, resulting in a rough surface and gradually loosening granules.

Curing (temperature, moisture, time) conditions No relevant information was found, probable not an aspect with respect to production of metabolites by a biofilm attached to the concrete surface. Some aspects will be similar as for the ageing by leaching or acid attack.

4.7.3 Influence of other ageing processes

4.7.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) – Leaching (submerged or buried structures) – Carbonation Each microbial group has an optimal pH but typically lives over a pH range of three to four pH units [308]. Division based on pH preferences is in (i) acidophiles (optimal pH < 5), (ii) neutrophils (optimal pH between 5 and 9), and (iii) alkaliphiles (optimal pH > 9). Thus, geochemical ageing processes that alter/decrease the pH of the concrete will activate microbial communities and thus initiate further concrete degradation by acid and or sulphate attack by these communities [285]. The evolution of pH decrease in concrete by these processes and by the microbial processes itself will lead to a succession of different microbiological communities [275, 284, 309]. Apart from inducing corrosion, carbonation often has a positive consequence on Portland cement due to porosity decrease; however, a combination of it with bacterial colonisation can harm the concrete [275].

Alkali-aggregate reactions No information found.

Ettringite and thaumasite reactions (DEF/sulphate attack)

4.7.3.2 Physical-Mechanical

Freeze-thaw No information/not relevant

Elevated and high temperature (<150°C, no fire) No information/not relevant

Irradiation Not relevant – the ageing changes due to irradiation will have no effect on the ageing effects of the bacterial processes. When irradiation is still present, effects may be expected (see above).

4.7.3.3 Mechanical

Effects of (surface) cracks – Cracks may serve as additional locations for the development of microbial communities or biofilms.

Except for possible cracks (with extra locations for microbial communities or biofilms), no other effects of the mechanical ageing processes on deterioration by bacteria is expected.

Abrasion/Erosion/Cavitation

Creep and relaxation

Settlements and movements

Vibration (and seismic)

Thermal stresses (gradients)

4.7.3.4 Electro-chemical

Pitting corrosion

General corrosion

Other corrosion mechanisms including crevice corrosion

If a biofilm is produced, in theory, iron reducing bacteria could transform ferric to ferrous iron, thus utilizing corrosion products. Their role is not very well investigated [310].

4.7.4 Rates of deterioration

No relevant rates of deterioration were found in literature. For the chemical deterioration via aerobic or anaerobic paths, there are a few prerequisites that need to be present:

- Presence of the metabolites as sulphur or nitrogen in the correct redox state
- Good pH conditions – the pH of concrete is typically too high for optimal conditions for microbial metabolism and growth and thus colonization for most bacterial species; typically a kind of pH-decreasing process is required to lower the pH of the concrete to more optimal pH values (see further for more details).

The process of microbial deterioration is sometimes described as a three-stage process. Again, this is developed in the framework of the conditions in sewer pipes, but the general idea is probable applicable in other environments as well [309, 311]:

- Abiotic stage (phase 1) – Chemical reactions that lower the pH – being carbonation or leaching by aggressive water or industrial acids[9]
- Biotic stage; Neutrophilic bacteria (phase 2) – When the pH reaches more neutral conditions (< pH 9), several neutrophilic bacterial communities may form, e.g. sulphur oxidizing, nitric acid producing nitrifiers, but also e.g. fungal species producing carboxylic acids.
- Biotic stage; acidophilic bacteria (phase 3) – when the pH drops below 4, acidophilic sulphur oxidising microorganisms colonize the concrete surface.

Note that there are indications that deterioration from acids from biogenic origin or from other chemical origin is similar; at least in case of sulphuric acid [312].

4.7.5 Impact on concrete properties

The action of micro-organisms deteriorates the concrete through effects such as erosion of the exposed concrete surface, reduction of the concrete cover, increase in porosity and transport of solutes into the concrete [279, 313].

Chemical – Pore water chemistry The sulphur oxidizing bacteria will lower the pH of the concrete, first to pH 4 (phase 2) by more neutrophilic communities, followed by a reduction to pH 2 (phase 3) by acidophilic bacteria.

Chemical – Solid phase composition The sulphur oxidizing bacteria will convert Ca-phases to gypsum. At pH < 3, the calcium aluminates will convert also to ettringite.

Structural – Microstructure Microstructure will change following the attack by the bacterial metabolism. This is probable similar to the effects of ageing by leaching or acid attack.

Structural – Cracking Formation of biofilms of autotropic-phototropic microorganisms (algae, cyanobacteria) can lead to drying and cracking of the concrete [314]. In the stage 3 of the

[9] Specifically in sewer systems, also gaseous H_2S (produced by the sulphate reducers) lowers the pH

degradation by sulphur oxidizing bacteria, ettringite formation may lead to cracking, opening the concrete structure for further microbial activity and acid penetration [280].

Transport properties – Porosity, permeability, diffusion coefficients Some studies show an increase in porosity and subsequently in permeability or diffusion. Effects are expected to be similar to those from leaching or acid attack.

Mechanical properties Aerobe conditions – sulphur oxidation – Eriksen [287] measured a decrease in mechanic strength (in a sewer system, large amount of gypsum present).

4.7.6 Assessment methods

Microorganisms can change concrete in many ways – By forming biofilms that are directly visible and may alter the microstructure of the structure or indirectly by chemical interactions between microbiologically induced redox reactions and organic acid production. The latter leads to alterations in the concrete similar to other chemical ageing reactions such as leaching, carbonation, and sulphate attack. Linking the microbiological processes and growth with alteration in the material is the research field called biogeophysics [315] that offer potential but mainly unexplored ways for assessing the consequences of biogenic concrete degradation.

4.7.6.1 Visual inspection

Biofilms on the surface of concrete structures can easily be detected. The chemical consequences can be detected as well, e.g. gypsum formation.

4.7.6.2 Continuous monitoring

We refer to the documents of leaching and acid attack for monitoring possibilities.

4.7.6.3 Destructive testing of sampling

We refer to the documents of leaching and acid attack for monitoring possibilities.

To characterize the bacterial communities in and on concrete, a DNA extraction protocol from concrete was tested in [316]; however, its use for routine monitoring is probably not straightforward.

4.7.6.4 Non-destructive techniques

We refer to the documents of leaching and acid attack for monitoring possibilities.

4.7.7 Performance indicators & acceptance criteria

At locations with visual inspection possibilities, the indicator could be the presence of biofilms, often visible by colour changes. Given the potential detrimental reactions, actions could be taken probable rapidly by removing these biofilms (e.g. cleaning).

For the consequences of the metabolites of the microorganisms, performance indicators and acceptance criteria similar to those for acid attack and leaching are valid.

4.7.8 Model approaches

4.7.8.1 Empirical models

As discussed above, a three-phase model could describe the evolution of the rate of deterioration caused by different communities of microorganisms (based on studies for sewer systems). However, even quantitative information and applications of such model for sewer systems are very rare. Wells and Melchers [311] expressed the loss of sound concrete (in a sewer system), by a bilinear model with two parameters: a time lag for corrosion to be initiated and a longer-time steady-state corrosion rate (in [311] in mm/year, thus not a diffusion-driven rate in mm/year$^{0.5}$). Both parameters depend on influencing factors as temperature, relative humidity, aggressivity of the environment, but, unlike as for other ageing processes as carbonation, the relations or numerical values of these parameters are not established. An attempt was made in [317]. However, this was done for sewer conditions which are probably not relevant for nuclear power plants.

4.7.8.2 Phenomenological models

No specific information at this subsection.

4.7.8.3 Complex coupled models

In theory, there are codes available that could implement relevant processes for calculating the impact of micro-organisms. Reactive transport codes can incorporate [318]:

- Metabolic reactions defined by Monod type of equations (see Eq. (8))
- Simple growth of biomass (typically linked to metabolic reactions with a yield coefficient) and first-order decay
- Transport (diffusion) of the metabolites inside the concrete
- Aqueous complexation reactions (e.g. formation of Al-Ca-Mg organic complexes)
- Reactions between the cement phases and the changed pore water concentration using thermodynamic models leading to mineralogical and pore water alterations
- Accounting for the evolution of porosity and diffusivity of the concrete.

Examples exist of such model approaches: De Windt and Devillers [319] simulated a bioleaching test applied to ordinary Portland cement; Yuan *et al.* [320] applied a reactive transport model to simulate biodeterioration of concrete in contact with H_2S gas (sewer system with a community of sulphate oxidizing bacteria). Note that there exist other studies that simulate the effect of organic or inorganic acids on cement without accounting for the specific rates of production imposed by the bacterial communities (e.g. [321, 322]). No studies with both the micro-organism and the chemical corrosion of the concrete in the framework of ageing of nuclear power plants were found.

The benefit of such type of models is that they can be applied to specific conditions relevant for the reinforced concrete structures in nuclear power plants. On the other hand, the data input requirement is high (both concrete properties and environmental conditions) and some parameters or variables are difficult to obtain, specifically those related to the rates of acid production by the micro-organisms.

4.8 Freeze-thaw

4.8.1 Process Definition

The freeze-thaw durability of concrete is defined by its ability to resist repetitive freeze-thaw cycles throughout a specified service life without exceeding specified deterioration levels. Freeze-thaw degradation is generally classified according to the type of damage induced: *internal structural damage* and *surface scaling*. Freeze-thaw deterioration is a dynamic process where the constant movement of heat, water and dissolved substances occurs over time, both into and from the concrete, and also inside the concrete microstructure.

Concrete is usually exposed to periodical wetting and drying, deicing salts and other aggressive forms of loading in addition to freeze-thaw cycles. The degree and type of damage caused by freeze-thaw attack depends on [323]:

- material properties determined by concrete composition including porosity, pores size distribution and strength;
- the actual service environment, i.e. the conditions at the concrete surface and their variation with time covering relative humidity, surface contact with water and temperature;
- the degree of saturation which varies with time and location in the concrete due to moisture transport by capillary suction, water vapour diffusion together with capillary condensation and water vapour sorption.

Besides moisture content, environmental factors such as the minimum freezing temperature, the rate of freezing and the cation types in the solution are important.

Freeze-thaw induced internal structural damage is caused by the freezing of nearly saturated concrete. Freeze-thaw surface scaling occurs when there is a freezing medium, typically weak salt solution, on the concrete surface. In both cases there will be moisture transport in concrete during the freeze-thaw cycles, and some form of cracking in the concrete. The mechanics causing the cracking and the type of cracking is however different based on the actual degradation mechanics [323].

There are still no generally accepted theories that describe all aspects of freeze-thaw deterioration. Details pertaining to theories for freeze-thaw induced internal structural damage can be found for the closed container theory [324], the hydraulic pressure theory [325-329], the microscopic ice lens growth and crystallization pressure [326, 330-333], the moving ice front theory [330], the osmotic pressure theory [333, 334], and the critical degree of saturation model [332, 335, 336]. Regarding freeze-thaw induced surface scaling, details pertaining to theories can be found for scaling mechanism presented by [337], the micro-ice-lens model, with pumping effect or cryogenic suction pump [329, 331, 332, 338-341], the glue spall mechanism theory [342-349].

4.8.2 Influential factors

Reference:
The phase transformation of water present in the tortuous porous network of concrete into a solid form is a complex phenomenon for which there is no generally accepted mechanism that account for the different manifestations of damage in concrete. This action is influenced strongly by the pore size, temperature, and the presences of solutes.

4.8.2.1 General conditions

Effect of moisture content and relative humidity The freeze-thaw deterioration of concrete is directly linked to the moisture content in concrete and whether there is ponding on the concrete surface. If the critical degree of saturation is exceeded, including all concrete porosity (capillary, gel and all air pores), freezing will cause internal cracking [336].

Effect of temperature The main temperature related factors influencing freeze-thaw damage are the minimum freezing temperature, the rate of freezing, the duration of the minimum temperature period and the number of freeze-thaw cycles [323, 336, 345].

Effect of irradiation

Effect of microbiological reactions

Effect of exposure condition (free, sheltered, buried, submerged) These influence directly the moisture content of concrete, and the chemical composition of the concrete pore water due to potential contact with external sources. Whether the external surface of concrete are horizontal or vertical, and whether chlorides exist in the solution in contact with the surface are critical aspects.

Effect of external water composition (submerged or buried structures) The freezing-point depression may be caused by ions in the solution. The vapour pressures of ice and liquid water are equal at 0°C. However, when ions are introduced to the liquid water, the vapour pressure of the liquid water is lower than that of pure ice. These conditions cause no ice to form. As the temperature drops, the vapour pressure of ice decreases more rapidly than that of liquid water. The vapour pressure of the liquid water and pure ice eventually become equal. The temperature at which the vapour pressures are equal is the freezing-point temperature. Typical cement pore solution contains calcium and alkali hydroxides with a total concentration of approximately 0.5 M [350]. As ice forms the solute is rejected by the ice crystals and the concentration in the remaining liquid rises [351].

With regards to scaling, surface ponding with low chloride concentrations (2 - 4%) produces more scaling than higher concentrations. It appears that the pessimal chloride concentration is not related to the concrete quality. The concentration of the outer solution had a greater influence on the scaling mechanisms than the chloride concentration in the concrete pore structure [352].

Effect of atmospheric conditions (rain, wind, freeze/thaw) See moisture content and temperature.

Effect of mechanical and structural loading Very few studies have been conducted on this topic. The combined loading of mechanical fatigue (compressive, flexural loading) and freeze-thaw has been shown to lead to premature deterioration of concrete materials [353, 354]. Furthermore, scaling of concrete surface under sustained load revealed that acceleration of surface scaling was observed [355].

4.8.2.2 Mixed design properties

The deem-to-satisfy approach is based on calibration to long-term experience. It considers limitations to the porosity of the concrete (maximum water/binder-ratio, minimum cement content, and required curing conditions, where the later are important for especially the porosity of the exposed surface). Alternatively, it can be expressed as a minimum strength requirement. It also considers limitations related to the binder composition (e.g. cement type and the use of additions like silica fume, fly ash, and slag) and on aggregates (on their own should be frost resistant). There are also requirements related to the air void structure, i.e. total air content and spacing factor. In some national provisions, adequate performance can be demonstrated by accelerated freeze-thaw testing.

Cement type and supplementary cementitious materials (SCM) The type of binders can affect the quality of the concrete to a certain degree, such as porosity, pore structure, etc., which in turn can affect the freeze-thaw performance of concrete. It is also known that air certain entraining agents can produced air more effectively with certain cements types than with others. For this reason, it is important to consider the compatibility of the air entraining agent with the binder materials, and also with the other admixtures used in the concrete.

Cement-water binder ratio Water-binder ratio is synonymous with concrete quality. As the ration drops, the quality improves (porosity, pore structure, etc.) and the mechanical properties increase, contributing to improved resistance against freeze-thaw cycles.

Concrete composition (aggregate content, types of aggregates) For structures exposed to freezing and with the potential for a higher level of moisture than the critical degree of saturation, a design according to the deemed-to-satisfy method is the normal procedure. The mix design requirements (air-void requirements, w/b-ratio, binder type and minimum content, composition, strength, freeze-thaw resistant aggregate, etc.) are chosen as a function of the environmental exposure conditions.

Examples of the deem-to-satisfy approach are given in the EN 206:2014, ACI 201.2R-16, ACI 318-14, and also in many national application documents.

Freeze-thaw resistant aggregates: concrete consists of 60% to 75% (in volume) aggregate. Therefore, aggregate freeze-thaw performance should be taken into consideration.

Air entrainment: the most important factor affecting freeze-thaw performance is the quality of the air-void system (volume, size, and spacing of the air-voids). The air-void system parameters typically used to assess the quality of an air-void system include the air content (volume %), spacing factor (mm), and specific surface (mm^{-1}). These are determined in hardened concrete. Field experience/research have shown that internal damage is almost non-existent if the spacing of air-voids is low enough (< 0.20 – 0.25 mm). An inadequate entrained air-void structure is generally detected in connection with both freeze-thaw cracking and surface scaling. There may be too low air content (< 3 – 5 %) or too big air-voids with too low aerage specific surface area (< 25 mm^2/mm^3).

Curing (temperature, moisture, time) conditions The effect of curing type on the freeze-thaw performance was significant. Proper concrete curing and high enough degree of hydration before any environmental exposure is known to be essential for concrete durability. The hydration during the first few days has the greatest effect on the overall performance of the concrete. Industrial curing practices incorporate a number of methods for good curing practices to minimize moisture loss. The ideal curing maintains saturation within the concrete surface for the shortest period of time, such that additional effort would have negligible effects on the desired concrete properties [323, 356].

4.8.3 Influence of other ageing processes

4.8.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) These process increases the pathway for moisture and ions into concrete and weakens the mechanical /physical properties of surface concrete. No literature found relating these two mechanisms.

Leaching (submerged or buried structures) This process increases the pathway for moisture and ions into concrete, and weakens the mechanical/physical properties of surface concrete [357]).

Alkali-aggregate reactions This process increases the pathway for moisture and ions into concrete, potentially resulting in increased deterioration [358, 359].

Carbonation This process increases the pathway for moisture and ions into concrete, and coarsens surface concrete porosity when coupled with freeze-thaw cycles [323, 360].

Ettringite and thaumasite reactions (DEF/sulphate attack) Ettringite can be formed from monosulfate during freeze-thaw attacks with and without de-icing salt [361, 362].

Bacterial processes Not relevant/applicable

4.8.3.2 Physical-Mechanical

Elevated and high temperature (<150°C, no fire)

Irradiation

4.8.3.3 Mechanical

Effects of (surface) cracks Crack influence freeze-thaw because they facilitate the ingress of moisture and chemical components that might influence performance [351].

Abrasion/Erosion/Cavitation This weakens the mechanical/physical properties of surface concrete, and increases pathway for moisture and ions into concrete [363].

Creep and relaxation

Settlements and movements

Vibration (and seismic)

Thermal stresses (gradients)

4.8.3.4 Electro-chemical

Pitting corrosion

General corrosion

Other corrosion mechanisms including crevice corrosion

4.8.4 Rates of deterioration

Freeze-thaw deterioration can be measured as degree of internal cracking (e.g. %) for internal structural damage and amount of scaling (kg/m^2) for surface scaling. The amount of damage is strongly linked to the number of freeze-thaw cycles a specific concrete is subject to, the minimum freezing temperature and duration it is maintained. While from literature it is very hard to find specific values for deterioration rate, or even quantified damage, from the concrete codes an indirect indication is given through the specifications for performance testing.

Typical measurements (based on laboratory measurements in accelerated condition) include sample dilation, relative dynamic modulus (measured by ultrasound or fundamental frequency) [364], cumulative amount of scaled off material [365]. Values vary with country specific guidance. For example, RDM_{56}> 67-85 %, or M_{56}< 0.2-1.0 kg/m^2), depending on exposure class, service life period, etc.

4.8.5 Impact on concrete properties

Chemical – Pore water chemistry Not relevant/applicable

Chemical – Solid phase composition Not relevant/applicable

Structural – Microstructure Other than microcracking/scaling, not relevant/applicable

Structural – Cracking When exposing concrete with poor freeze-thaw performance to a standardised freeze/thaw test in the laboratory, the orientations of the resulting cracks are more or less random. Hasholt [366] studied the influence of confinement during freeze/thaw loading on the developed crack pattern. The results show that confinement can change the outcome of a freeze/thaw test as regards extent of internal cracking, crack orientations, and amount of surface scaling. Samples taken from concrete from walls and pavements and studied under microscope have shown that the preferred orientation of the crack pattern parallel to the exposed surface [366].

Transport properties – Porosity, permeability, diffusion coefficients Research shows that concrete subject to freeze-thaw damage observe changes in the transport properties (pressure gradient, capillary suction, gas permeability, etc.). transport properties increased as a function of the number of freeze-thaw cycles. The amount of observed change is strongly dependant on the quality of the concrete and the freeze-thaw loading characteristics [367].

Mechanical properties Research results showed a significant influence of freeze-thaw loading on the compressive strength and even more influence on the modulus of elasticity and the compressive strain at peak stress. Reduced tensile strength and increased fracture energy were measured. Furthermore, from inverse analysis of wedge splitting test results, a significant effect of freeze-thaw on the shape of the tensile stress crack opening relationship was observed: tensile strength was reduced, while the post-peak behaviour was more ductile for the frost-damaged concrete. Pull-out tests showed the influence of freeze-thaw cycles on bond strength and slip [368].

4.8.6 Assessment methods

Several experimental procedures have been used to assess the freezing process. These test setups, conducted in laboratory, allow to a certain degree to better understand the initiation, continuation and the resulting damage of freeze-thaw loading. There is no standardized way of measuring ice/water phase change related parameters, so a wide variety of tests have been performed. Kaufmann [369] has studied several different test methods to monitor heat release, mechanical deformation and damage during a series of freeze-thaw cycles. The results of such tests help to interpret super cooling, salt segregation, ice front penetration and thawing characteristics. These include:

- Low temperature calorimetry [369];
- Expansion measurements [347, 369-371];
- Acoustic emission measurements [369];
- Ultrasonic measurements [369, 372, 373];
- Neutron diffraction [374, 375];

- Dielectric measurements [376],
- Nuclear magnetic resonance [377];
- Directional solidification [378];
- Environmental scanning electron microscopy [379];
- DMS [380].

Performance testing of concrete is typically based on accelerated freeze-thaw testing. In the European standard (CEN/TR 15177 [364]) there are three different methods for the estimation of the freeze-thaw resistance of concrete with regard to internal structural damage. These methods produce relatively consistent results. No single test method is established as a reference test method. In North America the most commonly used testing standard for freeze-thaw resistance is the ASTM C666 / C666M-03 [381]. In this method, the freezing rate is very high. There has been criticism on the method because it does not represent well natural freeze-thaw exposure. However, this method is widely known and frequently used. Another European testing specification, this one for surface scaling CEN/TS12390-9 [365] there are three testing methods for concrete freeze-thaw scaling. There is no established correlation between the results obtained by these three test methods. All tests will identify poor and good behaviour, but they differ in their assessment of marginal behaviour. The slab test is the reference method. The Swedish Standard SS 13 72 44 (´Borås method´) [382] is nearly identical to the ´Slab test´ in CEN/TS 12390-9 [365], and can also be considered as an improved version of the ASTM C672 method. The ASTM C672 method is intended for use in evaluating the surface resistance qualitatively by visual examination. The French XP P18-420 [383] standard presents the particularity of including specification on storage before scaling test.

4.8.6.1 Visual inspection

Of the freeze-thaw deterioration mechanisms, only concrete scaling can be observed visually.

4.8.6.2 Continuous monitoring

It is uncommon to monitor concrete structures for freeze-thaw deterioration. References in published literature refers mainly to research projects where temperature and moisture content (relative humidity) in the concrete are monitored as a function of depth from the surface and time [323, 384]. In some studies, strain gauges are also used [384].

4.8.6.3 Destructive testing of sampling

Freeze-thaw performance testing is by definition destructive. See above in the beginning of section 4.8.6.

4.8.6.4 Non-destructive techniques

It is uncommon to conduct non-destructive testing on concrete structures for freeze-thaw deterioration. As for monitoring, references in published literature refers mainly to research projects specific NDE methods used. These include acoustic emission in a passive mode to identify crack development in concrete specimens exposed to freeze-thaw cycles [385], ultrasonic measurements [386], dielectric measurements and fundamental transverse frequency [384].

4.8.7 Performance indicators & acceptance criteria

Freeze-thaw test methods vary considerably, and assess performance based on, for example, volume change, mass change or change in dynamic modulus of elasticity. These properties are of significant practical value, but they do not provide any clear direct evidence for a particular mechanism of freeze-thaw action. A test method should classify different concrete in the same way that exposure in the field would. No single test method can fully reproduce the conditions in the field in all individual cases. The definition of limiting values will require that the correlation between (laboratory) test results and field experience be defined, in accordance with local conditions. Table 8. Some international test methods for characterizing freeze-thaw performance presents some test methods used to determine performance indicators. Acceptable values are typically provided in national guidance documents (e.g. in Finland the SFS 7022:2018 [387]).

Table 8. Some international test methods for characterizing freeze-thaw performance.

Country & test method	Type of test	Comments /Measurement of resistance
Europe, CEN/TR 15177 [364]	Internal structural damage	Three test possibilities: slab test, CIF method and beam test. / length change, RDM. Values vary with country specific guidance (E.g. RDM56 > 67-85 %), depending on exposure class, service life period, etc.
Europe, CEN/TS 12390-9 [365]	Scaling	Three different test possibilities: slab test, CDF method and cube test. / cumulative amount of scaled off material.
North America, ASTM C666M-15	Internal structural damage	Values vary with country specific guidance (E.g. M56 < 0.2-1.0 kg/m2), depending on exposure class, service life period,
North America, ASTM C672-12	Scaling	Two test possibilities: rapid F/T in water, and rapid freezing in air and thawing in water. / durability factor based on RDM.

4.8.8 Model approaches

Currently there are no validated time-dependent models for the performance determination of structural concrete subject to freeze-thaw loading or freeze-thaw loading in the presence of salt solutions. Experimental studies on the freeze-thaw resistance of concrete reveal a complex behaviour due to intrinsic thermo-hydro-chemo-mechanical coupling. Published models that simulate freeze-thaw highlight the challenge to consider all freeze-thaw phenomena such as hydraulic, hydrostatic and osmotic pressures into pores, swelling and shrinkage, scaling, etc. [388].

Models show interesting results but are limited to a few of the freeze-thaw phenomena listed previously. In the following only a few models are mentioned of the many available in published literature.

4.8.8.1 Empirical models

The use of experimental data to predict the macroscopic behaviour of concrete subjected to severe external conditions.

Some examples of model used are [388]: service-life model to describe damage into concrete during freeze– thaw cycle based on the critical water content determined experimentally [336]; model based on many experiments to define specific mechanical relations for compressive and tensile strengths [368]; link between the fractal dimension of the internal structure of concrete and the characterization of the voids size distribution [389]; artificial neural networks based on data relating to the composition of concrete, more specifically the w/c ratio and the percentage of aggregates, the maximal number of freeze-thaw cycles [390]; fuzzy logic to assess the durability of concrete from its compressive strength, the level of loading and the number of freeze–thaw cycles [391]; and, model based on composition of the concrete and freeze-thaw loading conditions [360, 392, 393].

4.8.8.2 Phenomenological or analytical models

The description of mechanical, thermodynamic and/or hydraulic phenomena which take place in the material. These models are usually compared retrospectively with the experimental data in ordr to assess their validity.

Some examples of model used are [388]: poro-elastic model with prediction of the effect of air bubbles [327, 394-396]; thermo-hydro-mechanical model with damage and strain fields [357, 397, 398]; lattice model with stress field and change in the number of microcracks [399]; cohesive zone model with crack growth [400]; critical water content probabilistic model [336] and, theory of porous medium with volume fraction (scaling) [401].

4.9 Elevated and high temperatures

Concrete exposed to elevated temperature will experience both mechanical and physical changes. Because thermally induced changes, loss of structural integrity, and release of moisture and gases resulting from the migration of water could adversely affect the operation of the NPP and its safety, a complete understanding of the behaviour of concrete under elevated-temperature exposure is needed.

4.9.1 Process Definition

Concrete is a heterogenous and porous material. When exposed to high temperature the characteristics of the concrete can change (depending on the level of the temperature), which can lead to serious damage. Several factors will influence the susceptibility of the concrete to elevated and high temperature.

- Amount of capillary, absorbed and chemically bound water.
  During the production of concrete, water is added to the mixture to induce a reaction with the cement. However, part of the water added is not used for the hydration of cement and remains in the concrete as capillary, absorbed and chemically bound water (Figure 3). High temperatures cause the release and evaporation of these types of water, which induces temperature gradients. This phenomenon causes stress in the pores to build up, thermal stress to arise and the concrete to shrink and crack due to the release of water [402]. The degree of saturation of concrete will therefore have a significant impact on the severity of the effect of high temperatures. If concrete is completely dry, it will sacrifice non-evaporable water or hydrated water immediately which will lead to damage due to the decomposition of C-S-H. On the other hand, if concrete is thoroughly saturated, steam will be formed resulting in high internal pressure, which can also lead to damage as explained above. A well-considered choice of the amount of water is necessary to obtain a concrete with a high resistance to high temperatures.

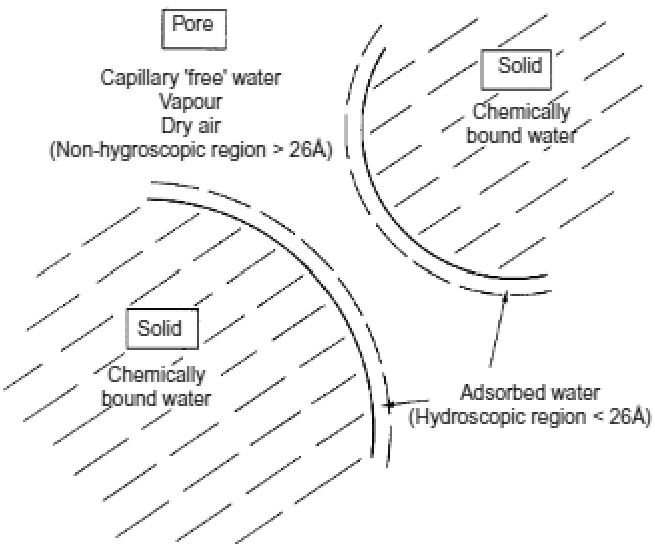


Figure 3. Different forms of water in concrete [402].

- Water-cement ratio
  The amount of water (free – absorbed – chemically bound) and the amount of hydrated cement particles in concrete depends on the amount of water added to the cement paste. This characteristic is expressed as the ratio of mass of water to mass of cement (W/C-ratio). In order to hydrate all the cement grains in a given cement paste, the mass of water added is ideally around 40 to 43 % of the mass of cement (0,40 ≤ W/C-ratio ≤ 0,43). For W/C-ratios lower than 0,40 not all cement particles will be able to react with water, leaving unhydrated cement grains in the cement paste. Under the influence of elevated temperatures, this type of concrete will not experience major effects from the evaporation of free internal water. However, chemically bound water will evaporate from between the C-S-H sheets, which will induce damage. At high W/C-ratios, the excess of water leads to the formation of capillary pores and the presence of a lot of free water. At high temperatures, this free water will evaporate resulting in an increase in volume and the creation of internal pressures. In order to obtain a structure that is as resistant as possible to high temperatures, it is necessary to ensure that as many cement particles as possible are hydrated and that as little free water as possible is present in the structure. Such a situation is obtained with a W/C factor between 0,40 and 0,43. In addition to having no excess water and fewer pores and capillaries at this W/C factor, the structure will also have a higher strength compared to the other situations [403].
- Temperature level
  Depending on the temperature level to which the concrete is exposed, changes in cement paste and aggregates will be induced.

  - Changes in aggregates
    Approximately 70-80% by volume of a standard concrete mix consists of aggregates. Therefore, this part of the mixture must be taken into account when determining the sensitivity of the concrete to high temperatures. However, aggregates will not be affected by temperatures up to 150°C, as commonly used aggregates are thermally stable up to 300°C-350°C. For example: quartz (aggregates and sands) are stable up to 570°C and carbonate stones (limestone and dolomite) are stable up to 600°C [404].

  - Changes in cement paste
    The degradation of cement starts at approximately 80ºC with the decomposition of ettringite. Subsequently, the cement paste dehydrates which induces microstructural changes in the concrete. First the free, capillary water, which is

not influenced by the Van der Waals attraction forces, evaporates followed by the by physically (lamellar and adsorbed) bound water. Lastly, the water chemically associated with C-S-H (calcium silicate hydrate) is lost. This C-S-H dehydration is a physicochemical process that causes the relaxation of microtensions and is responsible for transient creep. The loss of chemically bound water starts at 100 ºC and induces cement paste degradation, together with the cracking and porosity increase [405].

After exposure to high temperatures, cooling also promotes changes in the cement paste. The cementitious products in the paste may rehydrate, forming new gels or crystalline components. The formed lime also rehydrates and expands, creating new fissures [405].

- Heating rate
  Pooja Pooja *et al.* [406] investigated the effect of the heating rate on concrete. It was concluded that when a specific type of concrete is exposed to high temperatures for exposure periods of 30 minutes, 1 hour and 1,5 hour, that the decrease of the strength with respect to the increased temperature is the same for both slow and fast heating. For longer exposure periods, the concrete experiences severe deterioration for slower heating rate and higher temperatures, because the presence of heat lasts longer. However, A high heating rate can also be detrimental to the concrete, since in this situation the water in the surface of the concrete evaporates all at once, causing a huge increase in volume and pressure which results in spalling of the concrete. Spalling is a phenomenon whereby large shells of the concrete surface become detached.

  Naus [407] stated that a heating rate ≥ 2°C/min can have an influence on the structure (second order influence). Starting from a heating rate of 5°C/min, the concrete structure will be damaged anyhow (first order influence) (Table 9).

- Thermal cycling
  The number of hot and cold cycles also have an impact on the effect of high temperatures on concrete. In the first cycles the free, absorbed and chemically bound water will evaporate. Because of the increase in volume from water to vapour, internal stresses will occur, and the concrete will be damaged. In addition, because of the difference in temperature between the internal concrete, the concrete in contact with the external environment and the concrete exposed to the high temperatures a temperature gradient will be induced. This temperature gradient causes the hot concrete to expand and the colder concrete to contract. This phenomenon results in cracks. The formation of cracks allows the heat to penetrate deeper into the concrete every cycle, forming a temperature gradient deeper into the concrete and causing damage in that deeper part. Also, the cooling phase, after the concrete is exposed to elevated temperature, will have an influence on the structure of the concrete. As stated in Naus [407]a cooling rate less dan 2°C/minute will have a negligible influence on the concrete. In contrary a cooling rate above 2°C/minutes can induce cracking because of the fast decrease in temperature [407] (Table 9).

- Thermal conductivity and exposure duration
  Thermal conductivity is a material property that refers to the intrinsic ability of the material to transport or conduct heat across a temperature gradient (from high temperature to low temperature). The amount of moisture in concrete has a significant influence on the thermal conductivity.

  Thermal conductivity is usually measured by means of “steady state” or “transient” test methods. Transient methods are preferred over steady-state methods, as physiochemical changes of concrete at higher temperatures cause intermittent direction of heat flow. The thermal conductivity is expressed in $\left[\frac{W}{m.K}\right]$ [408].

In summary, damage due to elevated and high temperatures (<150°C) will be induced by two types of mechanisms. The first mechanism, in which water at the surface of the concrete evaporates under the influence of the high temperatures, induces stresses at the surface level. The stresses are the result of the volume increase related to the transition of water to vapour. The second mechanism is that of the formation of a temperature gradient between the warm concrete surface and the colder inner concrete. This temperature gradient also induces internal stresses.
Due to these two mechanisms, temperatures up to 150°C will predominantly cause damage at the surface layer of the exposed concrete. The damage will be characterized by cracking of the surface.

Table 9. Factors influencing the susceptibility of concrete to elevated and high temperatures [407].

Factor	Influence	Comment
Temperature Level	•••	• Chemical-physical structure (see Chapter 2) & most properties (see Chapters 6-14).
	••	• The properties of some concrete (e.g. compressive strength and modulus of elasticity) when heated under 20-30% load can vary less with temperature - up to about 500°C - than if heated without load (see Chapters 6 & 14).
Heating Rate	••	• *< 2°C/min*: Second order influence.
	•••	• *> about 5°C/min*: Becomes significant → explosive spalling.
Cooling Rate	•	• < 2°C/minute: Negligible influence.
	••	• > 2°C/minute: Cracking could occur.
	•••	• *Quenching*: Very significant influence.
Thermal Cycling	••	• *Unsealed concrete*: Significant influence mainly during first cycle to given temperature.
	••	• *Sealed concrete*: Influence in so far as it allows longer duration at temperature for hydrothermal transformations to develop.
Duration at Temperature	••	• *Unsealed concrete*: Only significant at early stages while transformations decay.
	•••	• *Sealed concrete*: Duration at temperature above 100°C → Continuing hydrothermal transformations.
Load-Temp. Sequence	•••	• Very important - not usually appreciated
Load Level	•••	• < 30%: Linear influence on Transient Creep (Chapter 9) at least in range up to 30% cold strength.
	•••	• >50%: Failure could occur during heating at high load levels
Moisture Level	••	• *Unsealed*: Small influence on thermal strain and transient creep particularly above 100°C.
	•••	• *Sealed*: Very significant influence on the structure of cement paste and properties of concrete above 100°C.

••• First Order influence. •• Second order influence • First order influence

An example of concrete subject to temperatures up to 150°C is the concrete from the steam exhaust rooms of the NPPs. The function of the steam exhaust rooms is to protect the steam circuit from an aircraft crash. Without this concrete structure, the aircraft's kerosene could enter the circuits, and this must be avoided.

In the steam exhaust rooms, the concrete is exposed both to high temperatures and to high humidity originating from steam. The steam is blown onto the concrete at high pressure, the water will penetrate deeper into the concrete compared to an environment with the same humidity but with a lower ambient pressure. The steam condenses in the concrete and will evaporate at the next release under the influence of high temperature. This creates internal pressures resulting in cracks. The cracks allow more water to penetrate deeper into the concrete at the next cycle, increasing the damage with each steam release cycle [402].

4.9.2 Influential factors

Reference: Naus [407] provides a short overview of all factors influencing the susceptibility of concrete to elevated and high temperatures together with the weight of those factors (Table 9).

4.9.2.1 General conditions

Effect of moisture content and relative humidity The higher the moisture content and/or the relative humidity of the environment, the higher the amount of water in the concrete and therefore the higher the internal stresses due to the evaporation of the unbound and chemically bound water in the pores. However, if the exposed concrete is bone dry, it will sacrifice the non-evaporable water or hydrated water which will also lead to damage due to the decomposition of C-S-H. Thus, a good balance must be found in terms of moisture content in the concrete, in that way the influence of high temperatures is limited. As explained in section 4.9.1, a W/C factor between 0,40 and 0,43 is favourable.

Effect of irradiation Not relevant

Effect of microbiological reactions Not relevant

Effect of exposure condition (free, sheltered, buried, submerged) Concrete structures exposed to an outdoor environment or exposed to an environment with a high relative humidity will be more affected by high temperatures compared to indoor concrete structures at low relative humidity. This is because the amount of water in the structures, exposed to a climate with high relative humidity, will be higher.

Effect of external water composition (submerged or buried structures) The composition of the external water will have no significant influence on the sensitivity of the structures to high temperatures. The fact that salt water will evaporate at a slightly higher temperature compared to pure water will, for example, not significantly affect this damage mechanism, since the boiling points will be situated in the same order of magnitude. However, when salt water evaporates salts remain in the concrete, the remaining chlorides can cause chloride-induced corrosion at the reinforcement level. Thus, the composition of the external water does not significantly affect the sensitivity of the concrete to high temperatures, but it may affect the sensitivity of the concrete to other ageing mechanisms.

Effect of atmospheric conditions (rain, wind, freeze/thaw) Same principle as described at 'Effect of exposure condition' as for humidity. However, freeze/thaw cycles induce cracking, making the concrete more susceptible to high temperature. Cracks will allow heat and moisture to ingress into the structure.

Effect of mechanical and structural loading Not relevant - At relatively low temperatures, whether or not the structure is loaded will not play a role in terms of its sensitivity to high temperatures. At temperatures many times higher than 150°C, loading will have an influence. For loaded structures exposed to very high temperatures, the stresses to which the structure is already exposed to will initially counteract the stresses induced by the high temperatures, making the loaded structures more resistant to high temperatures. However, after a certain period of time the stresses carried out on the structure will be added to the stresses induced by the high temperatures. Consequently, loaded structures will fail faster at very high temperatures [409].

4.9.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) At temperatures below 150°C, the decomposition of ettringite will occur [410]. However, this will have no significant influence on the concrete strength, since ettringite concentrations are usually very low as this is a primary constituent of the hydration of Portland cement concrete.

According to Lim [410] decomposition of C-S-H, which is the primary hydration product and the principal binding phase in cement paste/concrete, has been reported to occur within a wide temperature range due to its amorphous characteristics. The C-S-H will start to dehydrate at 100°C. In spite of the degradation of C-S-H at relatively low temperature, manifestation of this degradation on the micro- and macroscopic level does not occur until C-S-H is exposed to much higher temperature [410].

In general, the behaviour of concrete at maximum temperatures of 150°C will not be influenced by the cement type or by the added supplementary cementitious materials. The microstructure of the cement paste and therefore the properties of the concrete will not change significantly up to 300°C [410].

Changes in compressive strength depending on the temperature of thermal load – Above 100°C, residual water in cement paste can be liberated as steam which will affect the surrounding phases of the cement paste. Since the steam cannot be liberated fast enough, a high internal pressure will be created. The combination of the high pressure and high temperature can result in a condition in which the remaining unhydrated cement particles can hydrate, thereby initially increasing the overall strength of the concrete when exposed to elevated temperatures. At higher temperatures, the increase in strength will logically stop and the strength will decrease due to degradation of several cement components [409, 410].

The impact of grinding fineness on the heat resistance of concrete – Concrete containing finer ground cement will be less sensitive to high temperatures, because:

- Smaller cement particles result in larger specific surface area and therefore a lower number of unhydrated clinker particles. In other words, less unbound and chemically bound water will be present in the concrete;
- The initial hydration rate is higher for smaller sized cement particles. Therefore, the formed C-S-H fibres are also smaller and will have a smaller influence on the structure during degradation, and;
- Because more cement particles will be hydrated, a dense (less porosity) concrete structure will be obtained. Water from the environment will penetrate less easily into the concrete.

The impact of chemical composition of cements on their temperature resistance The binding phase of cementitious materials consists of C-S-H whose stoichiometry is defined by the ratio Ca/Si and H2O/Si in the cement [411]. At temperatures below 150°C it is mainly the amount of C-S-H gel which will have an influence on the resistance of the cement paste to high temperatures. Because of the elevated temperatures, the C-S-H gel will dehydrate causing a decrease in volume of hydrates. This will result in an increase in the porosity of the cement matrix and consequently an increase in the total volume of pores and the average size of pores.

Cement-water binder ratio A higher W/C ratio implies a higher porosity of the concrete. High porosity implies that environmental water will penetrate more easily into the concrete. This water will remain in the concrete as free water and will consequently, at high temperatures, expand and cause damage. In addition, a high W/C ratio also means a high concentration of unbound and chemically bound water in the concrete which will further increase the damage to the concrete structure. As explained before, the goal of a concrete technologist is to design a concrete mixture in which the ratio of water to cement is chosen in such a way that:

- Excess water is avoided. Excess water will not be able to participate in the hydration reaction and remains as capillary water in the concrete. This causes the creation of pores.
- Sufficient water is added. By adding the precise amount of water, as many cement particles as possible hydrate and the strength of the concrete is optimized.

Concrete composition (aggregate content, types of aggregates) Not relevant at temperature up to 150°C. This property is only relevant at very high temperatures. Depending on the type of aggregate, the aggregate will experience faster damage at higher temperatures. It is recommended, for structures that are subject to very high thermal loads or service temperatures, not to use no carbonated or siliceous aggregates (such as limestone and sandstone for example), but to use granules that are resistant to fire, such as basalt, expanded clay, terracotta, and so on.

Curing (temperature, moisture, time) conditions The higher the temperatures during hardening and the faster the curing time, the greater the chance of internal and external restrained shrinkage and therefore thermal cracking. In case the crack width is greater than 0,4 mm, these cracks form access routes for water and heat in concrete. Through the cracks, water migrates into the concrete and will cause damage deeper into the structure as a result of expansion due to evaporation [412].

4.9.3 Influence of other ageing processes

4.9.3.1 Chemical/biological processes

A chemical or biological attack consists of dissolution of substances or chemical reactions between substances and components of the concrete. Reaction products might cause problems, due to dissolution or expansion. Examples of these chemical attacks are:

- Acid attack dissolving the binder from the concrete surface;
- Sulfate attack from the surface, by ground water or sea water, or internal sulfate attack (‘delayed ettringite formation’) creating a reaction product that absorbs a significant amount of water, causing internal swelling and cracking;
- Alkali–aggregate reactions from alkali from the cement, or the exterior, reacting with components of certain reactive aggregates, to produce expansive products and cracks;
- Carbonation or neutralization from weak acids, including carbon dioxide, that reacts with components in the pore liquid, to reduce pH. The carbonation reaction results in a volume increase and the release of water ($Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$). This will generally reduce porosity and allow non-hydrated cement to further hydrate with the released water. The permeability in the surface zone may decrease and the surface strength and hardness increases. As long as the carbonation front does not reach the reinforcement, carbonation does not negatively affect concrete and will somewhat protect the concrete from high temperatures by lowering its porosity [412].
- Leaching of the alkalis and calcium oxide because of a soft water attack, that in turn causes dissolution of deposited calcium hydroxide $Ca(OH)_2$ and binder components.

It is clear that these damage mechanisms damage the concrete by cracking or by changing the chemical composition. Because of the damage, the concrete will be, in most cases, less resistant to high temperatures [413].

4.9.3.2 Physical-Mechanical

A physical attack could be a non-reacting liquid, or heat, penetrating into concrete or a concrete component, causing internal stresses and expansion, that will result in internal cracking or surface scaling.

Freeze-thaw This ageing mechanism will also cause cracking and damage to the concrete.

Irradiation Not relevant.

4.9.3.3 Mechanical

Effects of (surface) cracks When a crack opening is smaller than 0,4 mm, the crack usually becomes clogged, making transport by this route more difficult. In other words, water will not be able to enter the concrete through this crack and will therefore not be able to exert any additional influence on the concrete. However, for crack openings bigger than 0,4 mm, water will be able to penetrate into the concrete and cause damage deeper into the structure due to expansion [412].
The types of damage mechanisms related to mechanical causes cause cracking of the concrete which results in easy routes for water to enter the structure and to damage the concrete when subjected to high temperatures.

4.9.3.4 Electro-chemical

Corrosion of the embedded reinforcement induces cracks in the concrete. These cracks will form an entrance for heat (and water) into concrete.

4.9.4 Rates of deterioration

The graphs below (Figure 4, Figure 5, and Figure 6) show the effect of elevated temperatures on respectively the compressive strength, flexural strength, and modulus of elasticity of concrete. The longer the exposure period, the higher the internal temperatures will rise and the greater the damage will be. It may be concluded that the modulus of elasticity is most sensitive to elevated temperatures, followed by flexural strength and compressive strength [409].

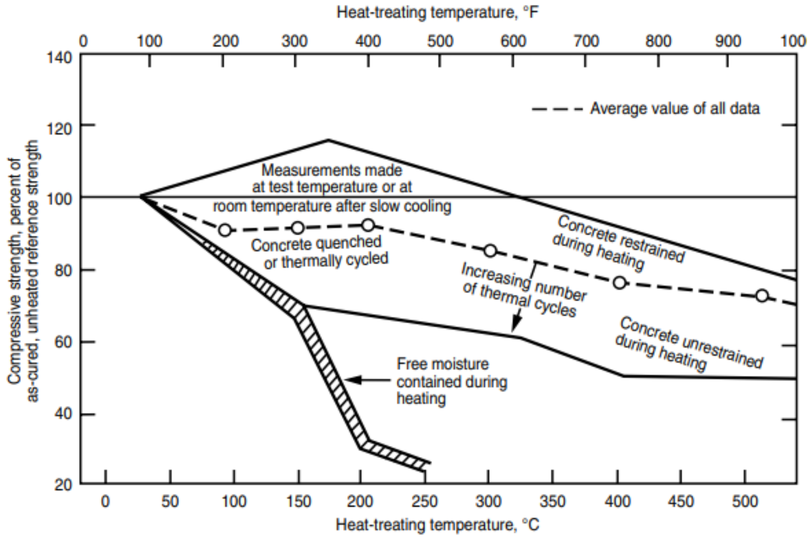


Figure 4.Temperature versus Compressive Strength [409].

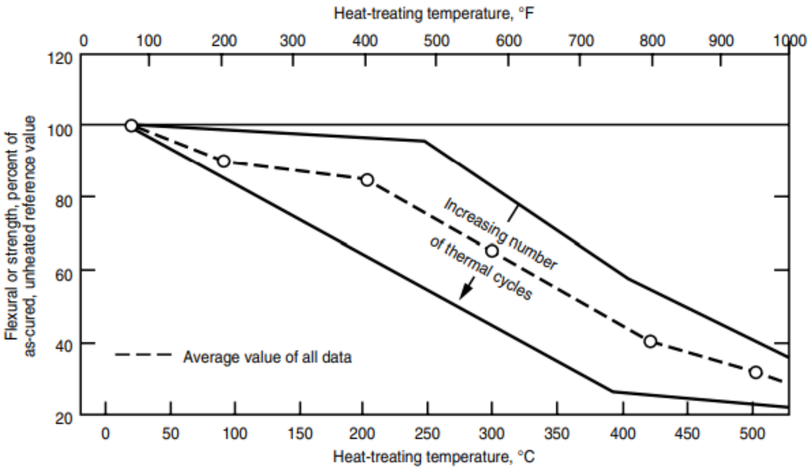


Figure 5.Temperature versus Flexural strength [409].

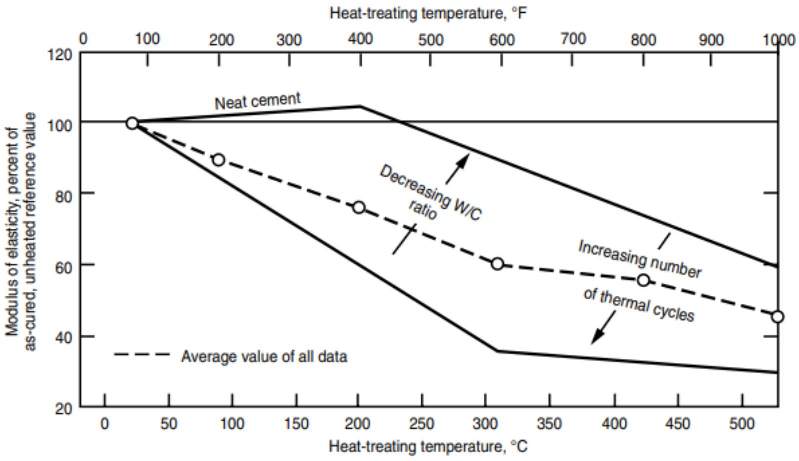


Figure 6.Temperature versus Modulus of elasticity [409].

4.9.5 Impact on concrete properties

Chemical – Pore water chemistry The pore water chemistry already changes at temperatures below 150°C due to changing solubility of phases. In addition, it is possible the composition of the pore water changes also at higher temperatures due to the disintegration of certain chemical components as explained before.

Chemical – Solid phase composition and microstructure Deterioration of concrete exposed to high temperatures is attributed to three factors: physicochemical changes in the cement paste, in its aggregates, and the thermal incompatibility between them. Other factors, such as temperature and heating rate, and structural element conditions, such as applied load and humidity, also play a role in concrete deterioration. However, only the physicochemical changes will occur in concrete exposed to temperatures up to 150°C [414].

Structural – Cracking At high temperatures, the concrete loses its moisture content by evaporation from the surface and by migration into the inner concrete mass driven by the temperature gradient. As a result of the thermal stresses, cracks will occur in the structure. Cracks will allow heat and moisture to ingress into the structure and to create damage in the inner part of the structure.

Transport properties – Porosity, permeability, diffusion coefficients The higher the porosity and permeability of the concrete, the greater the influence of high temperatures will be on the structure.

Mechanical properties – Pooja *et al.* [406] reported that between 20°C and 120°C, the compressive strength values of concrete decrease by 20 to 30% as shown in Figure 4. This decrease in strength can be attributed to:

- Stresses induced by the temperature gradient.
- The reduction of the cohesive forces between the C-S-H gel layers. When water adsorbed on the layers expands the distance between the layers also grows, causing the reduction of the Van der Waals forces. The weaker bonds between hydrates can cause microdefects, facilitating sliding of the layers.
- The build-up of internal pore pressure in the temperature range where water evaporates, which induces significant internal stresses exerted on the solid skeleton [406].

Lim [410] describes the phenomenon whereby the strength of the concrete increases at the beginning of heating, this is also visible in Figure 4. This phenomenon can be attributed to the dewatering of the samples, the activation of the cement hydration, and the formation of new C-S-H phases because of the hydrothermal reactions between the Ca(OH)2 in the cement paste and the SiO2 of the fine siliceous aggregates. This process is time dependent which explains the increase in compressive strength with the time of exposure [403].

4.9.6 Assessment methods

4.9.6.1 Visual inspection

Visual inspections are particularly useful to perform on concrete that is cyclically exposed to high temperatures. By periodically conducting a visual inspection to identify cracks and damages, all observed defects can be registered. Based on the documented damages, the evolution of the defects can be monitored and a decision to on-time repair can be made [404].

4.9.6.2 Continuous monitoring

Several parameters can be monitored in order to get a good picture of the current state of the concrete structure exposed to elevated and high temperature. Examples of the parameters that can be monitored are:

- Monitoring of cracks: A concrete structure that is exposed to elevated temperatures can be monitored using crack detection sensors, such as strain gauges and fibre optics systems. By monitoring cracking an overall picture of the condition of the concrete can be obtained [415].
- Monitoring of humidity of the concrete: this can be done by also using specialized sensors. Based on the received data an estimate can be made of the possible damage and its evolution [416].
- Temperature monitoring: By using temperature sensors, it is possible to determine exactly what temperatures the concrete surface as well as the internal concrete was exposed to. An estimate can be made of the possible damage.

4.9.6.3 Destructive testing of sampling

- To determine the residual strength of the structure after exposure to temperatures up to 150°C, cores can be extracted from the exposed structure according to EN 12504-

1. The extracted cores can be tested to determine their compressive strength according to EN 12390-3.
- To determine the extent of cracking, a core can be drilled at the location of a crack. By performing a visual inspection on the core, it can be determined how far the crack extends into the concrete.

4.9.6.4 Non-destructive techniques

- Ultrasonic Pulse Velocity (UPV) method. The UPV consists of measuring the transit time of an ultrasonic pulse through the material. UPV is used to assess variations in the apparent strength of concrete, the presence of voids, honeycombing or other discontinuities. The velocity of a pulse of ultrasonic energy in concrete is influenced by the elastic stiffness and mechanical strength of the concrete [417].

- The hammer sounding test can be applied to detect weaker parts in the concrete structure according to EN 12504-2. This way an overall picture can be obtained of the general condition of the structure exposed to high temperatures.

4.9.7 Performance indicators & acceptance criteria

In general, the performance of concrete is determined by measuring its stiffness or strength. Since concrete has a relatively low tensile strength, it is normally relied upon to take compressive forces, with tensile forces taken by steel reinforcement. As a consequence, much of the research conducted on concrete at elevated temperature has concentrated on compressive strength as the fundamental property in examining its deterioration. However, it has been noted that the compressive strength may not be as good an indicator of deterioration at elevated temperature as tensile or flexural strength under short-term loading [418].

4.9.8 Model approaches

The response of concrete under high temperature (and pressure) is highly non-linear. The factors that influence its behaviour are also non-linear. Hence, a large number of parameters are required for modelling the behaviour of concrete under a thermal load.

There are models for pressure build up/stress state, mechanical behaviour and damage evolution when concrete, or any porous material, is heated, including models for rapidly or slowly heated concrete, fire exposed concrete, etc.

• Computational numerical analysis where simulation with mathematical models is based on a semi-empirical relationship between the parameters that must be explicitly stated for computation:

- Li and Liu [419] – Coupled thermo-hygro-mechanical damage model for concrete subjected to high temperature.

- Ichikawa and England [420]– Prediction of moisture migration and pore pressure build-up in concrete at high temperatures.

• Mesoscale models (Figure 7):

Concrete is treated as a composite material where each component (cement paste, aggregates and ITZ), is considered as a multiphase system where the voids of the skeleton are partly filled with liquid and partly with a gas phase [421].

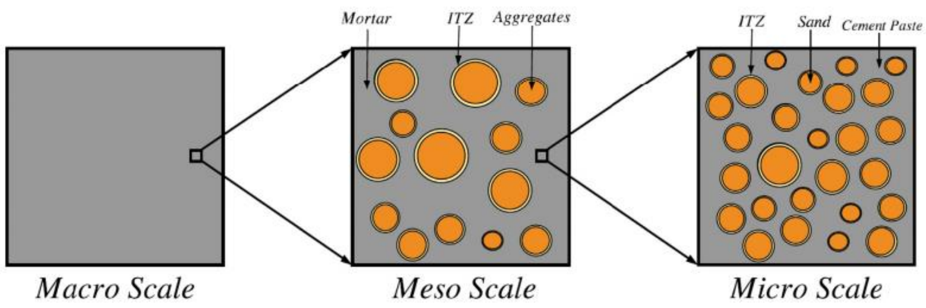


Figure 7. Different scales of concrete composition.

• Artificial neural networks (ANN):

In these models, the input of difficult-to-measure parameters can be avoided. The ANN has ability to learn complex relationships but must be tested against reliable experimental observations. For example:

- Uniaxial stress–strain behaviour of concrete [422].
- Pumice aggregate concrete properties at elevated temperatures (with high R2 for all mechanical properties) [423].
- Thermal conductivity of concrete with vermiculite aggregate [424].

4.10 Irradiation

4.10.1 Process Definition

Ionizing irradiation occurs in the form of high-energy subatomic particles (alpha, beta, neutron) and electromagnetic waves (X-ray and gamma rays). In light water reactors, the concrete biological shield (CBS), also called primary shield, is subject to the neutron and gamma irradiation emitted from the reactor pressure vessel (RPV). Collisions between high-energy neutrons (E > 10 keV [425]) and highly crystalline rock-forming minerals (or poorly crystalline \cement paste hydrates, although the consequences in terms of gradual disordering are radically different) result in knock-off atoms and the subsequent formation of complex damage cascades (secondary knock-offs), ending by means of a nuclear stopping mechanism. The gradual disordering causes in the amorphization, also known as metamictization, of the aggregates-forming minerals. The main effects of amorphization are a change of density (up to ~18% volumetric expansion in quartz), the “isotropization” of the physical and mechanical properties (e.g., stiffness tensor and coefficient of thermal expansion) and the chemical durability (higher dissolution rate in alkaline solution). The mismatch strain between expending minerals leads to additional volume formation and loss of mechanical properties of the aggregates. The radiation-induced volumetric expansion (RIVE) of the aggregates causes important elastic energy in the cement paste that is release either through cracking or relaxation. Overall, the neutron irradiation for fluence levels higher than ~10^{19} n/cm^2 (E > 0.1 MeV) results in the loss of mechanical properties. Gamma-irradiation mainly causes hydrolysis (e.g., hydrogen release) and contributes to the dehydration of the concrete.

References: For further references, the reader can consult [425-427].

4.10.2 Influential factors

Reference: Syntheses on influential factors are given in Ma *et al.* [428] and Maruyama and Muto [429].

4.10.2.1 General conditions

Effect of moisture content and relative humidity The moisture content affects the irradiation transport properties mainly governed to the high cross-section of hydrogen in natural-aggregate concrete, the hydrolysis rate (hydrogen rate production). The moisture content in the CBS depends on the reactor's design. In absence of a metallic liner, the reactor cavity is generally vented causing drying of the concrete and thus the deeper penetration of neutrons.

Effect of temperature The temperature in the indoor CBS is stable in operation and during shutdowns. By design, the CBS temperature is limited to 65°C with local exception up to ~95°C (e.g., near the RPV legs). The temperature radial profile is nonlinear because of the irradiation-induced heating. Increased annealing of neutron-induced defects reduces the RIVE rate. Higher temperature increases the cement paste relaxation.

Effect of microbiological reactions Not applicable. Indoor structure.

Effect of exposure condition (free, sheltered, buried, submerged) Not applicable. Indoor structure.

Effect of external water composition (submerged or buried structures) Not applicable. Indoor structure.

Effect of atmospheric conditions (rain, wind, freeze/thaw) Not applicable. Indoor structure.

Effect of mechanical and structural loading The structural function of the RPV supporting structure is to transfer the static or seismic load of the reactor to the foundation system. In service, the permanent loading on the CBS is minimal. However, the radiation fields exhibit important spatial variation: The areas directly exposed to the irradiation emitted by the reactor vessel are subject to irradiation levels several orders of magnitude higher than in the unexposed areas. Hence, the RIVE amplitudes varies spatially causing structural restraints. The exposed part of the CBS portion is subject to biaxial compression in the vertical and azimuthal directions while the radial direction is subject to tension.

Specific conditions: Reactor design and operation The neutron spectrum and flux vary with the reactor design. The high-energy neutron flux in boiling water reactors (BWRs) is much lower than in pressurized water reactors (PWRs) and water-water energetic reactors (VVERs). The cumulated fluence at the end of life of BWRs remains below the ~1019 n/cm2 (E > 0.1 MeV) threshold.

4.10.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM)

Cement-water binder ratio

Concrete composition (aggregate content, types of aggregates) Irradiation-induced damage is greatly influenced by the mineralogy of the aggregates. Rock-forming minerals exhibit a varied dimensional change that is a function of the doses, irradiation temperatures, and their mineralogical structures. At comparable irradiation exposures, carbonated minerals such as calcite $CaCO_3$ or dolomite $CaMg(CO_3)_2$ show less expansion at < 1% than silicates such as quartz SiO_2 or feldspars ($KAlSi_3O_8$-$NaAlSi_3O_8$-$CaAl_2Si_2O_8$) [430]. Silicates' RIVE susceptibility is mostly influenced by the ratio of covalent / ionic bonds. The maximum neutron-irradiation-induced RIVE observed from test reactors data [425] is ~18% for quartz (framework silicate containing only covalent bonds), ~7-8% for feldspars (Na-K-Ca bearing framework silicates), ~3% for pyroxenes (Mg-Fe-Ca-Na bearing mono-chain island silicates), and ~1.5% for hornblendes (Mg-Fe-Al-Ca-Na bearing double-chain island silicates). Due to their strong structural anisotropy, the irradiation tolerance of phyllosilicates is highly dependent on the orientation of the radiation direction relatively to the basal plane [428]. Phyllosilicates appear to be ranked according to their irradiation tolerance (from the highest to the lowest): chlorite, mica, pyrophyllite, smectite, kaolin [428, 429]. In metallic oxides like corundum Al2O3 or periclase MgO, RIVE appears to fall in an intermediate case [431].

Curing (temperature, moisture, time) conditions

4.10.3 Influence of other ageing processes

4.10.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) Not applicable

Leaching (submerged or buried structures) Not applicable

Alkali-aggregate reactions The mechanistic synergies between irradiation and alkali-silica reaction are still poorly understood. While it has been established that neutron-irradiation enhanced the dissolution rate of rock-forming silicates [174, 430], the stability of hydrophilic ASR-like gel under gamma-irradiation is not known. Conversely, ASR-susceptible aggregates contain poorly-/crypto-crystallized silicates that should have higher irradiation tolerance than their pristine crystalline forms. When aggregates are made of both poorly and well crystallized silicates, the synergetic effects need to be studied.

Carbonation Gamma-irradiation enhances the formation of carbonated polymorphic products (calcite, vaterite)

Ettringite and thaumasite reactions (DEF/sulphate attack) No supporting data

Bacterial processes Not applicable

4.10.3.2 Physical-Mechanical

Freeze-thaw Not applicable

Elevated and high temperature (<150°C, no fire) The CBS is subject to moderate temperature in operation in the range of 40-70°C contributing to the dehydration of the cement paste.

4.10.3.3 Mechanical

Abrasion/Erosion/Cavitation Not applicable

Creep and relaxation Aggregates' RIVE causes important mechanical stresses in the cement paste which can be relaxed through the cement paste's viscoelastic properties. The importance of this mechanism has only been explored by numerical models. Limited data on irradiated creep are available in the public literature. Under neutron irradiation, creep kinetics at 60-95°C appears to be one order of magnitude higher than the creep kinetics of specimens out of pile before or after irradiation under a similar range of temperatures [432]. Conversely, neutron- and gamma-induced radiolytic effects on water being similar in nature [433], the creep rate and compliance decrease [434].This indicates the possibility of a strong interaction between neutron and creep mechanism, even though experimental data support that neutron irradiation has a limited effect on the macroscopic mechanical properties of cement paste [432, 435].

Settlements and movements Not applicable

Vibration (and seismic) Vibration: not applicable; Seismic action: absence of data.

Thermal stresses (gradients) The average temperature gradient in the CBS is in the order of 0.3°C/m. However, local temperature spikes may cause important thermal stresses [436].

4.10.3.4 Electro-chemical

The synergetic effects of irradiation between irradiation and corrosion have not been demonstrated to date despite the radiolysis-induced formation of oxides.

4.10.4 Rates of deterioration

By lack of studies on concrete specimens harvested from LWRs' CBS, there is currently no data on the rate of degradation in-service. Data obtained in test reactors (flux higher by two or three orders of magnitude) shows deterioration at fluence higher than ~10^{19} n/cm^2 (E > 0.1 MeV). Important variations of the degradation rate are observed with the chemical composition of the aggregates. Siliceous aggregates exhibit the highest loss of mechanical properties. At ~10^{20} n/cm^2 (E > 0.1 MeV), the compression strength and Young's modulus can drop by ~50%, the tensile strength can decrease by ~75% [427].

4.10.5 Impact on concrete properties

Chemical – Pore water chemistry – *No information*.

Chemical – Solid phase composition – No modification of the chemical composition of the mineral phases.

Structural – Microstructure –Amorphization of the aggregates' minerals and moderate change of the porosity.

Structural – Cracking – RIVE-induced cracking in the aggregates and the cement paste.

Transport properties – Porosity, permeability, diffusion coefficients No information.

Mechanical properties – Gamma-irradiation does not significant reduction of concrete structural properties. Neutron-irradiation causes a gradual reduction of the mechanical properties (elastic properties and strength) for fluence higher than ~10^{19} n/cm^2 (E > 0.1 MeV). Post-irradiation properties vary with the aggregates' chemical composition.

4.10.6 Assessment methods

4.10.6.1 Visual inspection

The reactor cavity is inaccessible to visual inspection unless the RPV is removed.

4.10.6.2 Continuous monitoring

While surveillance capsules (RPV steel) are periodically removed from the reactor to assess the mechanical performance of the RPV, expect rare occurrences, there is generally no concrete specimens placed in comparable irradiation conditions in the reactor cavity.

4.10.6.3 Destructive testing of sampling

Testing
Most available data on irradiated concrete have been obtained under accelerated conditions in test reactors.

Intact cores
While cored specimens were harvested from decommissioned NPPs (e.g., Zorita, Hamaoka), there are none to date at high level of fluences.

Disturbed sampling (e.g. dust)
The characterization of the minerals' chemical composition of the concrete constituents is critical to the assessment of the radiation tolerance of irradiated concrete. A set of characterization techniques can be employed to this aim including but not limited to X-ray diffraction, Energy-dispersive X-ray spectroscopy, micro-X-ray fluorescence, petrography, Raman spectroscopy.

4.10.6.4 Non-destructive techniques

No relevant data.

4.10.7 Performance indicators & acceptance criteria

The research on irradiated concrete is still limited. Major knowledge gaps exist and need to be addressed before adopting adequate performance indicators and acceptance criteria. However, three preliminary criteria can be proposed:

1. Irradiation exposure: when the fluence level remains below ~10^{19} n/cm^2 (E > 0.1 MeV), irradiation damage is unlikely.
2. Concrete composition: Carbonated aggregates concrete containing only traces of silicates minerals are unlikely to develop significant irradiation damage.
3. Structural performance: When exposure and chemical composition both exceed the abovementioned conditions, an acceptance criterion can be defined by the penetration of irradiation damage not exceeding the rebar cover.

4.10.8 Model approaches

4.10.8.1 Empirical models

Radiation-induced amorphization is a complex process occurring at the atomistic scale. Although it can be addressed through molecular dynamics simulation, minerals RIVE's empirical models provide a more rapid assessment of the expansion under neutron-irradiation [425, 431, 437].

At the scale of concrete, literature data have been collected in [427] and can be used as a first estimate of the RIVE and loss of mechanical properties [438]. These empirical models require cautious use because of the varied uncertainties (see limitations in [427]).

4.10.8.2 Phenomenological models

Several models based on mean-field homogenization theory are available in the literature [439, 440]. These models can effectively estimate the concrete RIVE and provide a limited assessment of the loss of mechanical properties, The main inconvenient of these models to date is that they require the aggregates RIVE as an input. This issue has been partly addressed in [441] using aggregates' polycrystalline homogenization models and in [442] using lattice-based polycrystals simulations.

At the structural level, a 1D-model has been developed to assess the irradiation-induced damage through the depth of the CBS at the beltline level (highest flux). This model uses the irradiated concrete database developed by Field (see limitations) and does not account for the effects of structural-restraints-induced damage.

4.10.8.3 Complex coupled models

Varied advanced numerical models of irradiated concrete are currently being developed in European, North American and Asian institutions using varied techniques: finite element models [443], lattice-based models [444] and Fast Fourier Transform-based models [445]. These mesoscale models describe concrete either as a two-phase material (cement paste and aggregates) or as a multi-minerals phase material (accounting for the mineralogy of the aggregate).

At the structural level, there is currently no fully coupled models for irradiation transport, coupled heat-transfer and moisture transport, and structural analysis. Several irradiation transports are available: e.g., MCNP, Serpent, VERA-Shift. Several codes are also available for coupled heat-transfer and moisture transport and structural analysis including but not limited to GRIZZLY, Cast3m and other in-house codes.

4.11 Abrasion, erosion, cavitation

4.11.1 Process Definition

Abrasion of concrete is progressive loss of concrete mass due to mechanical degradation such as friction, grinding action, impact, overloading and local crushing. Vehicular movement and pedestrian traffic causes abrasion. The worst effect of abrasion is caused by vehicle with studded/chain tyres. Similarly, in industrial buildings, the concrete floors are subjected to impact load and scratching effects.

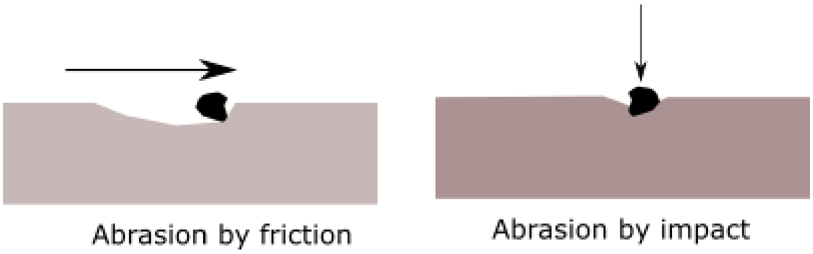


Figure 8. Main two kinds of abrasion.

References: For further references, the reader can consult Dhir *et al.* [446] and Scott and Safiuddin [447].

4.11.2 Influential factors

The abrasion resistance of concrete depends upon the paste hardness, aggregate hardness and the bonding between paste and aggregate.

In general, the higher the compression strength of concrete, the higher will be its abrasion resistance (Figure 9). But strength alone cannot be taken as the parameter to estimate the abrasive behaviour.

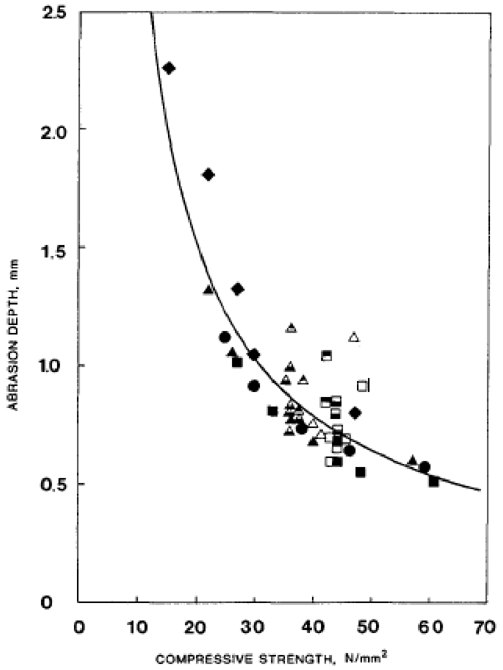


Figure 9. Relation between abrasion depth and compressive strength.

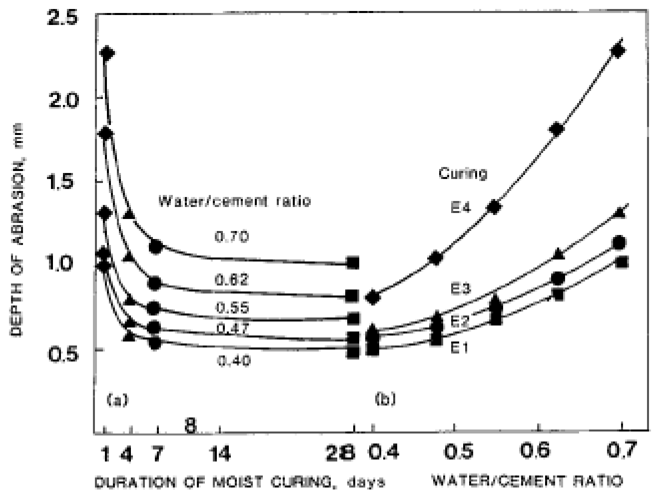


Figure 10. Effect of w/c ratio and duration of moist curing on abrasion depth.

In an extensive study made by Dhir *et al.* [446], the size of aggregate and curing time were observed to be dominant variables besides the compression strength (or w/c ratio). It was observed that the size of aggregate plays an important role for rate of abrasion (mainly in the case of impact loading). Due to higher stiffness of smaller aggregates, they are less prone to damage in impact loading, however, it is unclear why very small particles could be damaged easily.

Reference: Syntheses on influential factors are given in Dhir *et al.* [446] and Scott and Safiuddin [447].

4.11.2.1 General conditions

Effect of moisture content and relative humidity There is no relevant information about the effect of moisture on abrasion, erosion or cavitation of concrete.

Effect of temperature There is no relevant information about the effect of temperature on abrasion of concrete

Effect of irradiation There is no relevant information about the effect of temperature on abrasion of concrete

Effect of microbiological reactions There is no relevant information about the effect of temperature on abrasion of concrete

Effect of exposure condition (free, sheltered, buried, submerged) The construction can be free to be affected by wind and rain abrasion or by traffic; erosion is caused in by water covered structures

Effect of external water composition (submerged or buried structures) Not relevant for abrasion, important for erosion or cavitation [448]

Effect of atmospheric conditions (rain, wind, freeze/thaw) Rain, wind or ice are one of the factors which may cause abrasion, erosion or cavitation of concrete [449, 450]. Moreover, when surface layers are affected by freezing and thwing cycles their resistance against abrasion can be lowered [363].

Effect of mechanical and structural loading Not relevant - mechanical nor structural loading has no major effect on abrasion depth or resilience.

4.11.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) The type, amount and quality of cement and additions as silica has significant effect on abrasion depth or resilience [447].

Cement-water binder ratio The water/cement ratio and the length of curing has significant effect of abrasion depth [446, 447, 451].

Concrete composition (aggregate content, types of aggregates) Abrasion resistance of concrete depends upon the paste hardness, aggregate hardness and the bonding between paste and aggregate [446]

Curing (temperature, moisture, time) conditions – Length of moist curing as well as w/c ration has large effect of abrasion depth [446].

4.11.3 Influence of other ageing processes

4.11.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) Abrasive erosion, erosion or cavitation caused by aggressive water or acid rain hay major effect in comparison with non-polluted water [448].

Leaching (submerged or buried structures) Not relevant. Only abrasion can discover protecting surface layer and enable leaching.

Alkali-aggregate reactions Surfaces damaged by alkali-silica reaction cracks do not resist so much as non-damaged surface.

Carbonation It can corrode steel reinforcement and enable peeling of surface layers.

Ettringite and thaumasite reactions (DEF/sulphate attack) There is no relevant information

Bacterial processes There is no relevant information

4.11.3.2 Physical-Mechanical

Freeze-thaw *C*oncrete structures in Norway had experienced as much as 50 mm of abrasive wear due to ice over 20 years [449].

Elevated and high temperature (<150°C, no fire) *C*oncrete facing to abrasion, erosion or cavitation is designed also for wetting, drying or extreme temperatures [452].

Irradiation No information.

4.11.3.3 Mechanical

Creep and relaxation Not relevant

Settlements and movements Both can cause abrasion and consequent cavitation, their effect is large due to friction under extreme loading [447]

Vibration (and seismic) No relevant information.

Thermal stresses (gradients) Usually concrete areas facing to abrasion, erosion or cavitation are designed also for thermal shocks [452].

4.11.3.4 Electro-chemical

Pitting corrosion Not relevant

General corrosion Not relevant - carbonation of concrete or corrosion of its reinforcement has no known effect on abrasive properties of concrete.

Other corrosion mechanisms including crevice corrosion Corrosion of near surface layers can cause cracks and affect the resilience against abrasion or erosion of concrete [453].

4.11.4 Rates of deterioration

Most codes assess abrasion or erosion by depth of abrasion in mm or by loosing surface layer, also in mm, or in mass, e.g. in grams.

Abrasion test in concrete can be done by adopting the procedure of following codes:

- BS EN 13892-4 [454]
- ASTM C418, C779 and C944 [455-457]
- BS-8204-2 [458]

4.11.5 Impact on concrete properties

Chemical – Pore water chemistry – This can be effected by erosion or cavitation.

Chemical – Solid phase composition – Not relevant

Structural – Microstructure – Not relevant

Structural – Cracking – May occur after discovering surface layer [459].

Transport properties – Porosity, permeability, diffusion coefficients – Not relevant: abrasion, erosion or even cavitation can affect near surface properties of concrete structure.

Mechanical properties – Mechanical properties of whole structure can be affected (e.g. rigidity, load bearing capacity), nevertheless the material properties will keep the same.

4.11.6 Assessment methods

There are several methods of assessment well described by Bakke [460].

For example ASTM C779 [456] states three test methods which provide simulated abrasion conditions, which can be used to evaluate the effects on abrasion resistance of concrete, concrete materials, and curing or finishing procedures. They may also be used for quality acceptance of products and surface exposed to wear. They are not intended to provide a quantitative measurement of length of service.

The equipment used by each of these procedures is portable and thus suitable for either laboratory or field testing. The three procedures determine the relative wear of concrete surfaces as follows:

Procedure A—The revolving-disk machine operates by sliding and scuffing of steel disks in conjunction with abrasive grit (Figure 11).

Procedure B—The dressing-wheel machine operates by impact and sliding friction of steel dressing wheels.

Procedure C—The ball-bearing machine operates by high-contact stresses, impact, and sliding friction from steel balls.

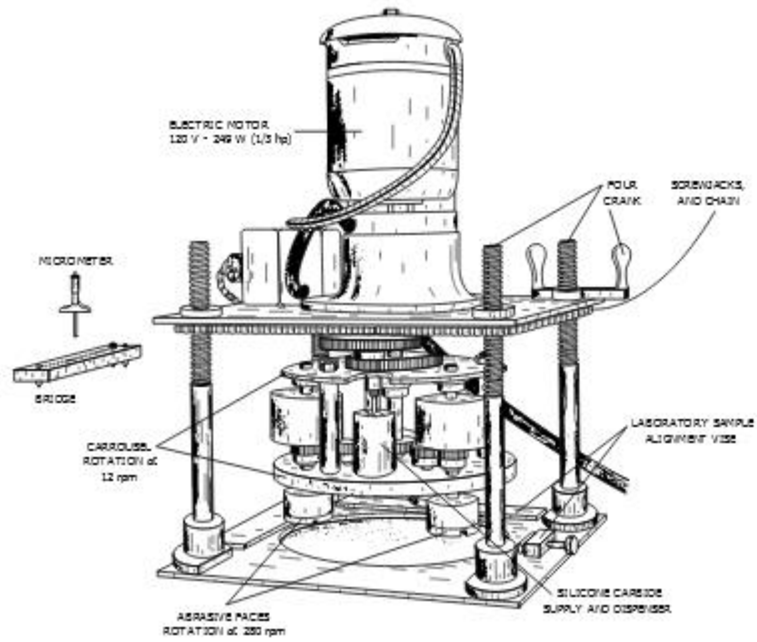


Figure 11. Abrasive test machine according to the procedure A in ASTM C779 [456].

4.11.6.1 Visual inspection

Visual inspection is dedicated to finding attacked areas. It is also possible to measure the depth, shape and diameter of abrasion, erosion or cavitation and determine their possible origin [461].

4.11.6.2 Continuous monitoring

There is no relevant information available about continuous monitoring of concrete abrasion, erosion or cavitation.

4.11.6.3 Destructive testing of sampling

There are different ways; testing is usually done by water or by different particles blasting or brushing.

Testing in situ It is possible to bring testing machine to the construction and several companies provide such service, e.g. Face Consultants Ltd, nevertheless most of the codes propose laboratory testing of effect of abrasion, e.g. BS 8204-2 [458], ASTM C779 [456]. It can be done either on samples taken from the construction or fabricated in laboratory.

Intact cores Take intact cores from the structure (colour, thin sections, …) - this is very often style of testing [462].

Disturbed sampling Not relevant.

4.11.6.4 Non-destructive techniques

This kind of testing was found as insufficiently representative, thus most of the tests have been done in a destructive way.

In older literature, few more or less successful tries of NDT have been documented. Three non-destructive test methods have been investigated to determine their suitability as indirect methods for assessing the abrasion resistance of concrete slabs [463]. The ultrasonic pulse velocity technique was found to be insufficiently sensitive to variation in the concrete mix design. The surface hardness method, using the Schmidt rebound hammer, was found to be partially sensitive to factors which are known to influence the abrasion resistance of concrete. The initial surface absorption method was found to be very sensitive to these factors and was closely related to abrasion resistance of concrete as determined by an accelerated abrasion test.

4.11.7 Performance indicators & acceptance criteria

There are various types of evaluation criteria for concrete abrasion, erosion and cavitation. All were described by Alexander [464] (Table 10). An example of limiting criteria by BS8204-2 [458] is in Table 11.

Table 10. Classification of concrete abrasion tests by type of wearing action [464].

Type of Practical Application	Light and Medium Traffic (Foot and Wheeled)	Heavy Tyre and Steel Wheel Traffic; Moving of Steel Rocks, etc.	Heavy Steel and Track Vehicles-Tyre Chains, Heavy Rocks, etc.	Hydraulic and Windblown Effects; Abrasion and Impact Erosion, Cavitation
Concrete floors Light to medium applications	BöHME ASTM C 418—C 779 (Proc. A—Proc. B) NF P 98-303 (France)			
Concrete floors Heavy applications		BöHME ASTM C 779 (Proc. B—Proc. C) ASTM C 418	BöHME ASTM C 779 (Proc. C—Proc. B)	
Concrete roads Normal traffic; clean surface	BöHME ASTM C 779 (Proc. A—Proc. B)			
Concrete roads Heavy trucking; abrasion grit on surface		ASTM C 779 (Proc. B) ASTM C 418	ASTM C 779 (Proc. B—Proc C)	
Hydraulic structures Water at low to medium velocities + abrasive medium				ASTM C 779 (Proc. A)— ASTM C 418 ASTM C 1138
Hydraulic structures Water at very high velocities				ASTM C 418

Table 11. Classification of abrasion resistance and limiting depths of wear for abrasion test: Recommendations for concrete bases, directly finished (DF) only, as wearing surfaces [458].

Class	Service conditions	Application	Maximum test wear depth mm	Typical examples (see 6.2) Type of concrete	Minimum compressive strength class[a] N/mm²	Minimum cement content kg/m³	Type of coarse aggregate	Type of A1 fine aggregate A1	Finishing process
AR0.5 (special)/DF	Severe abrasion and impact from steel or hard plastics wheeled traffic or scoring by dragged metal objects	Very heavy duty engineering workshops and very intensively used warehouses, etc.	0.05	Specially designed proprietary concretes	Special concretes which are not classified by strength class or minimum cement content and might contain aggregates that do not conform to **5.3**. Special finishing techniques may be used. The suitability of concrete flooring for this class should be established with the manufacturer or flooring contractor offering warranty				
AR1/DF	Very high abrasion; steel or hard plastics wheeled traffic and impact	Heavy duty industrial workshops, intensively used warehouses, etc.	0.1						
AR2/DF	High abrasion; steel or hard plastics wheeled traffic	Medium duty industrial and commercial	0.2	Direct finished concrete	C40/50 RC50	400	Aggregates conforming to **5.3.2**	A1 Fine aggregate A1 conforming to **5.3.3**	Power floating and repeated power trowelling as **10.7**
AR4/DF	Moderate abrasion; rubber-tyred traffic	Light duty industrial and commercial	0.4	Direct finished concrete	C32/40 RC40	325			

[a] Concrete should conform to BS 8500-2.

4.11.8 Model approaches

4.11.8.1 Empirical models

All models described below were developed in 19th and 20th century, for abrasion generally [465].

4.11.8.2 Phenomenological models

Physical models of mass evolution [466], collisional models [467], frictional models [468].

4.11.8.3 Complex coupled models

Collisional abrasion of an individual particle in constant environment was desribed by Bloore [469], Monge [470] and Kardar *et al.* [471], box equations [469], the spherical case (frictional abrasion of an individual particle; Bloore equations with friction),box equations with friction, volume weighted Bloore equations, and volume weighted box equations.

4.12 Creep and relaxation

4.12.1 Process Definition

Concrete, like a number of other materials, is subject to delayed strain phenomena, which relate to different types of strain, which can occur on very long periods of time, by opposition to instantaneous strain related to the elasticity of the material.

These delayed strains conventionally are separated, at the scale of macroscopic observation, in two groups:

- Shrinkage occurs without a macroscopic mechanical loading on the concrete sample
- Creep occurs under macroscopic mechanical loading (the word creep is taken from the field of viscoelasticity, where it refers to the strain behaviour under constant loading, by opposition to relaxation. Here, the word is used to refer to the physical observation without assuming constant loading).

Drying of concrete has a major impact on concrete delayed behaviour. Therefore, it is a convention to further categorize delayed strains depending on the occurrence of drying or not:

- In absence of drying:
    - Autogenous shrinkage refers to the free strain occurring in absence of drying after setting (the part that occurs before setting is called Le Chatelier shrinkage and is of much larger magnitude but of moderate significance since concrete is still fluid)
    - Thermal shrinkage refers to the free strain related to temperature variations, which occur after setting due to the dissipation of hydration heat. Strains also occur during the phase of temperature increase, but since concrete is still fluid, it is of little impact.
    - Basic creep refers to strain under loading when concrete is prevented from drying.
- When drying is allowed:

- Desiccation or drying shrinkage is the free strain related to the saturation change of concrete.
- Desiccation or drying creep is the strain under loading occurring when concrete dries.

As an illustration to this classification of deformations, see Figure 12 that is proposed by Neville *et al.* [472].

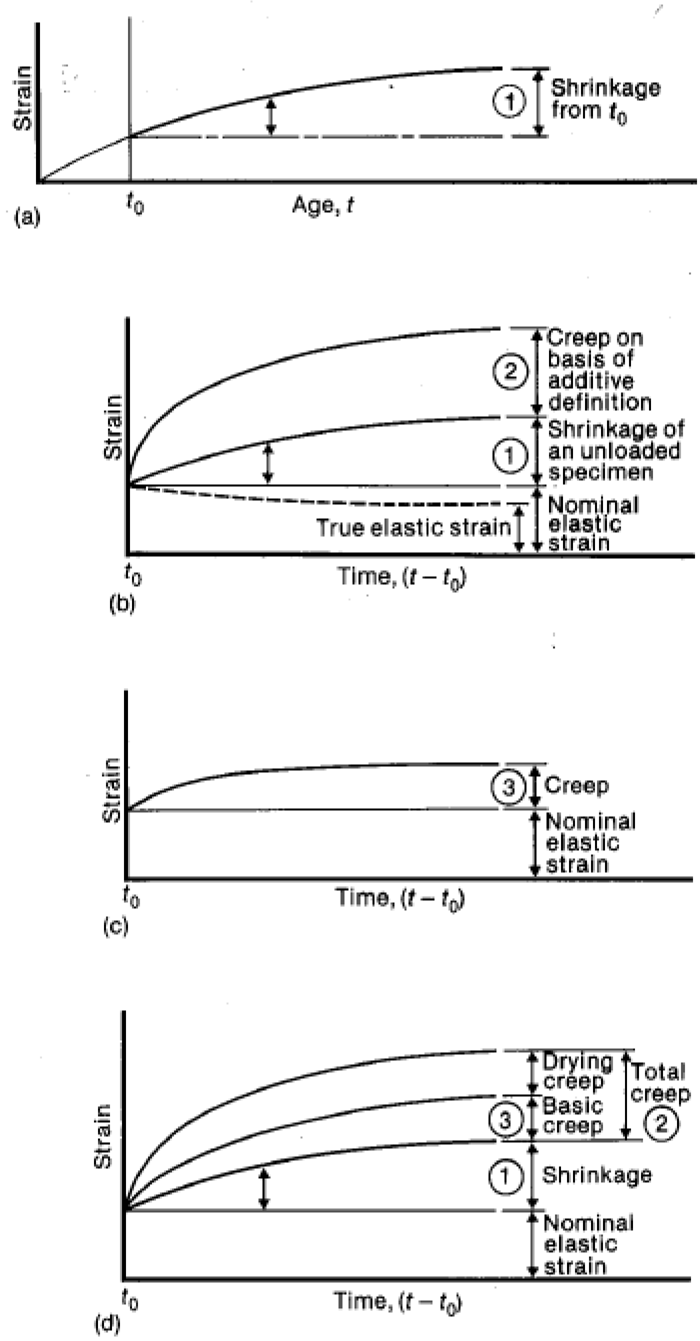


Figure 12. Classification of deformations [472]. t_0 is the age of the sample when drying starts and loading is applied. (a) Shrinkage of an unloaded companion specimen. (b) Change in strain of a loaded and drying specimen. (c) Creep of a loaded specimen in hygral equilibrium with the ambient medium. (d) Change in strain of a loaded and drying specimen.

Generally, there is a risk of confusion with these terms, since they are used differently by experimentalists (where they tend to refer to the results of the tests in different conditions:

drying or not, loaded or not) and the modelling community (where they tend to refer to terms of the models). In this process description, we will not enter into these details.

In this document, creep can be understood as creep under tension or creep under compression.

The origin of shrinkage and creep lies mostly in the cement paste. In this review, the detailed physical phenomena will not be described. These phenomena are still under discussion in the scientific community.

References: For further references, the reader can consult [472, 473].

4.12.2 Influential factors

Reference: Syntheses on influential factors are given in Neville *et al.* [472].

4.12.2.1 General conditions

Effect of moisture content and relative humidity – Moisture content has a direct effect on drying shrinkage: when the saturation degree decreases, different physical phenomena (capillary effect, disjunction pressure, and surface tension) induce traction on the surface of the solid skeleton which is therefore put under compression, inducing macroscopic shrinkage [474] It also has an effect on creep: when concrete dries at the same time as it is under load, the observed strain is more important than the sum of free strain under the same drying condition (drying shrinkage) and creep under constant humidity (basic creep). This effect is called Pickett effect and its physical origin is still not completely understood [475]. Moisture also has a direct effect on basic creep: creep at constant humidity is lower if humidity is lower [476].

Effect of temperature Creep and shrinkage are accelerated when temperature is increased [472], both if drying is prevented (basic creep) and also if drying is allowed (desiccation shrinkage and desiccation creep). In the case where drying is allowed, a significant part of this kinetics increase is due to the acceleration of drying at higher temperature. When heating is applied shortly after loading, additional strains occur compared to the case where heating is applied and equilibrated before loading. These additional strains are called thermal transient strains [477].

Effect of irradiation Irradiation does not directly affect creep since it mostly affects aggregates, while creep occurs in the cement paste. However, creep and irradiation can both induce cracking, and therefore, the mechanical consequences of creep and irradiation are not independent [478]. However, this is not of very big practical relevance since in NPP, the structures which are subject to high stresses and hence creep (containment buildings) are not subject to high levels of irradiation.

Effect of microbiological reactions Not relevant.

Effect of exposure condition (free, sheltered, buried, submerged) Exposure conditions influence creep and shrinkage through their influence on concrete internal moisture.

Effect of external water composition (submerged or buried structures) If the external water induces dissolution of some of the phases, it can induce changes in the stress repartition inside concrete and promote creep and shrinkage. This effect has been studied for example in [53] and is similar to what can be observed with carbonation shrinkage (carbonation promotes portlandite dissolution, and portlandite does not shrink nor creep; therefore, its dissolution can promote these phenomena at a macroscopic level).

Effect of atmospheric conditions (rain, wind, freeze/thaw) Rain and wind have an effect on creep and shrinkage since they affect the internal water content of concrete. Freezing does not specifically affect creep and shrinkage, except for the fact that these phenomena are slower at lower temperatures.

Effect of mechanical and structural loading Loading has a large influence on creep from at least two points of view:

- Age of concrete at the instant of loading: since concrete continues to hydrate slowly even after setting, its microstructure changes and becomes more compact with time. Therefore, creep (both basic creep and desiccation creep) are of lesser magnitude when the age at loading is larger.

- Magnitude of loading: creep is considered linear up to about 40% of the concrete strength. Above this level of loading, non-linear behaviour appears (i.e. the magnitude of the strain response is no longer proportional to the applied stress).

Another point which has been studied in the literature is the multiaxial creep behaviour. In recent works it is generally considered that creep under loadings smaller that 40% of the strength is isotropic and linear, and that the creep Poisson's ratio is constant [479].

4.12.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) The use of different kinds of supplementary materials has an influence on the proportion of different hydrates (the pozzolanic effect consumes portlandite), on concrete compactness, on strength, on water content. Hence, it also has an effect on creep and shrinkage, which can be partly related to their effect on strength and drying [480, 481].

Cement water-binder ratio – For a given type of concrete constituents, the lower the w/c ratio, the higher the strength. Creep magnitude is highly influence by concrete strength (which is usually used as a parameter in regulatory codes, see for example [482]. The broad picture is that a higher strength is associated with a lower creep.

Mix design properties (aggregate content, type of aggregates, …) – Aggregate content has a simple influence: the higher it is, the lower creep and shrinkage. The type of aggregate has a large influence as well: concretes with identical mixes can creep very differently if different aggregates are used [483].

Curing (temperature, moisture, time) conditions – As hydration progresses, the magnitude of drying shrinkage, basic creep and drying creep decrease. It is partly related to the "age at loading" effect mentioned earlier, and also to the fact that if drying starts at a later age, less water is available and the effects of drying are hence diminished. Therefore, the parameters that influence hydration rate and quality of curing also have an effect on creep and shrinkage.

4.12.3 Influence of other ageing processes

4.12.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) Can induce stress redistribution in concrete, so can influence creep and shrinkage (at the surface of concrete).

Leaching (submerged or buried structures) Can induce stress redistribution in concrete, so can influence creep and shrinkage (at the surface of concrete).

Alkali-aggregate reactions Gel swelling inside the pores of concrete during AAG induce local stress concentrations, which are partially redistributed or released due to relaxation of cement paste. At a macroscopic level, if swelling is restricted by boundary conditions on the structure, macroscopic stresses appear and also induce creep.

Carbonation Carbonation can induce shrinkage. Shrinkage experiments on small samples must be protected from carbonation, otherwise carbonation shrinkage can occur and prevent from measuring drying shrinkage.

Ettringite and thaumasite reactions (DEF/sulphate attack) The link with creep is the same as for AAR.

Bacterial processes If bacteria induce leaching, then they can have an effect on creep and shrinkage (however, it would be a surface effect, not very relevant when looking at the structure globally).

4.12.3.2 Physical-Mechanical

Freeze-thaw Not relevant since restricted to the surface

Elevated and high temperature (<150°C, no fire) Temperature accelerate drying, creep and shrinkage.

Irradiation No direct effect on creep and shrinkage, but coupled consequences on microcracking are possible.

4.12.3.3 Mechanical

Effects of (surface) cracks Cracking has no particular effect on creep and shrinkage.

Abrasion/Erosion/Cavitation Not relevant

Settlements and movements Creep can contribute to release stress concentrations due to settlements.

Vibration (and seismic) Not relevant

Thermal stresses (gradients) Relaxation can contribute to release stress related to thermal shrinkage.

4.12.3.4 Electro-chemical

Pitting corrosion Not relevant

General corrosion Not relevant

Other corrosion mechanisms including crevice corrosion Not relevant

4.12.4 Rates of deterioration

The order of magnitude of desiccation shrinkage for a regular concrete when drying is equilibrated at about 50% RH is 500 to 1000 microstrain.

Creep can induce in time stresses from 2 to 5 times that of elasticity (for the same applied stresses) depending on the quality of concrete, ambient conditions.

The simplest way to have a synthetic view on the order of magnitude of the different phenomena is though the formula proposed by codes such as [482].

4.12.5 Impact on concrete properties

Chemical – Pore water chemistry Not relevant

Chemical – Solid phase composition Not relevant

Structural – Microstructure Concrete creep and shrinkage can induce cracking at different levels. First, there is usually surface cracking due to drying shrinkage. At smaller scales, it is suspected that cracking occurs as a consequence of shrinkage and creep contrasts between different phases, but this is still an open research field.

Structural – Cracking Thermal shrinkage, which occurs at early age after setting during the temperature decrease, can induce significant structural cracking when restrained due to shrinkage gradients.

Transport properties – Porosity, permeability, diffusion coefficients Not relevant

Mechanical properties It is not known whether creep and shrinkage induce a change in mechanical properties. If this phenomenon exists, it is probably not of very large magnitude.

4.12.6 Assessment methods

General methodologies can be found in [2, 484].

4.12.6.1 Visual inspection

Visual inspection can be used to assess the spacing and opening of cracks formed by thermal shrinkage. For drying creep and shrinkage, it is not relevant (except in some extreme cases on bridges where creep deflexion can be observed visually).

4.12.6.2 Continuous monitoring

Creep and shrinkage are assessed by measuring concrete strain or displacements in different conditions.

- For field structures, this can be done using different kinds of embedded or surface sensors:
- Embedded vibrating wire sensors (strain measurement, mature, used since 30’s)
- Penduli and invar wires (differential displacement measurement, mature)
- Surface linear variable differential transformer (LVDT) (differential displacement measurement, mature)
- Embedded or surface optic fibers (strain measurement, mature since years 2010)
- Embedded or surface coaxial cables (strain measurement, should be mature shortly)

For characterization in the lab, different kinds of sensors can be used:

- Electric strain gages (mature but not easy to use in contact with concrete)
- LVDT (mature)
- Vibrating wires (depreciated)
- Optic fibers (mature)
- Digital image correlation (mature)

4.12.6.3 Destructive testing of sampling

Not relevant

4.12.6.4 Non-destructive techniques

Not relevant

4.12.7 Performance indicators & acceptance criteria

Not relevant. Totally depends on the use of the structure.

4.12.8 Model approaches

4.12.8.1 Empirical models

In France, concrete containment buildings of operating nuclear power plants were designed using [485]. A number of more modern regulatory codes have been proposed since then, e.g. EC2 [482]. These models propose simple empirical formula with parameters, which have been calibrated on large databases. Their precision for a given concrete mix is moderate, but they are very useful when no testing on a given formula is available.

4.12.8.2 Phenomenological models

A large number of models have been developed to account for the physical phenomena observed in the lab. For desiccation shrinkage, see [486]. For creep, different physical models have been proposed to account for observations on the dependence of creep on moisture [487], or temperature [477].

4.12.8.3 Complex coupled models

On top of models which are mostly used to understand the behaviour of concrete, finite element models aiming at reproducing or predicting the behaviour of full-scale structures have been developed. The main models on the international scene are the MPS (micro pre-stress) model developed by Northwestern University in the US and CTU in Czech Republic [473], the model developed at LMDC Toulouse, France [488]. EDF also has its own model and uses it for its nuclear power plants [489]. These models are constantly evolving. A series of benchmarks is organised by EDF on the VERCORS mock-up with many participants using different models [490].

4.13 Settlements and movements

4.13.1 Process Definition

Concrete foundation of structures serves as the interface between structures and the soil or rock and, as such, it is one of the most important and most difficult to predict when designed. The settlement of foundation is defined as a vertical downward movement. For majority of the nuclear facilities, the settlement is not of significant importance as the location of erection of such installations is selected carefully especially with regard to favourable geotechnical conditions with the emphasis on long-term stability. Therefore, the nuclear installations are mainly located on firm rock or other non-cohesive material which ensures that the settlement occurs only during construction or soon afterward, which may be even before the installation is set to operation. However, for those installations which are sited on loose but compacted soils, the settlement needs to be assessed carefully in the foundation design and necessary monitoring processes need to be adopted later so that it is confirmed well in advance during operation of the structure that the allowable limits are not exceeded.

The settlements may be uniform or uneven/differential. The uniform settlement is of less concern as it does not cause any additional distress for structures as such. The issues related to the uniform settlement are related to alignment of large structures or their segments as they may become vertically displaced and as a result, for example, the piping or other long equipment may become misaligned or damaged. The uneven, or differential, settlement is mainly caused by significant differences in the subsoil conditions under the foundation which was not adequately designed. The consequences may be very serious as the unaccounted-for stresses may exceed the tensile strength of materials and cause cracks, or the differential settlement may cause tilting of the affected structure.

The settlement is mainly irreversible, however, its progress over time, when identified early, may be stopped or controlled.

Reference: For further references, the reader can consult Naus *et al.* [491], NRC [492], and IAEA [2].

4.13.2 Influential factors

Reference: Syntheses on influential factors are given in Naus *et al.* [491], NRC [492], and IAEA [2].

4.13.2.1 General conditions

Effect of moisture content and relative humidity – The moisture content and relative humidity are related to the subsoil when the changes may be caused by fluctuation of the underground water table and removal or planting of vegetation. In the case of large nuclear installation, the underground water table change does not affect the structures as the buoyancy force does not exceed the self-weight of the structures. The only case of significant effect of moisture content may be the case when permafrost thaws and the foundation design did not reflect this possibility. All these may affect the moisture content on concrete foundation but without significant consequences.

Effect of temperature Not relevant

Effect of irradiation Not relevant

Effect of microbiological reactions Not relevant

Effect of exposure condition (free, sheltered, buried, submerged) The concrete foundations are mostly buried underground.

Effect of external water composition (submerged or buried structures) The quality and chemical composition of the underground water may affect durability of concrete foundation structures. In particular, the presence of sulfates or chlorides must be taken into account.

Effect of atmospheric conditions (rain, wind, freeze/thaw) Due to the freeze/thawing, the concrete foundation are places in sufficient depth where the freeze/thawing does not occur.

Effect of mechanical and structural loading This is the typical effect on concrete foundations as they are meant to transfer the load from the superstructure to the subsoil.

4.13.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) The cement type is typically the Portland cement or blended cements for better chemical stability.

Cement-water binder ratio Typical, i.e. 0.4-0.7.

Concrete composition (aggregate content, types of aggregates) Chemically stable and mechanically sound aggregates are used. Aggregates susceptible to ASR must be avoided.

Curing (temperature, moisture, time) conditions As the concrete foundations represent massive structures, the hydration heat generation and its dissipation need to be taken into account. Therefore, the production time, which includes slower rate of casting the concrete layers in order to dissipate the hydration heat, is rather longer than for the superstructures.

4.13.3 Influence of other ageing processes

4.13.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) This is avoided during geological survey.

Leaching (submerged or buried structures) he triggering agents are removed from subsoil or adequate insulation is provided.

Alkali-aggregate reactions The ASR susceptible aggregates are avoided.

Carbonation Not relevant when buried.

Ettringite and thaumasite reactions (DEF/sulphate attack) This potential risk must be avoided at least by responsible control of hydration heat dissipation. Settlement could contribute to risk of external sulphate attack, for example, by inducing contact with groundwater, or increasing the exposed surface area when such contact already exists.

Bacterial processes No information.

4.13.3.2 Physical-Mechanical

Freeze-thaw Not relevant when buried deep enough, as it is common for foundations.

Elevated and high temperature (<150°C, no fire) Not relevant.

Irradiation Not relevant.

4.13.3.3 Mechanical

Effects of (surface) cracks – The cracks developed during production may affect integrity of the foundation, which then may not be adequate for equalizing the possibly differential settlement beneath the foundation. Specific crack width limits differ. Generally, zero crack width is pursued. The cracks may compromise the passivity of steel reinforcement and thus onset its corrosion and swelling which may further damage the concrete.

Abrasion/Erosion/Cavitation Only erosion may occur if the surrounding soil is eroded first, but this would be avoided by countermeasures.

Creep and relaxation These are possible in case of differential settlement. These mechanisms will help to relieve the stress and this reduce damage to concrete (its cracking), especially due to the very slow progress of settlement over time.

Settlements and movements These will occur mainly due to misconduct when assessing subsoil parameters and subsequently underestimation of the foundation load bearing capacity. Excessive differential settlement may result in cracking of concrete foundation and acceleration of ingress of external chemicals and reinforcing steel corrosion.

Vibration (and seismic) Vibration may cause fatigue of concrete in terms of very slow spalling process within the concrete microstructure. Earthquake will result in immediate severe cracks when not designed adequately with subsequent acceleration of ingress of external chemicals and reinforcing steel corrosion.

Thermal stresses (gradients) Not relevant as long-term effect. The effect of hydration heat is described above.

4.13.3.4 Electro-chemical

Pitting corrosion Not relevant

General corrosion Not relevant

Other corrosion mechanisms including crevice corrosion Not relevant

4.13.4 Rates of deterioration

The greatest magnitude of deterioration may come from the curing period when excessive temperatures and shrinkage are not avoided. The cracks caused by hydration heat, the plastic shrinkage and the subsequent uncontrolled drying shrinkage may render the concrete foundation unusable. The potentially delayed ettringite formation due to excessively high temperatures during hydration may also compromise long-term use of the concrete foundation.

ASR may also cause significant problems to the supported structures as the concrete foundation would weaken.

The excessive stresses in concrete foundation may cause mechanical damage in terms of cracks, which would allow for ingress of chemically dangerous liquids from the subsoil and thus compromise the durability.

4.13.5 Impact on concrete properties

Chemical – Pore water chemistry No information

Chemical – Solid phase composition Originally, the concrete should be not susceptible to ASR, it should be chemically inert to the local subsoil and ground water composition. Otherwise, the solid phase composition will change both chemically and topologically (dissolution, leaching, etc.).

Structural – Microstructure The original microstructure should be reasonably closed with minimum continuous pores and capillaries. Any ingress of external chemical may cause undesired chemical reactions leading to decomposition of concrete and reinforcing steel corrosion.

Structural – Cracking The developed cracks from the early ages may compromise stiffness of the foundation which may result in tilting of the structure or misalignment of its segments. If the crack penetrate the entire depth of the foundation, the watertightness may be also compromised.

Transport properties – Porosity, permeability, diffusion coefficients If continuous cracks are not developed, most of the concrete foundation due to its mass is safe from ingress of external liquids and chemicals. The porosity, permeability and diffusion coefficients are typical for commonly used structural concrete and they remain as such for the lifetime.

Mechanical properties Stiffness and strength of concrete depend on quality of concrete microstructure. If the microstructure is not compromised with cracks from the curing period, early-age shrinkage or unaccounted for differential settlement, the mechanical properties remain constant for the lifetime.

4.13.6 Assessment methods

4.13.6.1 Visual inspection

Visual inspection consists of periodic mapping and measurements of crack and their appearance in order to track the history of their development [493]. The inspections are conducted both above ground and underground and they focus mainly on checking the condition of concrete surface and are done once or twice a year

4.13.6.2 Continuous monitoring

Continuous monitoring mostly focuses on topographic inspections of foundation displacement which will indicate possible damage to the concrete of the foundation.

4.13.6.3 Destructive testing of sampling

Not applicable on general basis. In case of serious doubts, core samples can be obtained. Standard testing (uniaxial stiffness and strength, chemical ingress identification, etc.) is then performed.

4.13.6.4 Non-destructive techniques

The georadar inspection is used in case of doubts about subsoil changes.

4.13.7 Performance indicators & acceptance criteria

The performance indicators and criteria are mainly focusing on settlement magnitude itself than on performance of concrete as material used in foundations. The only instant during the lifetime of concrete foundations when concrete is heavily checked is the curing period when its temperature, volumetric changes, strength and crack occurrence are monitored and recorded.

4.13.8 Model approaches

There is no direct relation between the settlement of foundation and concrete performance. All the relations are secondary effects, such as overstressing concrete due to differential settlement or excessive settlement due to failure of concrete foundation caused by corrosion of chloride-attacked steel reinforcement. Therefore, it is impossible to define a model of a direct relation between settlement and concrete performance.

4.14 Vibration

4.14.1 Process Definition

Concrete structures can be subjected to various periodic changes in loading, temperature and moisture content which after a sufficient number of cycles may result in mechanical damage, known as fatigue damage. The frequency of the mechanical cyclic loading varies from few over years, such as earthquake, to thousands of cycles per minute, such as vibration of rotating equipment or flow-induced vibrations. The low-cycle loads tend to reach more significant over stresses than the high-cycle loads, whose amplitude/stress level may be controlled in most cases. The temperature and moisture fluctuations can be considered as mostly low-cycle and with the potential of higher overstressing of concrete when occurring.
As for the high/cycle loading, the fatigue damage of concrete is less frequent for concrete as concrete is designed to transfer compressive loads, while the fatigue damage is related mostly to tensile failures. That means that fatigue failure is usually caused by spitting tension in compressed concrete. The fatigue damage initiates in cement paste close to aggregates in the form of microcracks and with the increasing number of load cycles, it propagates in formation of significant cracks. This process is very nonlinear when the initial decrease of about 15% occurs at the beginning of the cyclic loading. Then, the crack propagation slows or almost stops from most of the fatigue life of the concrete (millions and more of cycles). The final stage when the fatigue failure occurs is quite dramatic and may come unnoticed, which makes this process worth studying.
As for the low-cycle loading, such as earthquake, the damage is obvious and often considerable, therefore, it is easier to assess the residual fatigue life of such damaged concrete structural component.
The typical concrete candidates susceptible to fatigue damage are the support blocks under turbines and generators, anchor zones for fixtures of pipes, which may be also affected by elevated temperature and highly stressed structural component when hit by earthquake.

References: For further references, the reader can consult Do and Chockie [494], IAEA [2] and Morita *et al.* [495].

4.14.2 Influential factors

Reference: Syntheses on influential factors are given in IAEA [2], Morita *et al.* [495] and NRC [492].

4.14.2.1 General conditions

Effect of moisture content and relative humidity - The fluctuation of moisture content may cause additional stresses and, when frequently repeated, the fatigue damage may occur.

Effect of temperature The fluctuation of temperature may cause additional stresses especially in statically indeterminate structures and structural members. When frequently repeated, the fatigue damage may occur.

Effect of irradiation Not relevant.

Effect of microbiological reactions Not relevant.

Effect of exposure condition (free, sheltered, buried, submerged) Not relevant.

Effect of external water composition (submerged or buried structures) Not relevant.

Effect of atmospheric conditions (rain, wind, freeze/thaw) The freeze/thawing is inherently cyclic loading and as such it can be also considered as fatigue triggering. However, it is solved separately.

Effect of mechanical and structural loading Cyclic loading leads to fatigue damage or even failure of concrete. Low-cycle loading (e.g. earthquake) tends to damage concrete at lower total load-cycle count. High-cycle loading (support of turbine or vibrating pipe) tends to damage concrete at very high total load-cycle count, but suddenly. The fatigue damages is usually local.

4.14.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) Rapid hardening cement of high strength class may cause microcracks during violent hydration. These microcracks may expedite fatigue failure

Cement-water binder ratio Lower w/c (below 0.4) indicates high-strength concrete which is more brittle and thus susceptible to fatigue damage.

Concrete composition (aggregate content, types of aggregates) For civil engineering structures at NPPs, the concrete is of average strength and massive, therefore deviation in actual mix proportions can be neglected. Highly stressed concrete components, such as anchor zones and support blocks, may be made of high strength concrete, but they can be easily monitored.

Curing (temperature, moisture, time) conditions High temperatures during hydration and water loss during curing may cause microcracks.

4.14.3 Influence of other ageing processes

4.14.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) High temperatures during hydration and water loss during curing may cause microcracks.

Leaching (submerged or buried structures) Leaching reduced mechanical parameters of concrete, therefore reduces fatigue life.

Alkali-aggregate reactions Reduces fatigue life, must be avoided.

Carbonation Carbonation may accelerate fatigue failure due to reinforcing steel corrosion.

Ettringite and thaumasite reactions (DEF/sulphate attack) Reduces fatigue life, must be avoided.

Bacterial processes Not relevant.

4.14.3.2 Physical-Mechanical

Freeze-thaw Reduces fatigue life when in combination with mechanical cyclic loading [391].

Elevated and high temperature (<150°C, no fire) Reduces fatigue life in combination with mechanical cyclic loading (common case of supports of steam pipes).

Irradiation Reduces fatigue life in combination with mechanical cyclic loading (mostly in combination with low-cycle loading/earthquake).

4.14.3.3 Mechanical

Abrasion/Erosion/Cavitation Not relevant.

Creep and relaxation Creep and relaxation may help to reduce the effect of high-cycle loading when vibrating at low stress level amplitude.

Settlements and movements Differential settlement may cause cracks which reduce the fatigue life.

Vibration (and seismic)

Thermal stresses (gradients) Thermal stresses in constrained concrete member or high thermal gradients may cause cracks, and thus they reduce the fatigue life[2]

4.14.3.4 Electro-chemical

Pitting corrosion Not relevant.

General corrosion Not relevant.

Other corrosion mechanisms including crevice corrosion Not relevant.

4.14.4 Rates of deterioration

High-cycle loading occurs at low stress levels, therefore, the fatigue failure occurs at much higher total load-cycle count (orders of 10^6 and more).

Low-cycle loading occurs at higher stress levels, typically during earthquake, therefore, the fatigue failure may occur after just a few load cycles (units to hundreds, or more).

Higher strength (and corresponding mix design) allows for higher stress levels during cyclic loading, therefore, the fatigue failure occurs earlier.

Cracks caused by uncontrolled hydration heat release and water evaporation (due to reduced mechanical parameters, such as strength and stiffness) reduce fatigue life of structure.

Thermal and moisture fluctuations are rather slow processes in concrete, therefore, their effect on fatigue life is negligible. The exception is quenching of fire when temperature gradient is enormous and thus the single-cycle damage to concrete is devastating.

4.14.5 Impact on concrete properties

Chemical – Pore water chemistry – No information

Chemical – Solid phase composition – Concrete should be not susceptible to ASR and other deleterious chemical reactions. RIVE-susceptible minerals in aggregate should be avoided.

Structural – Microstructure – The concrete microstructure should be sound after production, i.e. the production processes should be controlled.

Structural – Cracking – Pre-cracking from production should be avoided. The cracks reduce fatigue life of concrete.

Transport properties – Porosity, permeability, diffusion coefficients – Thermal and moisture diffusivity help to reduce the detrimental effect of fluctuation to concrete life. During cyclic loading, which causes microcracks in the cement paste around the aggregates, the moisture diffusion coefficient may increase.

Mechanical properties – Due to the cyclic loading, the mechanical properties of concrete tend to decrease (effect of microcracks).

4.14.6 Assessment methods

4.14.6.1 Visual inspection

Visual inspection consists of periodic mapping and measurements of crack and their appearance in order to track the history of their development, or after each major event (earthquake) [493].

4.14.6.2 Continuous monitoring

Continuous monitoring mainly focuses on gathering the data on vibration characteristics.

4.14.6.3 Destructive testing of sampling

Not applicable on general basis. In case of serious doubts, core samples can be obtained. Standard testing (uniaxial stiffness and strength, chemical ingress identification, etc.) is then performed.

4.14.6.4 Non-destructive techniques

Pulse velocity, impact echo or pulse echo may be used [22].

4.14.7 Performance indicators & acceptance criteria

The performance indicators and criteria are mainly focusing on vibration characteristics, which are easily collected online. Deviation in vibration characteristics indicates changes in the rotating system, or its supports (concrete).

4.14.8 Model approaches

4.14.8.1 Empirical models

The traditional Wohler’s S-N curve can be used.

4.14.8.2 Phenomenological models

Paris’ law can be used.

4.14.8.3 Complex coupled models

Fuzzy logic can be used for coupling different models, such as fatigue damage evolution caused by chloride ingress, freeze-thawing and mechanical cyclic loading [492], or other relevant models when data for fuzzy model validation are available.

4.15 Thermal stresses

4.15.1 Process Definition

Concrete ageing produced by thermal stresses can be defined in more detail as the process during which a variety of degradation phenomena (e.g., micro-cracking) are caused by thermal gradients across the structure, and the associated stresses owing to thermal expansion, during its manufacture and/or operation. These gradients can be grouped into three categories based on their evolution over time:

Group 1 – Short-term temperature gradients, for example those which take place during the manufacture of concrete structures due to the exothermic reaction of cement binder. These types of thermal gradients can also occur in service, such as when a structure is exposed to fires. While short in duration (relative to the service life of a structure), the typically high magnitude of such gradients can cause significant stress and associated damage.

Group 2 – Long-term temperature gradients, generally arising from the intended service conditions of the structure, for example:

- Structures of facilities housing heat- or cold-generating equipment.
- Structures in contact with heated or cooled fluids discharged from such facilities, e.g. effluent, cooling water, etc.

In such concrete structures, because of sustained temperature and stress, the process of creep can develop (in particular for heated elements), in addition to formation of micro-cracks.

Group 3 – Periodic temperature gradients, for example as a result of:

- Cyclic industrial heat- or cold-generating processes
- Internal heating or cooling of residential buildings
- Diurnal temperature fluctuations

In such concrete structures with cyclic stresses the process of fatigue can occur, leading to significant micro-cracking.

References: For further references, the reader can consult [496-508].

4.15.2 Influential factors

4.15.2.1 General conditions

Effect of moisture content and relative humidity Due to the high heat capacity of water, presence of moisture can mitigate the development of thermal gradients to a degree by slowing temperature rise. However, upon sufficient heating for evaporation an initially high moisture content may exacerbate damage due to drying effects occurring over short timescales. As such, moisture content is reported to play a significant role in regulating development of thermal stress gradients and associated damage [507].

Effect of temperature As a temperature gradient is requisite for thermal stress to be a relevant deterioration mechanism, the design and operating temperatures of the concrete are quite relevant: if they differ substantially from an external source of heating or cooling, thermal stress deterioration will of course be more severe.

Effect of irradiation Irradiation primarily acts as a source of heating, with some secondary effects in reducing the moisture state of the concrete. Both of these effects manifest as described above, similarly to any other source of temperature or moisture changes. The main feature unique to irradiated concrete may be associated influences on the mechanical properties of the concrete, for example loss of strength due to de-hydration of calcium silicate hydrate, exacerbated ASR (indirect impact of aggregate amorphization), or changes in thermal expansion coefficient (direct impact of aggregate amorphization).

Effect of microbiological reactions Microbiological reactions, aside from benefiting from enhanced reaction rate at elevated temperatures, or suffering from extreme temperatures that do not support microbial growth, are most likely to impact thermal gradients indirectly via their influence on permeability and associated moisture retention of the concrete (thereby mitigating thermal gradients).

Effect of exposure condition (free, sheltered, buried, submerged) Exposure condition dictates the expected nature of heating/cooling, and is thus instrumental to determining when and to what degree thermal gradients and stresses will develop. Beyond this, the material with which the structure is in contact will of course act as a heat sink, e.g., maintaining the exposed surface at or near its own temperature, and so the heat capacity of contacting material (high for water, low for air) and tendency to transfer heat within itself via convection (e.g., free flowing fluid like air or water) as opposed to conduction alone (in a static material like soil) can also play a large role in thermal stress deterioration.

Effect of external water composition (submerged or buried structures) Water composition typically does not substantially impact its heat capacity, aside from in the case of extremely concentrated solutions (e.g., brines). Water composition is not expected to play a direct role, other than via other concurrent deterioration processes which induce changes to the concrete's mechanical properties (i.e., indirectly).

Effect of atmospheric conditions (rain, wind, freeze/thaw) Atmospheric conditions are most likely to influence thermal stress for exposed structures due to diurnal or seasonal temperature fluctuations. Indirect impacts may also be achieved based on moisture state, especially in the case of freeze/thaw cycles, wherein a constant temperature is maintained as the water undergoes its phase transition.

Effect of mechanical and structural loading Mechanical loading is likely to influence thermal stress deterioration, especially via creep[498]. Based on recent advances in understanding the mechanism of creep in moist concretes [34], elevated temperature, and temperature gradients, are likely to increase both the rate of creep as well as the sections of concrete in which it is most prone to occur. Different rates of creep across a thermal gradient may exacerbate other thermal stresses, likely causing more rapid deterioration in cases where mechanical loading is sufficient to induce such levels of creep.

4.15.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) The amount of cement in the mixture affects the specific heating, in which saturated mixtures experience a lot of latent heating due to the dehydration effect. The moisture content of concrete is very important at temperatures below 200°C, because the evaporation of water at 100°C causes an apparent specific heating, that is, there is the effect of twice dries.

In other words, due to the practically doubled amount of thermal energy, doubled "drying" will take place - vaporous release of free and bound water from the concrete. This is taken into account when creating a model of water evaporation with latent heating or artificial increase in the specific heating of concrete in the temperature range from 100°C to 200°C[498].

Cement-water binder ratio Water-binder ratio has a direct influence on both concrete's porosity (e.g., moisture content) and its mechanical properties (i.e., thermal expansion coefficient and durability under stress). Higher water-binder ratios will generally result in more porous and lower-strength concrete, which is more susceptible to thermal stress and associated deterioration.

Concrete composition (aggregate content, types of aggregates) The type of aggregate will dictate the mismatch in thermal expansion coefficient between these materials and the cement paste, significantly influencing the effect of thermal gradients on stress development.

Curing (temperature, moisture, time) conditions Generally, appropriate curing conditions (adequate moisture, controlled temperature) are associated with a greater degree of reaction for a given water-binder ratio of cement paste. As such, curing conditions will have an indirect influence on later development of thermal stress, as well as being of direct relevance in the case of massive structures for which the heat generated by cement hydration reactions can be sufficient in and of itself to create appreciable thermal gradients, due to low rate of heat transfer from the bulk to the exterior environment. These stresses during early curing are particularly important, given the concrete has not yet developed its full strength and thus may be more prone to deformation/damage. Typically such stress gradients can be easily avoided by proper mix design and, in the case of massive structures, adequate insulation and/or cooling considerations during curing.

4.15.3 Influence of other ageing processes

4.15.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) If substantial acid attack has occurred to diminish the dimension of the concrete element across which thermal gradients are likely to develop, this may result in steeper such gradients, and greater stress.

Leaching (submerged or buried structures) Leaching may have an indirect impact on deterioration due to thermal stress, if it is sufficient to change the mechanical properties of the concrete (e.g., via extensive decalcification of CSH).

Alkali-aggregate reactions Sufficient progress of AAR to weaken the area of cement paste surrounding aggregates may result in increased levels of damage following development of thermal gradients and stress in proximity to these weakened regions.

Carbonation Similar to leaching, this may have an indirect impact due to a reduction in concrete's mechanical properties via induced changes to CSH (transformed to calcite).

Ettringite and thaumasite reactions (DEF/sulphate attack) DEF is likely to be associated with high curing temperatures, and thus may co-occur with thermal stress gradients during curing. Later-stage sulphate attack may also pre-dispose the concrete to greater impact of subsequent thermal gradients, due again to an indirect influence on mechanical stability (dissolution of calcium-containing phases). However, sufficiently high temperatures may also destabilize ettringite, potentially compounding the effect of thermal stress.

Bacterial processes See above, regarding microbial reacitons.

4.15.3.2 Physical-Mechanical

Freeze-thaw See above.

Elevated and high temperature (<150°C, no fire) Typically this is closely related to formation of thermal gradients unless the high temperature is maintained uniformly in the concrete, which is rare except in cases of microwave heating.

Irradiation See above.

4.15.3.3 Mechanical

Abrasion/Erosion/Cavitation Not directly relevant.

Creep and relaxation Highly relevant. May compound deterioration due to thermal gradients.

Settlements and movements Not directly relevant, unless they introduce contact of the concrete to a new or increasingly effective heat source/sink.

Vibration (and seismic) Not directly relevant.

4.15.3.4 Electro-chemical

Pitting corrosion Indirectly relevant if sufficient to alter mechanical properties of reinforcement, i.e., enough to influence the overall concrete’s response to thermal gradients in developing stress and damage.

General corrosion Indirectly relevant if sufficient to alter mechanical properties of reinforcement, i.e., enough to influence the overall concrete’s response to thermal gradients in developing stress and damage.

Other corrosion mechanisms including crevice corrosion Indirectly relevant if sufficient to alter mechanical properties of reinforcement, i.e., enough to influence the overall concrete’s response to thermal gradients in developing stress and damage.

4.15.4 Rates of deterioration

Highly dependent on the type of exposure.

4.15.5 Impact on concrete properties

Chemical – Pore water chemistry Other than in the context of other deterioration mechanisms, this is not expected to be relevant.

Chemical – Solid phase composition While solid phase composition undoubtedly influences the response of concrete to thermal gradients, the reverse is likely not the case, with the exception of destabilization of major cement paste phases following prolonged or extreme heating (e.g., fires)[498].

Structural – Microstructure Microstructure will be impacted by thermal gradients via microcracking in response to sufficient stress buildup.

Structural – Cracking Uneven heating of concrete (during hardening), rapid cooling of the surface layers, and maintaining a high temperature inside the product leads to thermal stresses. If they overcome the concrete's inherent strength, cracks form.

Transport properties – Porosity, permeability, diffusion coefficients If thermal gradients are sufficient to induce cracking, these will increase permeability of the concrete[498]. If the magnitude of temperature rise is sufficient to destabilize major cement paste phases, porosity will also increase as these phases dissolve or decompose.

Mechanical properties A decrease in the modulus of elasticity during heating occurs due to an increase in the deformability of concrete and an increase in its elastic deformations, as well as a decrease in the prismatic strength of concrete at these temperatures.

At temperatures above 200˚C, destructive processes start in concrete. The gradual decrease in strength is due to the dehydration of the material and the decomposition of the binding compounds. The degree of destruction is in direct proportion to the growth of the temperature range. The addition of various mineral additives to its composition increases the resistance of concrete to temperature effects [498].

See above "Structural – Microstructure"

4.15.6 Assessment methods

4.15.6.1 Visual inspection

Only when the destruction of concrete reaches the outer surface (outer layer of the material) of the structure, visual inspection is possible.

4.15.6.2 Continuous monitoring

Continuous monitoring using phantom samples - cubes placed in the same environment as the structure under study - cannot be efficient enough, since it is almost impossible for a cube to simultaneously:

- create a configuration of the stress-strain state (SSS) corresponding to a given fragment of the structure body;
- create a configuration of a variable temperature field (temperature gradients) corresponding to a given fragment of the structure body;
- to determine in advance a fragment of the structure, in which the combination of SSS parameters and temperature gradients will be the most unfavourable (critical).

The use of temperature and relative humidity sensors, strain gauges, etc. does not allow directly obtaining reliable information about the integrity (continuity) and parameters of the stress-strain state of concrete inside the structure body, since:

- the placement of sensors on the surface of the structure does not provide information on the parameters of the state of the material inside the concrete mass without additional computational modelling by mechanical and mathematical methods;

- the placement of sensors in the body of the structure during its concreting does not allow calibrating the sensors and communication lines in order to guarantee the reliability of the information received;

4.15.6.3 Destructive testing of sampling

Destructive testing or sampling may not be effective enough for the following reasons:

- it is not known unambiguously in advance in which part of the structure the ageing zone (degradation, destruction) to be investigated is located; "blind" searching for this zone and/or its boundaries may require drilling a large number of holes;
- a large number of holes or holes of a relatively large diameter (for taking cores) can unacceptably reduce the load-bearing capacity of the structure;
- a change in the configuration of the stress-strain state of the structure associated with drilling can accelerate the process of ageing (degradation, destruction);
- analysis of drilled cores will not provide information on the predicted dynamics of the process of ageing (degradation, destruction) [496-505].

4.15.6.4 Non-destructive techniques

The mentioned references [496-505] provide multifaceted information on the results of various studies, as well as on the experience of using various technologies for controlling the stress-strain state of concrete structures, the analysis of which gave the basis for the above conclusion.

In principle, there are many non-destructive testing methods that can be used to assess changes in concrete properties. If the concrete is exposed to high temperature gradients and micro-cracks occur, NDT can detect this.

However, such methods and technologies essentially can only record the consequences - the result of an unfavourable process in concrete, which has led to damage in the body of the structure. But the indicated NDT methods and technologies do not allow identifying the main and accompanying causes of such a destructive process (corrosion, local temperature gradients, external loads and influences, etc.), as well as the current parameters of these processes. At least, the corresponding information has not been found in the available scientific literature.

4.15.7 Performance indicators & acceptance criteria

For ageing process in form of destruction caused with temperature gradients

- about current indicators – No information;
- about possible indicators beyond current practices – a very perspective technology involves mechanical-strength calculative modelling of ageing process in form of destruction caused with temperature gradients with following validation of the results of calculations with destructive testing and/or sampling.

Performance indicators and acceptance criteria have to be developed for mentioned technology [498].

4.15.8 Model approaches

The available scientific literature contains information on the results of studies of the effect of high or low temperatures on concrete. Some information on this aspect is given above. However, information on the effect of temperature gradients on the aging (destruction) of concrete - in the form of some kind of material science and mathematical models, and not local mechanical and strength calculations of the parameters of stress-strain state of structures - has not been found in the available scientific literature.

4.16 Pitting corrosion

4.16.1 Process Definition

Pitting corrosion is a type of localised steel corrosion that forms in the presence of certain aggressive ions, most commonly chlorides. It is characterised by deep and narrow pits of various sizes and depths that are unevenly distributed on the steel surface. Pitting corrosion generally occurs when a large enough chloride concentration locally depassivates the steel surface and causes the formation of macro-cells with a small anodic and a relatively large cathodic area [509]. After the depassivation, pits start forming in three stages: pit nucleation, metastable pitting and pit propagation [510]. During pit propagation, the inside of the pit generally maintains an environment with high acidity and an increased chloride concentration. Equations (9)-(11) describe the anodic reactions inside the pit, where the latter two equations show the autocatalytic nature of the chloride ions, the formation of new hydrogen ions and the resulting increased acidity. This promotes the continuation of the anodic reaction and further pit growth [511].

$$\mathrm{Fe} \rightarrow \mathrm{Fe}^{2+} + 2\mathrm{e}^{-} \tag{9}$$

$$\mathrm{Fe}^{2+} + 2\mathrm{Cl}^{-} \rightarrow \mathrm{FeCl}_2 \tag{10}$$

$$\mathrm{FeCl}_2 + 2\mathrm{H}_2\mathrm{O} + \mathrm{OH}^{-} \rightarrow \mathrm{Fe}\left(\mathrm{OH}\right)_2 + 2\mathrm{Cl}^{-} + \mathrm{H}^{+} \tag{11}$$

References: For further references, the reader can consult [509-513].

4.16.2 Influential factors

Reference: Syntheses on influential factors are given in [511, 512].

4.16.2.1 General conditions

Effect of moisture content and relative humidity Relative humidity has a general influence on the corrosion kinetics, as a certain level of moisture is needed inside the concrete pores to create the necessary ionic transport paths [514, 515]. Typically, relative humidity between 70 and 100 % is needed to achieve significant corrosion rates. Cyclic wetting and drying is the most problematic process, as both sufficient oxygen and water content are needed to initiate and propagate pitting corrosion.

Effect of temperature An increase in temperature on the steel surface proportionately increases the corrosion rates for all corrosion types, including pitting corrosion. When only the kinetic parameters of the corrosion are affected, the relationship is almost linear [516]. Additionally, temperature effects concrete resistivity which, in turn, effects the ion mobility in the pore solution and the corrosion rates [511].

Effect of irradiation Irradiation may cause hydrolysis reactions including the pore water in concrete. This so-called radiolysis could potentially influence also corrosion processes of steel in concrete, but the wider study would be needed to investigate this topic.

Effect of microbiological reactions Certain microorganisms (i.e. SRB – sulfate-*reducing bacteria)* could influence the corrosion processes of steel in concrete. A wider study would be needed to investigate this topic.

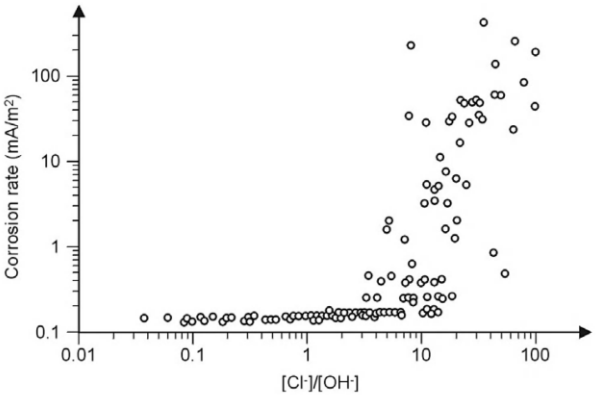
Effect of exposure condition (free, sheltered, buried, submerged) The most problematic exposure condition for steel corrosion in concrete is cyclic wetting and drying, as both sufficient water and oxygen content are needed for the corrosion processes [511, 512]. If the structure is fully submerged or always completely dry, the corrosion processes are impaired due to limited oxygen or moisture [517].

Effect of external water composition (submerged or buried structures) Chlorides in seawater or groundwater can permeate towards the steel surface if they come in contact with the concrete [518]. If oxygen access around the steel surface is limited due to saturation, the corrosion processes are impeded [517].

Effect of atmospheric conditions (rain, wind, freeze/thaw) Wind has no direct impact on the corrosion processes as the steel surface is shielded under a layer of concrete, while freeze/thaw only affects corrosion indirectly, through concrete cracking [511, 519]. Although rain can increase corrosion rates by increasing the moisture content in concrete, it could also initiate pitting corrosion indirectly by leaching the chlorides into the concrete from the environment.

Effect of mechanical and structural loading Mechanical loads don't directly affect the corrosion process. They can, however, affect concrete transport properties by creating microcracks. It was found that 25% of ultimate capacity load is the critical limit where microcracking starts increasing the chloride ion flow [520].

Specific conditions - Effect of chloride content and alkalinity An important parameter when determining the impact of chlorides on pitting corrosion is the critical chloride threshold [521, 522]. Following this concept, there supposedly exists a critical concentration of chlorides on the steel surface that depassivates steel and causes corrosion initiation. Critical chloride threshold is most commonly represented as a quotient of chloride concentration and hydroxide ions in the pore solution $[Cl^-]/[OH^-]$. If pH of the pore solution is not available, then chloride concentration per mass of cement or concrete is used instead. Figure 13 (left) shows the chloride threshold in relation to the expected corrosion rates, while Figure 13 (right) shows the probability of corrosion rate based on chloride content found in the cement matrix.



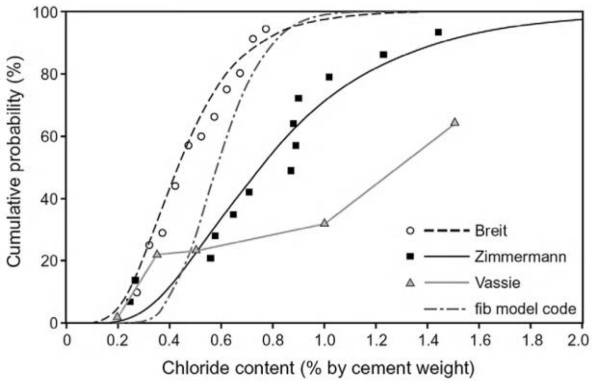


Figure 13. Relationship between the [Cl−]/[OH−] ratio in the pore solution and the corrosion rate of steel (left). Cumulative probability of corrosion initiation versus chloride content (right). Both figures taken from [511].

There is currently no consensus on the chloride threshold that depassivates steel, as the results from the available literature are contradictory and there are many concrete microstructural properties that affect the threshold level.

4.16.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) Supplementary cementitious materials change the microstructure of the concrete and, in turn, affect the corrosion properties of steel. Multiple authors [523-525] have shown that the addition of slag, fly ash and natural pozzolan slow down chloride ingress towards the steel surface by refining pores, making the concrete structure denser and reducing the diffusion coefficient. Additionally, increased alumina content typically found in SCMs has a beneficial effect on binding chlorides and reducing the concentration of free chlorides in the pore solution [526, 527]. On the other hand, cements with SCMs are generally more prone to carbonation, which reduces the pH of the pore solution and influences both the critical chloride threshold and corrosion type [528-530].

Cement-water binder ratio The cement water-binder ratio primarily effects corrosion of steel in concrete through changes in the total porosity [511, 519]. Generally, the higher the water-binder ratio, the higher the resulting total capillary porosity. A higher total capillary porosity will ultimately result in higher corrosion rates due to more interconnected pores and better chloride ion transport paths. A recent study [515] on the kinetics of metals in porous media proposed a theoretical model where total porosity is directly correlated to the measured corrosion rates.

Concrete composition (aggregate content, types of aggregates) This does not directly affect the corrosion processes. On the other hand, certain processes related to mix design properties (i.e. the possibility of ASR) could cause micro-crack in concrete and indirectly affect the corrosion processes.

Curing (temperature, moisture, time) conditions Not relevant. Deterioration due to corrosion generally takes years or even decades before significant corrosion damage can occur. Curing period represents a very small portion of this lifespan, and the immediate effects of curing on corrosion are negligible.

4.16.2.3 Influence of consequences of ageing processes

Effects of mineral phases (depleted mineral phases, secondary phases) No information on this topic was found. It could be relevant, but the authors are not familiar with the subject.

Pore water chemistry The main parameters of pore water chemistry that influence steel corrosion are chlorides and pH of the solution[531, 532]. Both parameters influence the critical chloride threshold needed for depassivation, and the type of corrosion. Pore waters extracted from cements with high alkalinity (Portland cements) tend to exhibit more localised (pitting) corrosion, while pore waters from blended cements or from carbonated concrete tend to have more uniform corrosion, even in the presence of chlorides [533].

Effects of the steel-concrete interface – The steel-concrete interface, which represents the layer where steel and concrete are in contact with each other, has been identified as a potential influencing parameter as far back as 1975 [534]. Despite this, however, not enough extensive research has been done on its effect on corrosion. Two literature reviews [535, 536] were recently published, where the authors identified the knowledge gaps and features present on the steel-concrete interface. Figure 14 shows a schematic representation of the complexity of the steel-concrete interface, including cracks, crevices, slips, air voids and rust. Most of these spatial inhomogeneities are randomly distributed along the steel surface and can cause corrosion processes to behave differently compared to bulk material, such as the formation of pitting corrosion. Generally, the impacts of different features on steel corrosion were found to be contradictory. However, moisture content and steel properties, such as metallurgy and surface finish, have been identified as having a greater impact on chloride induced depassivation compared to the type of cement.

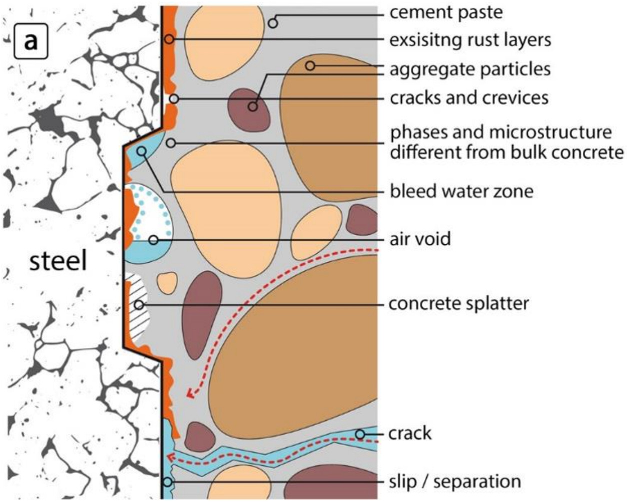


Figure 14. Schematic representation of the steel-concrete interface (adapted from [536]).

4.16.3 Influence of other ageing processes

4.16.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) Acids in combination with chlorides can cause excessive pitting corrosion damage, provided they reach the embedded rebars.

Leaching (submerged or buried structures) Since leaching increases the porosity of the concrete, it will accelerate chloride permeation towards the steel surface and reduce the concrete resistivity. Both factors can hasten the pitting corrosion processes.

Alkali-aggregate reactions ASR could cause micro-cracks in concrete, and therefore indirectly influence the corrosion processes.

Carbonation By reducing the pH of the pore solution, carbonation affects the critical chloride threshold needed to depassivate steel and initiate pitting corrosion.

Ettringite and thaumasite reactions (DEF/sulphate attack) These processes could change the microstructure of concrete, and therefore indirect influence on the corrosion processes may be possible. A wider study would be needed to investigate this topic.

Bacterial processes Certain microorganisms (i.e. SRB) may affect the corrosion processes of steel in concrete. A wider study would be needed to investigate this topic.

4.16.3.2 Physical-Mechanical

Freeze-thaw If concrete gets cracked to a sufficient amount, this will impact its chloride and oxygen transport properties, generally increasing them both. The impact on corrosion is inconclusive and depends on other factors as well.

Elevated and high temperature (<150°C, no fire) High temperature generally accelerates corrosion processes and increases concrete resistivity, but is only problematic if it's subjected to these temperatures in the long term (multiple months or years, can be cyclical).

Irradiation Radiolysis could change the composition of pore water and indirectly influence to corrosion processes of steel in concrete.

4.16.3.3 Mechanical

Effects of (surface) cracks If cracks have a smaller width than 0.3 mm, corrosion rates generally increase with increasing crack width. For cracks wider than 0.3 mm, corrosion stops increasing [537]. In terms of corrosion rates, longitudinal cracks have been found to be more critical than transverse cracks. The impact of cracks on chloride diffusion is conflicting and no conclusions can be made.

Abrasion/Erosion/Cavitation Not relevant

Creep and relaxation These processes could induce microcracking of concrete, and therefore indirect influence on the corrosion processes may be possible.

Settlements and movements Not relevant

Vibration (and seismic) Not relevant

Thermal stresses (gradients) They can cause local cracks in concrete, and indirect influence on the corrosion processes is possible.

4.16.3.4 Electro-chemical

General corrosion Carbonation decreases the pH and affects both the ability to form pitting corrosion and the critical chloride threshold for depassivation.

Other corrosion mechanisms including crevice corrosion No information.

4.16.4 Rates of deterioration

Figure 15 shows the expected range of corrosion rates for concrete in different exposure conditions. The level of chloride contamination and relative humidity are the main influencing factors when it comes to localised and pitting corrosion. Both parameters are heavily influenced by the transport and chemical properties of the concrete microstructure, and the external exposure conditions.

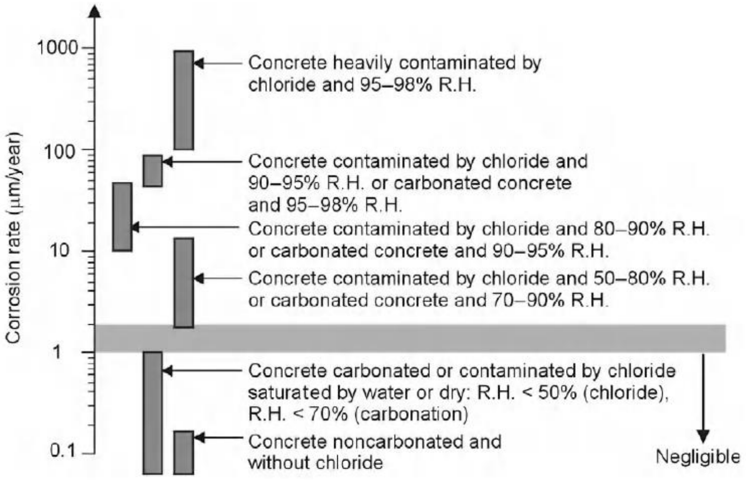


Figure 15. The expected range of corrosion rates for concrete in different exposure conditions (taken from [511]).

4.16.5 Impact on concrete properties

Chemical – Pore water chemistry – Little information is available on how corrosion changes the pore water chemistry. It is known that ferric ions dissolve into the solution and react with other species [511], however, solid corrosion products on the steel surface are mostly formed. In the immediate vicinity of the pit, the pore solution generally contains higher chloride content and a lower pH value [511]. This has no significant impact on the bulk solution, however.

Chemical – Solid phase composition – No information on this topic was found. It is likely not relevant.

Structural – Microstructure – As the steel is depassivated and the corrosion products are formed, they can change the steel-concrete interface, which further impacts the pitting corrosion processes [536]. Due to the small size and the inaccessible nature of the steel-concrete interface during corrosion monitoring and exposure, this topic is not well researched and the effects are largely unknown.

Structural – Cracking – Corrosion products that form during the corrosion process generally have a larger volume compared to its base constituents. This results in the expansion of the material and, ultimately, leads to concrete cracking. Visible cracking occurs only after the corrosion process is well into its propagation stage. Figure 16 shows the relative volumes of the most common corrosion products compared to iron.

Transport properties – Porosity, permeability, diffusion coefficients – The change in transport properties of concrete due to corrosion is mostly related to the level of concrete cracking caused by corrosion products. Once the state of concrete progresses to this point, the corrosion is well past its initiation stage and transport properties of concrete become less relevant.

Mechanical properties – Corrosion has a three-part influence on the mechanical properties of reinforced concrete. The first is the reduction in concrete strength due to the cracking described above [519]. The reduction depends on many parameters, including the degree of corrosion, total porosity, and cement type. The second is the influence of corrosion on bond strength between the reinforcement and concrete, which increases for lower corrosion damage, but significantly decreases for high corrosion damage [538]. The third influence of corrosion is on the cross-section reduction of the steel in concrete, e.g. rebar. Pitting corrosion in particular is problematic, as it can exhibit very high localised corrosion rates If the location of the pitting is in the critical point, e.g. an area with the highest structural load, the service life of a structure can end much sooner than with more uniform corrosion.

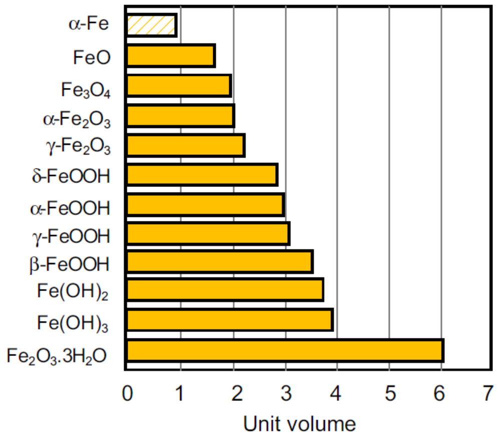


Figure 16. Volume of corrosion products relative to iron (adapted from [512]).

4.16.6 Assessment methods

4.16.6.1 Visual inspection

In order to preform visual inspection of reinforcement, the concrete cover needs to be removed. To differentiate between pitting and other types of corrosion, corrosion products need to be removed. There are different techniques to remove corrosion products, including etching with acids, brushing and sandblasting [539].

4.16.6.2 Continuous monitoring

For continuous monitoring, embedded sensors need to be used. These sensors generally need to be embedded at the time of concrete casting. Multiple types of corrosion sensors exist, such as multifunctional sensors that measure Electrochemical impedance spectroscopy (EIS), pH and chloride concentration [540], electrical resistance (ER) sensors that measure thickness reduction of the steel electrode [541, 542], and coupled multi-electrode array (CMEA) sensors that measure spatial anodic and cathodic current distribution over time [541, 542]. Only CMEA electrodes are capable of correctly detecting localised (pitting) corrosion, but both EIS and ER techniques can give some information about localised corrosion processes.

4.16.6.3 Destructive testing of sampling

For in-situ destructive testing, visual inspection described previously is primarily used. If specimen cores with intact rebars are drilled and brought to a laboratory, multiple more advanced destructive or non-destructive techniques can be used [543]. These include some electrochemical techniques (potentiodynamic polarisation, electrochemical impedance spectroscopy), computed tomography, and various microscopy methods. The electrochemical techniques can provide a more general electrochemical response of steel in a certain concrete environment, while computed tomography and microscopy are able to provide information also about pitting corrosion in particular.

4.16.6.4 Non-destructive techniques

For in-situ corrosion monitoring, two electrochemical techniques are primarily used: galvanostatic pulse and potential mapping [541, 544]. Both techniques utilize a probe, which is pressed against the concrete surface when measurements take place. Both techniques are able to detect locations of different corrosion intensities (localized damages), but neither technique can distinguish between pitting or uniform corrosion. Other non-destructive monitoring techniques require laboratory equipment but can be adapted for in-field use in a simplified form (e.g., electrochemical impedance spectroscopy). For in-situ continuous monitoring coupled multi-electrode array (CMEA) sensors and electrical resistance (ER) sensors can be embedded in reinforced concrete structure [545].

4.16.7 Performance indicators & acceptance criteria

Some performance indicators and acceptance criteria can be found in ASTM standards, but are mainly limited to specific conditions. For example, ASTM C876 defines the standard test method for corrosion potentials of uncoated reinforcing steel in concrete and also provides a relationship between the potential values and corrosion probability. It should be emphasized, however, that specific parameters can significantly affect the results and therefore their interpretation has to be performed by certain care [546]. It was suggested that beside the absolute values of corrosion potentials, their relative differences need to be taken into account. In particular, there are currently no performance indicators related to pitting corrosion in concrete that authors are aware of.

4.16.8 Model approaches

Various deterministic models based on the calculation of the carbonation rate or chloride ingress were proposed [511, 521, 522]. All of them include a critical parameter for the corrosion initiation: pH value and chloride threshold value. Since a significant scatter of these parameters in different conditions was found, the reliability of the models is limited. For pitting corrosion, a model was proposed where propagation rates of pits are evaluated using a pitting ratio [547]. This method consists of estimating the maximum depth of localised corrosion as a function of the general corrosion depth.

An overview of existing phenomenological models with emphasis on chloride-induced (pitting) corrosion of steel liners in concrete will be performed in the scope of Subtask 2.1.3 of the ACES project. Existing key models for the probabilistic analysis of chlorides-induced corrosion will be analyzed within Task 2.2 of the ACES project.

4.17 General corrosion

4.17.1 Process Definition

Corrosion induced by carbonation takes place on the whole surface of steel in contact with carbonated concrete. Carbonation was detailed previously in section 4.4. Carbonation induced a pH change of pore water. Corrosion in NPP affect the containment building, the cooling tower, the cooling pipe made of concrete with steel liner and all structures [17].

As soon as the outcome of the steel rebar from manufacturing, the steel is immediately, recover from a very thin oxide thickness (so-called mill scale with FeO and Fe_3O_4 and Fe_2O_3). On a (NPP) construction site, before being placed in the formwork, a steel rebar is rusted because it is first exposed to atmospheric conditions (oxidant conditions by O_2, H_2O). When fresh concrete is placed around this steel, the mixing water reacts with the metallic steel and forms a thin layer made of iron [$Fe(OH)_2$] and calcium [$Ca(OH)_2$] hydroxides on it. The mixing water in the concrete leads to the formation of a protected layer around the steel of iron oxide, which is called passivation.

Sound concrete is characterized by its high alkalinity thanks to the hydration of the cement. The pH of a basic interstitial solution is around 13. Corrosion does not develop as long as the concrete provides chemical protection for the reinforcements [511].

However, CO_2 in the atmosphere combined with water in cement forms carbonic acid H_2CO_3 (dissociates in water into two species, hydrogen carbonate (HCO_3^-), the dominant species at a pH between 6.3 and 10.3 and carbonate ion (CO_3^{2-}) dominant at a pH greater than 10.3) [548, 549]. As a consequence of carbon dioxide dissolution, the global pH decreases to a value of about 8 and the dissolution of the hydrates, especially the portlandite $Ca(OH)_2$. That phenomenon is called carbonation and is driven by the CO_2 diffusion (see section 4.4).

In agreement with Pourbaix diagram (Figure 17), at high pH, iron oxides (Fe_3O_4, Fe_2O_3) are stable and the concrete-embedded steel is in a passive state. When the carbonation front cross the rebar, steel corrosion proceeds as an electrochemical process. Iron atoms convert to positively charged ions (Fe2+) and generated electron, which is called oxidation.

The electrochemical reaction is composed of four partial processes [549, 550] (Figure 18) :

- iron anodic reaction: at the active sites on the rebar, the oxidation of iron (anodic process) liberates electrons that move into the surrounding concrete as ferrous ions;
- cathodic reaction: the reduction of oxygen (cathodic process) consumes the electrons released from the anodic reaction;
- transport of current within concrete: possible due to the electrolytic conductivity of concrete, for which the presence of water is essential: this is ensured by the ion movement in pore solution from the cathodic regions to the anodic ones;
- transport of current in the metal: the transport of electrons within the metal from the anodic regions to the cathodic regions induces an electrical current in the opposite direction because electrons carry a negative charge.

To summarize, when corrosion starts, the steel is attacked by electrochemical reactions which lead to a transfer of ions and electrons at the metal-solution interface, under the influence of a potential difference between the two phases.
Succession of micro-cells along the rebar constitutes the general corrosion process.

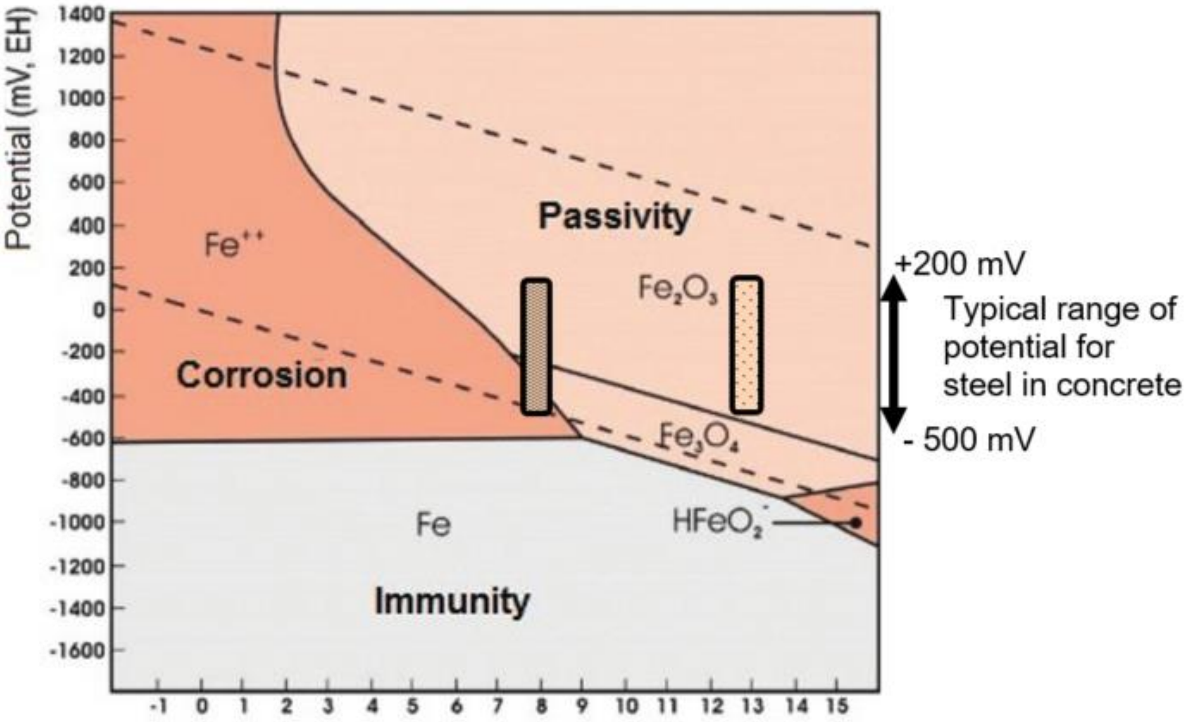


Figure 17. Pourbaix diagram at 25°C for Iron-water system from [551].

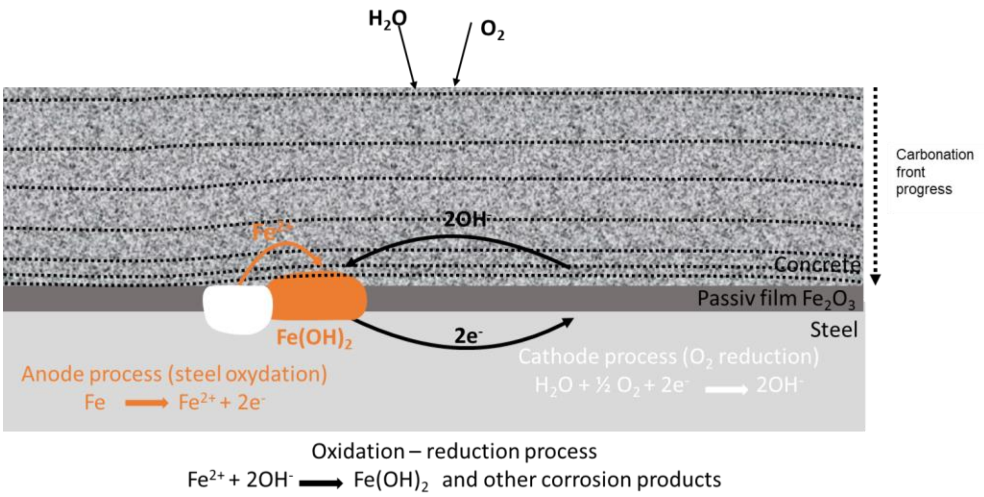


Figure 18. Scheme of cathodic and anodic corrosion cell

When depassivation of the rebar starts, the metallic iron is transformed to a ferric oxide (rust), and the corrosion products begin precipitating in the connected in the porosity around the rebar ; at the beginning with small product volume and without creating negative side effects on the concrete. However, at an advanced state of corrosion, the volume of corrosion products can even reach six times the metallic iron volume [552]. This increase in volume causes pressure on the steel/concrete interface that can lead to cracking of the concrete once the tensile strain capacity of the concrete is exceeded. A small volume growth may lead to cracks in the concrete cover along the line of bars. A more severe volume growth can lead to delamination or even to spalling if this volume growth is excessive.

In the life of a reinforced concrete structure, we can distinguish two periods for general corrosion: an initiation period and a propagation period which is discussed further in section 4.17.8.1.

References: For further references, the reader can consult [513].

4.17.2 Influential factors

Reference: For a complete critical review of influential factors refer to Stefanoni *et al.* [514].

Remark: In the following, we focus on factors influencing corrosion and not on carbonation that is developed in the section on carbonation (section 4.4).

4.17.2.1 General conditions

Effect of moisture content and relative humidity Moisture is essential for corrosion because it decreases the electrical resistance of concrete. The electrochemical activity of steel is highly dependent on ambient humidity. Thus, corrosion occurs in wet but unsaturated concrete. In water or saturated concrete, oxygen diffusion becomes very low compared to that in air, which also limits the cathodic reaction. It is generally estimated that the corrosion rate increases from 60% of relative humidity to its maximum in a concrete in equilibrium with a relative humidity close to 95% [553]. Below 60 % corrosion appears to be negligible due to very dry cement. But above 95%, Tuutti show that no corrosion seems possible due to very low oxygen diffusion, while other study make a difference and report an average corrosion rate between 5–20 µm/year in the range 95 % to 100 % [514] (Figure 19).

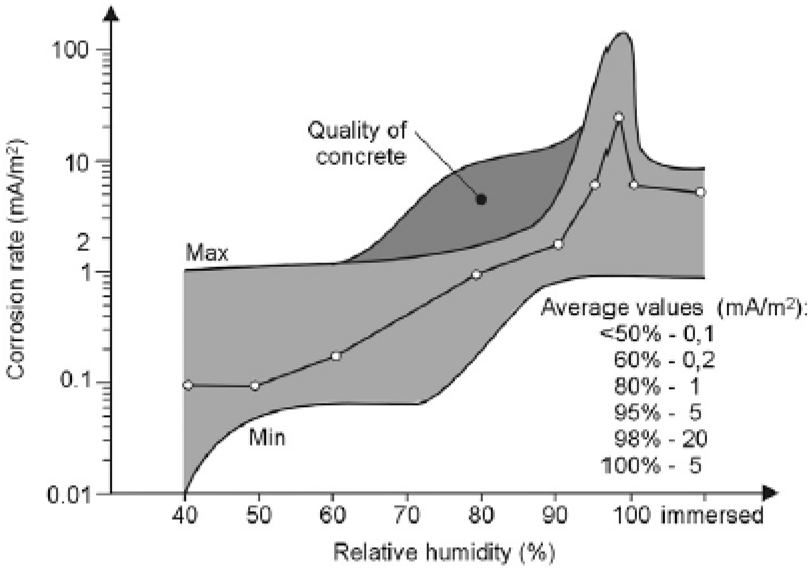


Figure 19. Maximum and minimum values of the corrosion rate in carbonated concrete versus relative humidity [511, 554].

As shown in Figure 20, when relative humidity is about 60%, carbonation rate is maximal but corrosion rate is very low because water saturation level is not enough to promote corrosion.

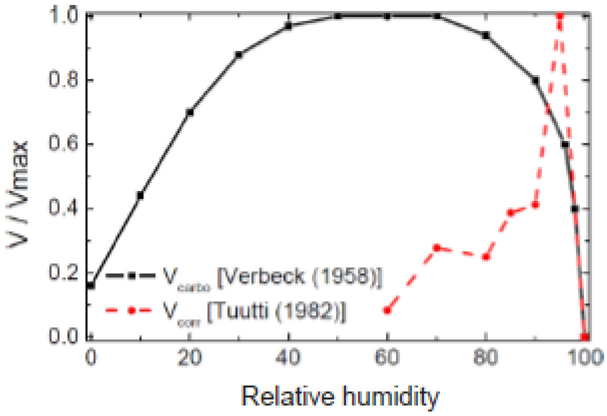


Figure 20. Comparison between carbonation rate and corrosion rate versus relative humidity [549, 553].

Effect of temperature In concrete, the corrosion rate follows approximately the same relation as other corrosion processes, which entails doubling the corrosion rate for a temperature increase of 10°C [553]. This is true for low ambient temperature. At high temperatures above 40°C, the effect of temperature should never be considered alone but coupled with electrolyte availability and degree of pore saturation. High temperature increases the risk of corrosion in moist concrete (especially with chloride), whereas they have the opposite effect on dry or semi-dry concrete [555]. At temperatures higher than 40°C/50°C, the oxygen is less soluble in water and, therefore, the corrosion rate drops down with the increase in temperature [556, 557]. However, in a ventilated environment, this conclusion appears as not reliable.

Effect of irradiation Gamma irradiation induces to pore water hydrolysis and the reduction of the oxides formed. Moreover, neutron irradiation produces changes in the mechanical properties of carbon steels (e.g., increased yield strength and rise in the ductile-to-brittle transition temperature). A fluency of 1 x 1018 neutrons per square centimetre is considered as threshold and is possibly only in the concrete primary biological shield.

Those two phenomenon are not the same than corrosion induce by pH decrease due to carbonation and no relationship was found [552].

Effect of microbiological reactions Not relevant for NPP civil engineering structure as cooling tower and containment building exposed to external conditions.

Effect of exposure condition (free, sheltered, buried, submerged) Andrade *et al.* [558] studied four main weather events that have influence on the corrosion rate of reinforcements due to the changes of the hydrothermal situation of the concrete: day–night cycles, seasonal cycles, extreme temperatures and rain periods. Temperature is the main driving force of the moisture content of concrete in sheltered from rain conditions, while rain periods (length and frequency) are responsible for the moisture content in unsheltered conditions.

Effect of external water composition (submerged or buried structures) Not relevant, in case of chloride water, see section 4.16.

Effect of atmospheric conditions (rain, wind, freeze/thaw) - *Effect of CO_2* Concentration of CO_2 in atmosphere is the main factor for cement carbonation.

From 1800 the CO_2 concentration increases from 300 pm to about 400 ppm with a discrepancy between rural area and urban cities (up to 430 ppm). In tunnels, CO_2 concentration reaches 1%.

In the future, considering climate changes, that parameter has to be considered for assessing corrosion [559]. Depending on climate scenario, CO_2 concentration can reach on average between 421 ppm (RCP2.6), 538 ppm (RCP4.5), 670 ppm (RCP6.0), and 936 ppm (RCP 8.5) by the year 2100 [560]. Impact on carbonation depths for concrete infrastructures located in specific cities could be consistently 1.5 times higher than others. For normal strength concretes, carbonation depths increase by up to 31% while service lifespans correspondingly reduce by up to 24%, owing to the rise in [CO_2] levels between the years 2000 and 2100. The consequences are that the use of higher strength concretes ≥40 MPa appears to be necessary for new structures and for existing infrastructures, the resulting shorter lifespans imply an earlier onset of corrosion problems, which in turn imposes higher repair [561].

For general corrosion assessment, scientists perform accelerated experiments in carbonated chamber with higher CO_2 concentration.

Effect of oxygen Oxygen is essential for steel depassivation because directly involve in chemical reactions. However, there is discussion on the role of oxygen in the phase of corrosion progression [562].

Effect of mechanical and structural loading Mechanical loading leads to damage at the steel/mortar interface characterized by debonding and microcracks. In case of high-level concrete cover damaging, the later facilitates the spread of chemical agents and corrosion can initiate at different close locations quickly followed by general corrosion [549]. In case of low cracks density, see section 4.18.

4.17.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) The influence of cement type on corrosion exists in carbonation propagation and stability of the corrosion products associated with the composition of the pore solution and its pH [563].

The most resistant cement to carbonation is the ordinary Portland cement having the higher portlandite content [564]. The carbonation rate of blended cements has been found to be a factor of 2–4 higher than for Portland cement [514]. Cement type can also influence the electrical resistivity of the concrete [565] parameter that is discussed later.

The review of Stefanoni *et al.* [514] reports that the corrosion rate of steel in carbonated mortar or concrete increases for clinker replaced binders by fly ash, blast furnace slag, natural pozzolans, Limestone or Silica Fume.

Cement-water binder ratio It was experimentally observed in several old studies that the corrosion rate increased with the water to cement ratio. In 1996, Balabanic *et al.* [566] analysed the influence of concrete quality (w/c ratio) on the corrosion current density and deduced that the reduction in w/c ratio influences much more the corrosion current density value than an increase in the thickness (from 5 to 10 cm) of the concrete cover with the same quality.

A general trend is that a decrease in w/b ratio corresponds to a reduction in corrosion rate of the embedded steel in carbonated mortar or concrete. For example a decrease of w/b from 0.8 to 0.55 led to a decrease from 2 $\mu A/cm^2$ to 0.8 $\mu A/cm^2$ (~2.5 times), at 100% RH.

In some studies a significant influence of the w/b ratio on the corrosion rate of steel in carbonated mortar was found at high RH. At low RH the influence of the w/b ratio on the corrosion rate is negligible, suggesting that in low moisture condition the corrosion is so slow that the w/b ratio does not play an important role [511, 514].

Higher porosity has been found to correlate with higher corrosion rates [564]. Also a main parameter, because high porosity has to be linked with Cement water-binder ratio, cement type and moisture [567].

Concrete composition (aggregate content, types of aggregates) Not relevant.

Curing (temperature, moisture, time) conditions Because curing has an influence on cement hydration it also has influence on carbonation. The depth of carbonation was affected more by cement type than by curing. The longer initial curing period resulted in lower carbonation depth. The effect is more marked with moist curing [568].

However, curing has little effect upon the rate of corrosion but higher rates were observed when the cement contained granulated blastfurnace slag [569]. Water curing periods (from 1, 7, 28 and 91 days), the indoor and outdoor curing influence the carbonation and corrosion [564, 569].

4.17.2.3 Other influential factors

Concrete resistivity – Another main parameter that influence the corrosion rate is the electrical resistivity of the concrete, but there is no consensus in community.

In [565], six types of carbonated cement paste were tested and the relation between the corrosion rate and the resistivity was quite similar in all the cases. Bamforth [570] (from [549]) shows different electrical resistance versus cement type and additives.

The frequently mentioned inverse relation between concrete resistivity and corrosion rate is considered as an empirical correlation, both parameters depend (inversely) on the degree of pore saturation of the concrete [514]. Each improvement in resistivity induces a decrease in corrosion rate.

Effect of Steel/mortar interface quality / cavity – The steel/mortar interface quality, influenced by the mix design and the concrete casting process, appears to be an important parameter for corrosion. A bad quality interface is a privileged path and location for oxygen that is the primary factor of depassivation.

Moreover, several recent studies show a settlement and water bleeding or higher porosity of fresh concrete under horizontal reinforcement with respect to casting direction creating gaps along the steel/mortar interface that was not observed for vertical bars. Corrosion starts in the voids in contact with the steel bar preferably at entrapped air voids with diameter bigger than 2.5 mm considered as macro defects [549, 571]

At last, the quality of concrete and steel–concrete interface, decreasing with height of concrete section, affects directly the corrosion rate [572].

The corrosion is more developed in the lower surface of the steel reinforcement located at the higher section in each block of concrete with respect to casting direction.

That is in agreement with the hypothesis proposed where the corrosion rate is controlled by the amount and distribution of electrolyte at the steel/concrete interface.

Effect of concrete cover depth – The concrete cover ensures the protection of the steel reinforcement. For a given water to cement ratio, the corrosion rate decreases with increasing cover thickness. Furthermore, the thickness of the concrete cover is one of the factors affecting the cracking and spalling of concrete as a result of the corrosion development [549].

Standards such as Eurocode 2 specify for each environmental condition a corresponding concrete cover depth. General corrosion occurs when the distance between carbonation front and reinforcement bar surface (the uncarbonated depth) is <5 mm [103].

4.17.2.4 Corrosion prevention and protection

Adding corrosion inhibitors to the mixing water of concrete is the most frequently used technique for new structures or in repair mortars in order to prevent or at least delay the onset of corrosion [511].

Protectives products represent another solution to limit the penetration of carbon dioxide, it is possible to apply protection products [573] on the concrete surface to completely or partially seal pores and micro-cracks. These products are classified by the standard NF EN 1504-2 in three families [574].

At last, cathodic protection represents an electrochemical solution that was proven [575].

For more details on good practices and concrete technologies for corrosion prevention refer to [511]and European standards.

4.17.3 Influence of other ageing processes

4.17.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) No information

Leaching (submerged or buried structures) Pure water or treated water (used in NPP) can leach concrete by dissolving cement phases and increase the porosity of the concrete and at last the saturation rate.

Alkali-aggregate reactions ASR expansion does not significantly influence the time to corrosion initiation of reinforced concrete systems for laboratory specimens exposed to wetting / drying cycles at 100 ° F (38 ° C) [576].

Carbonation See Process Definition in section 4.17.1.

Ettringite and thaumasite reactions (DEF/sulphate attack) not relevant

Bacterial processes not relevant

4.17.3.2 Physical-Mechanical

Freeze-thaw not relevant

Elevated and high temperature (<150°C, no fire) not relevant

Irradiation not relevant

4.17.3.3 Mechanical

Abrasion/Erosion/Cavitation Any phenomenon inducing a reduction in the depth of the concrete cover contributes to increase the carbonation speed and the initiation of corrosion.

Creep and relaxation not relevant

Settlements and movements not relevant

Vibration (and seismic) not relevant

Thermal stresses (gradients) In case of high thermal stresses, differential gradient and strain at the binding/steel interface, they contribute to local debonding and cracks

4.17.3.4 Electro-chemical

Pitting corrosion not applicable because another type of corrosion.

Other corrosion mechanisms including crevice corrosion not relevant. In that case, crevice corrosion appears before general corrosion.

4.17.4 Rates of deterioration

From Stefanoni *et al.* [515], the average value of the minimum corrosion rate for steel, due to general corrosion in chloride-free concrete is about 0.08 µA/cm^2 (or 0.9 µm/year) and the average of the maximum is about 2 µm/cm^2 (or 23 µm/year). In the case of steel, 0.1 µA/cm^2 corresponds to a penetration rate of about 1.17 µm/year.

When in the passive state, the corrosion rate of steel reinforcement is very low. Its value is approximately equal to 0.1 µm / year [577]. Under 1 µm/year general corrosion is considered negligible (Figure 15). Above 1 or 2 µm/year general corrosion is considered active by the scientific community but very low [511].

4.17.5 Impact on concrete properties

Chemical – Pore water chemistry Around rebars, pore water chemistry will be modified due to development of corrosion products.

Chemical – Solid phase composition No information found

Structural – Microstructure Not applicable

Structural – Cracking General or uniform corrosion also refers to the corrosion that proceeds at approximately the same rate over the exposed metal surface. So cracks will appear as the first visible surface damages along the rebar. The next step is the spalling of concrete cover and finally the exposure of the steel rebar.

Transport properties – Porosity, permeability, diffusion coefficients –

Mechanical properties The not avoidable mechanical consequences of corrosion are: the creation of expansive products (commonly denoted as rust), the reduction of the resistive section of reinforcements, the fragilization of steel; and finally the cracking and spalling of concrete. All of them conduct, in case of general corrosion, to the programmed repair or destruction of the structure that had been planned for a specific service life. In addition to cracking and spalling, corrosion will result in a reduction of steel cross-section and load capacity, a decrease in ductility, and loss of composite interaction between concrete and steel due to bond deterioration.

However, it is important to consider that all the corrosion products do not have the same mechanical resistance, which depends from their crystallinity, herself function of the environmental conditions at the time of their formation [549]].

Moreover, Naus [552] mentioned that homogeneous corrosion is negligible in terms of section reduction for high diameter bars, but pitting corrosion has a relevant effect in small diameter bars.

At this level of the document, it is important to specify that these high degrees of corrosion, which may exist in conventional civil engineering structures, are not acceptable by the nuclear authorities for NPP structures in particular for containment building

4.17.6 Assessment methods

As corrosion is an electrochemical process, electrochemical techniques are especially well suited to assess the corrosion state of the reinforcement before any external visible effect (Figure 21). Visual inspection of the cover depth remains however a way to detect first external signs of corrosion.

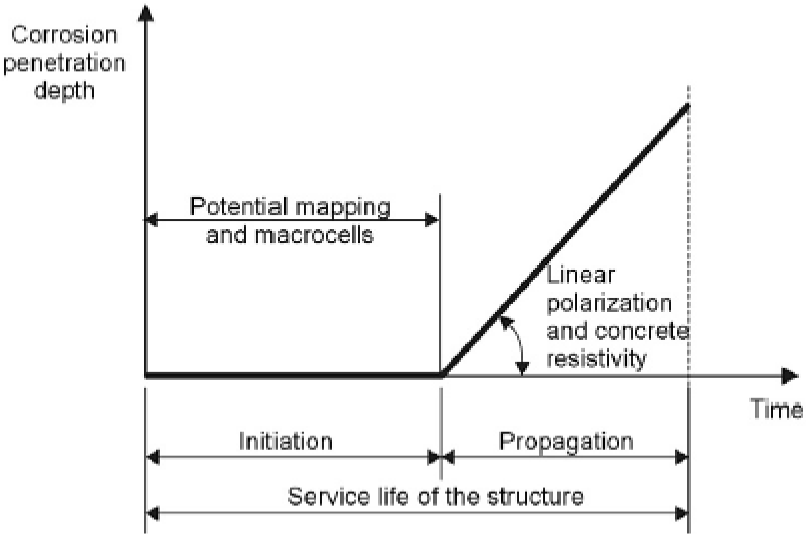


Figure 21. Different monitoring techniques for different stages of corrosion [511].

4.17.6.1 Visual inspection

Whatever the others method, visual inspection is generally unmissable. A full surface inspection assesses the global state of the reinforced concrete. This includes a visual inspection of the concrete reporting any visible deterioration, e.g. surface cracks, delamination or rust.

Corrosion products have higher volume than sound steel, induce swelling inside the concrete and cracks along rebar. In case of general corrosion, concrete blocks fall down. Best way to avoid global corrosion is to dimension the thickness of concrete cover in agreement with the expected lifetime of the structure.

4.17.6.2 Continuous monitoring

One classical method is to use of ghost samples – cubes placed in the same environment to measure

Commercial continuous corrosion monitoring that can be embedded in the structure is not found.

Commercial sensors for pH monitoring are not available so far, but several materials have been tested in the laboratory [511, 563].

Miniaturized referenced electrodes, to be attached to rebars, constitute an available corrosion continuous monitoring[416]. The working principle is commonly based on the monitoring the potential drop of single sensors (anodes) in different depths as a function of time. They enable locating the depassivation front.

4.17.6.3 Destructive testing of sampling

Not applicable.

4.17.6.4 Non-destructive techniques

Electrical methods are the most used to evaluate the corrosion rate. These methods require the use of an electrical system with two-, three- or four-electrode configurations to determine three main parameters: corrosion potential, concrete resistivity and polarization resistance [513]. Many commercial devices are available based on three-electrode configuration for polarization resistance measurement [578]. The principle is based on the measurement of the electrical conductivity of the fluid that can be related to its corrosiveness. For [511], it is the only technique available today to provide quantitative information on the corrosion rate of steel.

Measurements of concrete cover depth [579, 580], or ultra-sound /radar technologies for rebar positioning and corrosion inspection are also used [581].

4.17.7 Performance indicators & acceptance criteria

Performance indicators and criteria for quantifying the state of corrosion are associated with the electrochemical effect of corrosion [582]:

For corrosion current :

- icorr < 1 mA/m^2 passive condition
- 1 < icorr < 5 mA/m^2 low to moderate corrosion
- < icorr < 10 mA/m^2 moderate to high corrosion
- icorr > 10 mA/m^2 high corrosion rate

For potential of corrosion E (mV CSE) [511] [Bertolini]:

- E > −200 probability of corrosion <10%
- −200 > E > −350 unknown
- E < −350 probability of corrosion >90%

For concrert resistivity, despite some differences, we can mention that after steel rebar depassivation :

- ➢ 1000 Ωm low corrosion rate (no clear distinction with passive state)
- 500–1000 Ωm low to moderate corrosion rate
- 100–500 Ωm high corrosion rate
- < 100 Ωm very high corrosion rate and the concrete resistivity is not more the controlling parameter

The European standard on concrete EN 206-1 published in the year 2000 classified the risk of carbonation-induced corrosion depending on the severity of the environment (XC1 to XC4) for lifteime up to 100 years.

4.17.8 Model approaches

Carbonation models to represent the passive phase were presented in section 4.4.8.

4.17.8.1 Empirical models

The first empirical model is based on sequential stages [553] (Figure 22):

- Initiation: this long period is characterized by the ingress of aggressive species from the environment to the steel. There is no active corrosion on the rebar. Consultant engineers should be aware that the initiation phase of corrosion can become an important part of the total service life of a structure, thus blended cements have to be applied carefully, and taking into account the exposure condition (moisture).
- Depassivation: this step happens when the conditions required for the onset of active corrosion are fulfilled thanks to the transport of aggressive species through concrete cover (chloride ingress or carbonation front evolution) and the oxygen availability for corrosion.
- Propagation: the reinforcement corrosion causes significant loss of section of the reinforcements, especially in the case of chloride-induced-corrosion. Internal micro cracking and spalling of the concrete cover appear. They are due to the high tensile stresses generated by the expansive nature of the corrosion products.

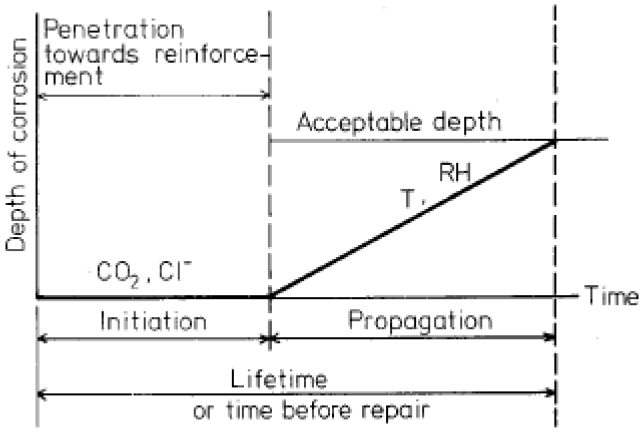


Figure 22. General corrosion process: Initiation and propagation phase [511, 553].

Raupach [582] referred in his review to following types: empirical models based on corrosion rate calculated from experimental measurements, on assessment of deterioration using fuzzy logic and on probabilistic models. Most of them consist in fitting experimental data with empirical formula assuming direct relationship between the corrosion rate of the reinforcement and basic parameters of the concrete like w/c-ratio, type of binder, etc. and the exposure conditions. They are interesting in case of parametric studies of the corrosion rate. Their limitations come from the lack of corrosion process and mechanisms and the impossibility of extrapolation.

Bazant's Model [583, 584] predicts the corrosion damage based on volume expansion due to the formation of hydrated red rust over the residual rebar core. This rust is expansive in nature and occupies four times the volume of parent steel that creates a pressure on the concrete to produce cracks on its surface.

4.17.8.2 Phenomenological models

These models are based on the nature of electrochemical corrosion of steel in concrete taking in account other influencing factors like exposure conditions, water/cement ratio, curing conditions, type of cement, etc. only indirectly into account.

Different mechanisms of steel corrosion in carbonated concrete have been proposed [514, 582] and the topic is still under discussion. As shown previously, correlation exists between corrosion rate and the different durability parameters (electrical resistivity of concrete, degree of water saturation, pore distribution…). Thus, different options for steel corrosion rate mechanisms in carbonated concrete are proposed:

- Based on ohmic resistance or concrete resistivity (with or without pH control)
- Controlled by oxygen diffusion (amount and composition of electrolyte present at the steel surface, degree of pore saturation)
- Controlled by anodic process.

For a dry environment electrical resistivity is supposed to be the corrosion controlling factor and such models are highly valid, while for wet environments oxygen diffusion becomes the controlling parameter.

Stefanoni *et al.* [514] considered that: “the service life of concrete structures made with ordinary Portland cement can be predicted from the air permeability and corrosion rate determinations. However, mechanistic understanding is still lacking, not allowing us to deduce a quantitative model in the case of blended cements due to the enormous variety of formulation, different experimental approach and corrosion measuring techniques”.

4.17.8.3 Complex coupled models or numerical models

An introduction to the different approaches of numerical models (resistor networks and transmission line models, finite element models and boundary element models) applied to the corrosion in concrete is given in [582]. Whereas, transmission line models and resistor networks are suitable to estimate electrical fields of simple geometries, when the corrosion systems become more complex and when more accurate results are required, computational methods like finite difference, FEM or BEM methods give more reliable results as far as convenient electrical parameters are choice.

Millard and L’Hostis [585] proposed an approach dealing with the coupling between the concrete hydric behaviour, the rebar corrosion due to oxygen transport and reduction and finally the concrete cracking due to the expansion of the corrosion product layer. He mathematically described the corrosion rate as a function of oxygen concentration, corrosion product porosity iron density and some reaction kinetic constant.

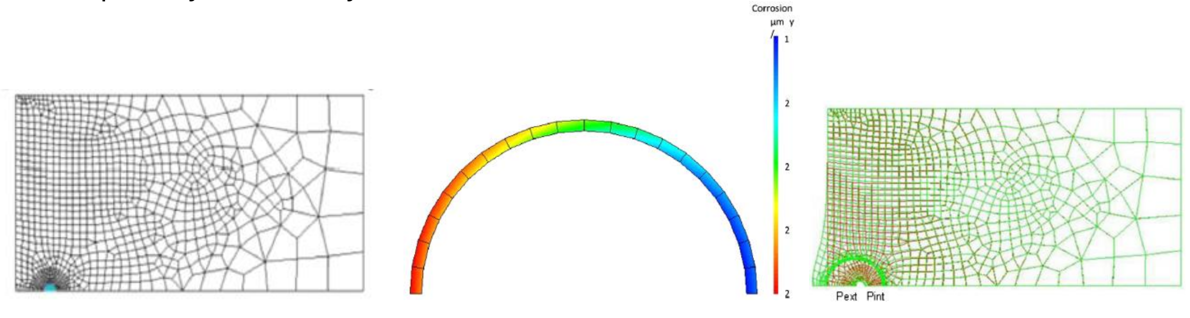


Figure 23. Mesh of the half slab (a), average corrosion rate at metallic interface, rust layer (c) [585].

4.18 Cracking corrosion

4.18.1 Process Definition

Cracks in concrete can appears as a consequence of rebar general corrosion and expensive volume of corrosion products (see also section 4.17).

But cracks can also appear due to others phenomenon and have as consequence to induce corrosion. Micro-cracks are inherent and unavoidable in normal concrete structure, form at aggregate boundaries, and propagate through the surrounding mortar. Some micro-cracks can be generated at the interface between the steel and the concrete due to tensile stress. Cracks may arise due to drying shrinkage, plastic shrinkage or due to reactions between the aggregate and cementitious materials in alkali silica reactions. Other sources of mechanical loading can induce cracks (see previous sections).

In any case, a crack is a preferred path for aggressive agents (CO_2 , O_2 or chlorides and water) that follow the crack profile to reach the rebar, reduce corrosion initiation time and provide the initiation of the cathodic/anodic process.

A general agreement exists in the literature on the effect of cracks in inducing interfacial slip and separation between steel and concrete and in accelerating the corrosion initiation deep in the crack and a few millimetres around the rebar intercepting the crack. However, the propagation of reinforcement corrosion induced by carbonation in the crack is stills a subject in debate.

Ten years ago, when the effect of cracking on corrosion of reinforcing steel was concerned, there are two schools of thought: while some researchers consider cracking to accelerate both the onset of corrosion and its propagation, others argue that, even though cracks could shorten the initiation phase, the propagation of corrosion is not affected. Recent research tends to conclude to the second hypothesis for crack widths up to 0.5 mm. In that case, after the initiation of corrosion on the steel surface, the corrosion rate is low. Formation of corrosion products may seal the crack near the reinforcement, act as a barrier to oxygen and water diffusion and allow the protective oxide film to form again. Repassivation can occur when the alkalinity around the rebar reaches again values around 11.5 until other pathologies become predominant in the damage.

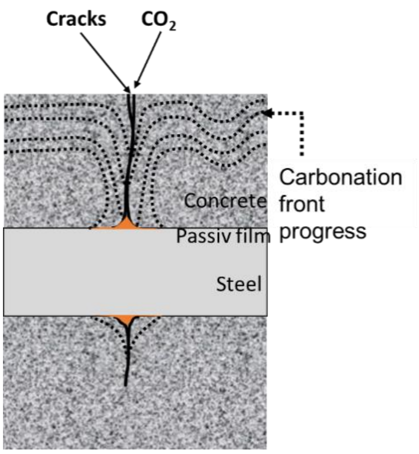


Figure 24. Scheme of cracks, carbonation front and rebar corrosion (after [511]).

Remark: In NPP, beyond corrosion, cracks have another important consequence regarding the leakage rate in case of accident and is regularly (generally once every ten years) measure during swelling test.

References: For further references, the reader can consult [511, 537, 549, 553, 586-590].

4.18.2 Influential factors

4.18.2.1 General conditions

Effect of moisture content and relative humidity Because they can influence cement binder permeability, moisture and relative humidity have an effect on the concrete cover and favour cracks formation [511, 553].

Effect of temperature Temperature is one of the main factors inducing cracks generation because of the different thermal expansion between concrete and rebar. In case of cooling tower, cracking is created in part by the thermal loadings induced during operation [591].

However, after the initiation phase, corrosion kinetics inside cracks are the same at different temperature (evaluated in laboratory for 20°C and 40°C). The same corrosion rate is found under these two temperatures because the drying phase at 40°C induces an increase in the mortar resistivity and limits the macrocell corrosion process [592].

Effect of irradiation no relevant in case of cracks corrosion specifically, in comparison with other factors.

Effect of microbiological reactions Not relevant for NPP civil engineering structure as cooling tower and containment building exposed to external conditions. Microbiological reaction could occur in hydraulic concrete structures for the cooling circuit in case of sea water or river water but firstly the sulphate and nitrite concentration is very low and secondly no article refer of specific effect on cracks [593].

Effect of exposure condition (free, sheltered, buried, submerged) no specific information found

Effect of external water composition (submerged or buried structures) For a long time it was considered that the chloride concentration had an influence on the phenomenon of corrosion at the bottom of the crack and that repassivation only appears to be possible when no chlorides are present [553]. A 2020 study showed that the same crack-sealing phenomenon appeared in the case of a salt spray attack [594].

No information found in case of leaching with both free-chloride water and water chloride.

Effect of atmospheric conditions (rain, wind, freeze/thaw) Atmospheric conditions like rain direction and prevailing wind represent an important issue for structure such as cooling tower and containment building.

Cracks exist on the extrados of a cooling tower because of gradients of both humidity and temperature between the extrados and intrados and also because of loading actions such as wind and stresses due to differential settlement at the foundations [595]. Another study shows for cooling towers, the importance of the state of cracking of the shell and of the differential settlement on its strength. In this specific study corrosion is not considered, but only the cracks contribution on mechanical strength [594].

Experiments in laboratory show that corrosion is mostly dependent on the number of spray-drying cycles after initiation and not of the cycle duration [549, 596]. However, carbonation-induced corrosion rate is independent from the crack orientation with respect to rain [588].

Freeze-thaw induce deterioration of concrete, can accelerate cracks formation and the initiation phase of corrosion process. However, up to now no study was provided on the effect of freeze-thaw on corrosion kinetic (induction phase) when the crack is open.

CO2 Concentration of carbon dioxide in atmosphere is the main parameter of concrete carbonation also in case of not yet open cracks. Recent research show that when crack is open, carbonation does not propagate along the rebar, but is limited in front of the rebar and just where the concrete is mechanically damaged [549].

O2 Oxygen is essential for depassivation of steel and to initiate corrosion as soon as the crack is open [511].

Effect of mechanical and structural loading Mechanical and structural loading represents one of the main mechanical effect that can induce cracks initiation, opening and re-opening during the structure life. Moreover, it is the main way (mechanical flexion) in laboratory to generate and control cracks opening to study corrosion. Recent study shows that reopening of cracks induces an immediate increase of the current of corrosion but for a very short time and quickly after a return to previous low current value [597].

It concludes that one reopening has no visible effect on current of corrosion whatever the crack width (100 µm and 500 µm) [597, 598].

The effect of sustained or repetitive loading on macroscopic cracking, however, can be an important consideration in the serviceability of reinforced concrete members, especially in terms of corrosion of reinforcing steel and appearance.

4.18.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) Because cement type is one of the parameters influencing concrete quality and mechanical performances it has an effect of cracks development. Moreover, corrosion length along the rebar depends on concrete quality. For instance, experiments show that the corrosion length around rebar in case of pre-cracked specimen in concrete and (CEM I + slag) mortar specimens is lower than the one measured in CEM I mortar specimens. The difference noted in the corrosion lengths measured in specimens containing slag may be related to the high resistivity of the medium surrounding the rebar which limit the flow of hydroxyl ions from the cathodic zone to the anodic one [588].

However, corrosion kinetics seems not to be different with binder.

Cement-water binder ratio This ratio is one of most important parameters influencing porosity and porosity impacts mechanical resistance and chemical agent penetration. So indirectly, water-binder ratio can facilitate the appearance of cracks especially at young age.

Carbonation speed increases with porosities and cathodic reactions may also be increase because of the higher of oxygen due to porosity. Because porosity is bigger on the lower part of the rebar than on its upper part following the cast direction, the thickness of the corrosion products layer is higher. The greater interface damage and numerous pathways to the interface via secondary cracks result in a larger surface of the rebar affected by corrosion [596].

Concrete composition (aggregate content, types of aggregates) Not relevant

Curing (temperature, moisture, time) conditions Curing represents a very small part of concrete structure life, but is essential considering cracks at young age. Among all the parameters that can disturb the hydration of the cement, the temperature is the one that has the great impact [511].

4.18.2.3 Other influential factors

Effect of cavities filled with air - Corrosion is observed preferentially at defects (cavities filled with air or poral solution) of the steel-mortar interface [596].

Effects of (surface) cracks – European standard EN 1992-1-1 [590] risk of corrosion codes specifies that the maximum crack opening is about 300µm (in loaded conditions) for the XC4 concrete class. But, recently, several studies tend to show that cracks width up to 0.5 mm have the same behaviours as lower ones: whatever the cracks width, after a certain number of raining/drying cycles, a decrease in the corrosion rate is detected. This is attributed to rebar repassivation which is due to the corrosion products that seal the cracks and act as a protective layer for the rebar by limiting the access of aggressive agents. During the repassivation phase, the rate of the corrosion process is independent from the residual crack opening. This hypothesis that the formation of corrosion products plugs the crack tip and limit the access of oxygen and water to the reinforcements, thus slowing down the corrosion process is increasingly favoured. Even if corrosion processes are different, that observation is the same for chloride corrosion and carbonation corrosion [596].

The crack width: The crack width mostly appears only to influence the initiation of corrosion. But crack width does not influence de corrosion rate and does not limit the sealing phenomenon previously described. The ratio between the width of the cracks on the surface and at the level of the reinforcing bars is between 1/2 and 1 depending if you consider mortar or concrete [549, 599].

Carbonation propagation: the carbonated area is limited to the concrete mechanical damaged zone around the rebar and does not progress along the rebar in case of good bonding of the binder and before the seal formation [549, 574].

Cracks direction: crack perpendicular to the rebar has lower effect than cracks parallel to the rebar. In fact, corrosion cracks are generally parallel to the rebar and this promotes homogenous carbonation deep in the crack [549].

4.18.3 Influence of other ageing processes

4.18.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) not relevant in NPP.

Leaching (submerged or buried structures) no information in NPP

Alkali-aggregate reactions Can act as cracks initiation.

Carbonation The influence was discussed above.

Ettringite and thaumasite reactions (DEF/sulphate attack) Can act as cracks initiation.

Bacterial processes not relevant in NPP.

4.18.3.2 Physical-Mechanical

Freeze-thaw Detailed above

Elevated and high temperature (<150°C, no fire) not relevant.

Irradiation not relevant.

4.18.3.3 Mechanical

Abrasion/Erosion/Cavitation not relevant.

Creep and relaxation can induce cracks but no relevant for corrosion process inside the cracks

Settlements and movements can induce cracks but no relevant for corrosion process inside the cracks

Vibration (and seismic) can induce cracks but no relevant for corrosion process inside the cracks

Thermal stresses (gradients) can induce cracks but no relevant for corrosion process inside the cracks

4.18.3.4 Electro-chemical

Pitting corrosion not relevant.

General corrosion not relevant.

4.18.4 Rates of deterioration

Not relevant on corrosion considering sealing effect previously presented.

4.18.5 Impact on concrete properties

Chemical – Pore water chemistry – no relevant, cracks have only local impact on pore water chemistry.

Chemical – Solid phase composition – no relevant.

Structural – Microstructure – no relevant.

Structural – Cracking – no relevant.

Transport properties – Porosity, permeability, diffusion coefficients – Cracks have only local impact on transport properties.

Mechanical properties – Of course multiplication of cracks represents mechanical weakness and a resistance loss, which is more du binder discontinuity instead of corrosion development inside the cracks.

4.18.6 Assessment methods

4.18.6.1 Visual inspection

In NPP, visual inspection is essentially based on video measurement with drone, followed by manual inspection. This method is acclaimed for the crack and is depends on the specialist's knowledge and experience, it lacks objectivity in the quantitative analysis.

4.18.6.2 Continuous monitoring

Due to the unpredictable nature of its location, the cracks line monitoring on civil engineering infrastructure appears not possible. For experiments in laboratory, all well-known methods based on potential measurements or current measurements or electrochemical impedance spectroscopy Sallow the monitoring of the rebar corrosion kinetic [597].

4.18.6.3 Destructive testing of sampling

Not relevant

4.18.6.4 Non-destructive techniques

Automatic image-based crack detection is proposed as a replacement: Infrared and thermal testing, Ultrasonic testing, Laser testing, and Radiographic testing… All of them are based on image processing but the crack detection criteria can be based on different parameters: surface (most often), length, width, depth, direction of propagation [600].

Remark: the link between cracks and leak rate is not understood essentially because of the difficulty of modelling cracks. In case of containment building, the global leak rate is determined by measuring the pressure of the dry air (corrected for the partial pressure of water vapor obtained by measuring the hygrometry) contained in the enclosure, by measuring the temperature in various interior places to the enclosure and by applying the ideal gas law: the variation of the quantity PV / T over time makes it possible to know the loss of mass over time and therefore the leakage rate defined as the ratio of the mass d air having escaped from the enclosure for a period of 24 hours out of the total mass of pressurized air present in the enclosure [601].

4.18.7 Performance indicators & acceptance criteria

No Information

4.18.7.1 Reparability

To limit the penetration of carbon dioxide, it is possible to apply protection products [573] on the concrete surface to completely or partially seal pores and micro-cracks. These products are classified by the standard NF EN 1504-2 in three families [574].

A study of the effectiveness and long-term behaviour of several concrete surface protection products with regard to all phenomena leading to carbonation-induced corrosion shows that one product stands out, the acrylic coating [598].

4.18.8 Model approaches

4.18.8.1 Empirical models

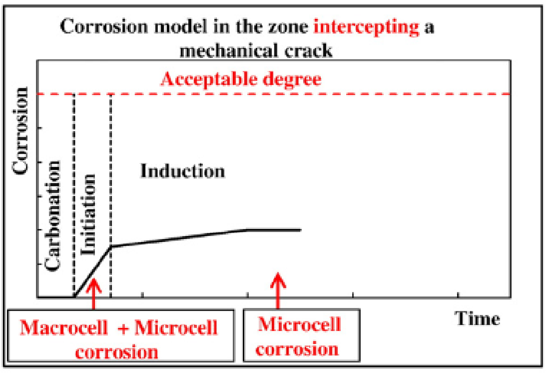
No information

4.18.8.2 Phenomenological models

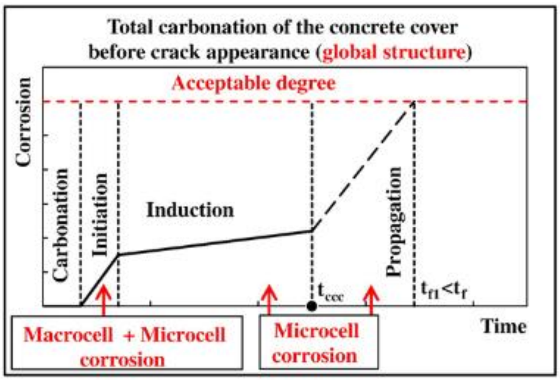
A summarize of the main models described in [553, 585, 602] are proposed and completed in the following:

- Model assuming that corrosion will not have any detrimental effect on the durability of structures (Figure 25a).

- Model assuming that cracks cannot appears before total carbonation of the concrete cover (Figure 25b).
- Model assuming that corrosion cracks can appears before the total carbonation of the concrete cover based on 5 steps (Figure 25c):
    - Carbonation: time for carbon dioxide to penetrate the crack and access the steel / binder interface.
    - Initiation: corrosion begins along the reinforcement intercepting a carbonated concrete.
    - Induction: this period begins when the corrosion kinetics decrease and will be controlled by the corrosion products developed at the bottom of the crack.
    - General carbonation of the concrete cover.
    - Propagation: the period during which the corrosion kinetics increase again and the corrosion propagates until reaching an acceptable limit degree.



(a)



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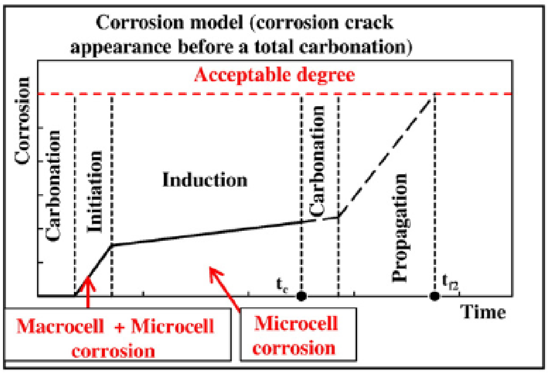

(c)

Figure 25. Cracks corrosion models [603].

4.18.8.3 Complex coupled models

No information

4.19 Crevice Corrosion

4.19.1 Process Definition

Crevice corrosion is another form of localized corrosion usually associated with a stagnant solution on the micro-environmental level that occur in specific isolated or shielded areas. In essence, crevice corrosion is due to a differential oxygen access between two areas of rebar. In a crevice, the liquid convection is limited. Oxygen in the liquid which is deep in the crevice is quickly consumed by reaction with the metal inducing anode/cathode process and metal dissolution When oxygen content of liquid at the mouth of the crevice which is exposed to the air is greater than deeper in the crevice, a local cell develops in which the anode, or area being attacked, is the surface in contact with the oxygen-depleted liquid.

Comparison with pitting corrosion:

- Crevice corrosion presents the same localized initial steel depassivation as pitting corrosion but it presents some differences.
- Both forms of corrosion occur in a chloride-rich environment. However, crevice corrosion can also occur in acid environments.
- Pitting corrosion can occur in stagnant or circulating solutions, at least at low speed. On the other hand, crevice corrosion is widely favoured in stagnant environments;
- Pitting corrosion can appear in all areas of the metal surface whereas crevice corrosion is limited to confined areas.
- Crevice corrosion generally occurs at a lower potential and with a shorter initiation time than pitting corrosion.

Crevice corrosion can appear on prestressing reinforcements that are produced in a stranded geometry, around welding defects, or other rebar mechanical defect where pore water can stagnate.

References: For further references, the reader can consult [604, 605].

4.19.2 Influential factors

4.19.2.1 General conditions

Effect of moisture content and relative humidity Relevant considering the role of pore solution in all the corrosion processes.

Effect of temperature A lower increase of temperature triggered and exacerbate the crevice corrosion [606]. In chloride environment at 0.1 M, and in the case of classical steel rebar HRB400:

- Temperature in the range under 30°C has no specific influence on crevice corrosion.
- 35°C is defined as the critical temperature for crevice corrosion.
- 45°C is considered as the temperature that accelerates the rapid anodic dissolution.

Effect of irradiation Not relevant

Effect of microbiological reactions Not relevant

Effect of exposure condition (free, sheltered, buried, submerged) No specific information found

Effect of external water composition (submerged or buried structures) Relevant in case of marine environment for instance considering the chloride source that can filled crevices and voids.

***Effect of chloride**:* Chloride concentration is one of the main parameter to be considered for crevice corrosion. In the case of classical steel rebar HRB400, and at room temperature, concentration of chloride ions under 0.2 mol/l does not influence crevice corrosion. The critical chloride concentration threshold that accelerated crevice corrosion is 0.2 mol/l [606].

Effect of pH: In comparison with general corrosion due to carbonation front an ad pH decrease, the evolution of pH concentration inside the crevice has to be more considered as consequence of the corrosion initiation due to oxygen depletion rather than as a the initiation parameter. However, in carbonated concrete, the pH value will gradually decrease with time, particularly in the region with micro-crack defects. Consequently, the crevice corrosion risk will increase more quickly than that of pitting corrosion according to [606].

Effect of atmospheric conditions (rain, wind, freeze/thaw)

O2 Oxygen in the pore solution is one of the trigger of the crevice corrosion. Its depletion in one part of the crevice generates the displacement of the cathodic reaction to another parts of the steel surface which maintains the anodic reaction inside the crevice.

Effect of mechanical and structural loading No effect

4.19.2.2 Mixed design properties

Cement type and supplementary cementitious materials (SCM) No information found and not really a main parameter.

Cement-water binder ratio Because this ratio can influence porosity and as a consequence the saturation rate it can play a role in crevice corrosion, but it is not a main parameter. The porosity has a same influence as defined above considering the role of porosities connected to crevices.

Concrete composition (aggregate content, types of aggregates) A high aggregate to binder ratio or a lack of binder in an area with a high density of rebar is a factor inducing voids, and stagnant pore water that initiate crevice corrosion.

Shape and size of aggregates determine different contact points with the rebar that can be the sites of crevice corrosion.

Remark: In civil engineering for specific reasons (i.e. better drainage of water, thermal insulation, low density), precast concrete blocks with high aggregate/binder ratio are used. But this is not relevant for NPP [607].

Curing (temperature, moisture, time) conditions Relevant considering that curing affects porosities size and pore water content.

4.19.2.3 Other influential factors

Effect of crevice shape Crevice corrosion is more likely to occur in narrow and deeper crevice into which oxygen is more difficult to diffuse

***Effect of stagnate pore solution**:* As previously presented, the main parameter for the crevice corrosion is the lack of liquid convection.

4.19.3 Influence of other ageing processes

4.19.3.1 Chemical/biological processes

Attack by aggressive water, acid-base attack (industrial processes) Not relevant.

Leaching (submerged or buried structures)

Alkali-aggregate reactions No information.

Carbonation Not relevant, previously detailed.

Ettringite and thaumasite reactions (DEF/sulphate attack) Not relevant.

Bacterial processes Not relevant.

4.19.3.2 Physical-Mechanical

Freeze-thaw Not relevant.

Elevated and high temperature (<150°C, no fire) Not relevant.

Irradiation Not relevant.

4.19.3.3 Mechanical

Abrasion/Erosion/Cavitation Not relevant.

Creep and relaxation Not relevant.

Settlements and movements Not relevant.

Vibration (and seismic) Not relevant.

Thermal stresses (gradients) Not relevant.

4.19.3.4 Electro-chemical

Pitting corrosion Not relevant.

General corrosion Not relevant.

4.19.4 Rates of deterioration

Experiments on crevice corrosion of steel rebar in concrete or in a representative pore solution are not very numerous and the rate of corrosion was not studied. However, rate of deterioration was evaluated for stainless steel used for prestressed wires by [605]. He estimated that the decrease of the time to corrosion initiation could reach 34% when chloride threshold level is exceeded.

4.19.5 Impact on concrete properties

Chemical – Pore water chemistry Crevice corrosion influences pH in the solution around the crevice.

Chemical – Solid phase composition Not relevant.

Structural – Microstructure Not relevant.

Structural – Cracking Crevice corrosion induces steel rebar dissolution, corrosion products increase as described in previous chapter and at last cracks.

Transport properties Porosity, permeability, diffusion coefficients –

Mechanical properties The main crevice corrosion effect is the reduction of the rebar section inducing a reduction of the mechanical resistance.

4.19.6 Assessment methods

The same assessment methods as those for cracks (section 4.18.6) or pitting corrosion (section 4.16.6) can be used for crevice corrosion.

4.19.6.1 Visual inspection

In NPP, visual inspection is essentially based on video measurement with drone, followed by manual inspection. This method is acclaimed for the crack and is depends on the specialist's knowledge and experience, it lacks objectivity in the quantitative analysis.

4.19.6.2 Continuous monitoring

Due to the unpredictable nature of its location, the crevice corrosion on line monitoring on civil engineering infrastructures appears not possible. For experiments in laboratory, all well-known methods based on potential measurements or current measurements or electrochemical impedance spectroscopy Sallow the monitoring of the rebar corrosion kinetic [597, 606] [].

4.19.6.3 Destructive testing of sampling

4.19.6.4 Non-destructive techniques

Automatic image-based crack detection is proposed as a replacement: Infrared and thermal testing, Ultrasonic testing, Laser testing, and Radiographic testing… All of them are based on image processing but the crack detection criteria can be based on different parameters: surface (most often), length, width, depth, direction of propagation [600]

4.19.7 Performance indicators & acceptance criteria

No information found.

4.19.7.1 Reparability

In civil engineering, there is no specific reparability process for crevice corrosion.

4.19.8 Model approaches

Crevice corrosion models were essentially developed for metallic structures in various environment and for various type of materials. For a wide overview of chemical reaction involved, mass transfer reaction and different models developed, reader can refer to [608].

4.19.8.1 Empirical models

No specific information

4.19.8.2 Phenomenological models

For carbon steel rebar, Ghods [609] proposed a mechanism in several stages describing the crevice corrosion induced by defect on the mill scale on the rebar surface:

- Initiation that corresponds when oxygen is readily available
- Depassivation of the rebar following by the local acidification of the solution.
- Propagation that corresponds to the metal dissolution. One specificity of crevice corrosion is that the process is an autocatalytic process.

For prestressing strands, Moser [610] proposed a model to described mechanism based on:

- Crevice corrosion initiation at the impingement sites between adjacent prestressing wires once Cl– concentration exceeds the CTL (Figure 26a).
- Localized corrosion continues to occur at crevice sites accompanied by acidification of the interstitial region of the prestressing strand, as indicated by a change in color from black to white (Figure 26b).
- Once corrosion products have built up a sufficient amount such that the mass transport of reactants to crevice sites is limited, corrosion attack spreads to the surface of the strand (Figure 26c).

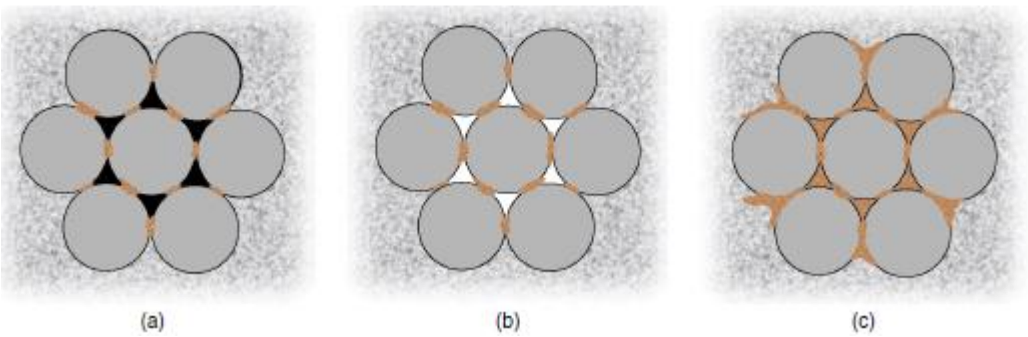

Figure 26. Model for corrosion initiation in prestressing strands [610].

4.19.8.3 Complex coupled models

No information found applied to concrete.

A very few literature deals with crevice corrosion models in case of concrete or civil engineering applications.

Table 12. General crevice corrosion models [608].

Model	Input	Theory / idea	Output
Empirical models			
Hakkarainen	Own experiments	Simple mathematical equations for the critical Cl- concentration for pit initiation	The probability of an attack is estimated by the effective chloride content
Statistical models			
Williams et al.	Mathematical equations	Pitting is a series of events which were randomly distributed in time and space	Potential and concentration distributions
Laycock et al.	Potentiostatic and potentiodynamic experiments	Deterministic model for the growth of single pits combined with a purely stochastic model of pit nucleation (Monte Carlo)	Pitting potentials
Mechanistic models			
PDM	Concentration of aggressive species and inhibiting anions	Breakdown of passive film varies with concentration of aggressive species and inhibiting anions	Dependency of pit initiation potential on Cl- concentration
Sharland et al.	Experimental data by Beavers and Thompson	Diving the crevice into a series of finite elements (FEM)	Rate of enlargement of pits or crevices as a function of metal potential, external solution chemistry & crevice dimensions
Heppner et al.	Experimental data of Alavi and Cotbs	Computer algorithm adjusts crevice solution composition for chemical and electrochemical equilibrium	Crevice solution composition as a function of space and time
Sun et al.	Experimental data by Alavi and Cotta	COMSOL Multiphysics	Dynamic pH distribution inside the crevice and corroded crevice shape
Engelhardt et al.	Thermodynamic and kinetic data from elsewhere	Coupled environment approach	Concentration and potential distributions inside and outside crevice, critical permeability coefficient *Kcr*
Engelhardt & MacDonald	Mathematical equations	Potential distribution inside a crevice obeys Ohm's law	*Wmax* as a function of time and position from the anodic polarization curve
Walton et al.	Experimental data by Alavi and Cottis. and Valdes-Mouldon	Mathematical reactive transport model	Potential. pH & anodic current density distribution
Stewart	Own experiments + Valdes-Mouldon data	CF approach + CREVICER	Chemical concentration and potential fields of crevices
Kennel I et al.	Experimental data of Alavi and Cottis	IRDT+CCS theories	Predict the dynamic crevice pH profile
Anderkoet al	Stainless steel 316L. AL6XN and alloy C-276 tests	Repassivation theory	*Erp* and *Ecorr* from pitting corrosion rates

5. Challenges

Degradation processes without further research needs and challenges

For settlements and movements, after review of the settlement monitoring and related issues regarding concrete performance at Czech NPPs, there are no distinct challenges nor gaps. The reason is that the foundations get proper attention during design, the geotechnical conditions are investigated with utmost diligence and the foundation performance is monitored and diagnosed during construction and afterwards [495, 611, 612]. Another aspect is the huge mass of concrete foundations under the nuclear facilities which prohibit appearance of commonly encountered degradation phenomena.

For vibration, after review of the vibration monitoring and related fatigue of concrete at Czech NPPs, there are no distinct challenges nor gaps. The reason is that most of the vibrating pipes are damped by viscous dampers or springs and far from enough from the concrete surface to have any thermal effect. The vibrating turbo-generator foundation slab is massive and so the stress level from vibration is negligible. The earthquake is non-existent in Czechia. Potentially, there may be some issue later on with the anchor zones of vibrating machinery, such as pumps, if any.

Similar opinion can be found in Do and Chockie [494]: “Because of the typically low normal stress levels in reinforcing steel elements in NPP safety related concrete structures, fatigue failure is not likely to occur.”

For *general corrosion,* it does not represent a challenge for NPP civil engineering structures considering their actual surveillance and the role of the authorities.

General remark

For *corrosion* process in general, some main knowledge gaps and challenges to further research that limit our current understanding all corrosion phenomena for NNP are described in L'Hostis and Gens [17].

Understanding

Understanding the development of degradation and corresponding pathologies in massive structures (being complex due to e.g. hydric, thermal and chemical gradients and linked with processes such as drying, leaching, mechanical loading), relevant to NPP is crucial for different processes, relevant for

- AAR
- DEF
- Crack corrosion

For some processes, there is further need for experimental data or studies for specific phenomena:

- Irradiation: Rate Effects/Annealing, requires the characterization of in-service LWRs' concrete at high dose, i.e., > 10^{19} n.cm^{-2} E > 0.1 MeV with substantial silica content in the aggregate
- Irradiation: Irradiated Steel-Concrete Bond Strength; possible loss on bond due to the irradiation-induced damage of concrete around reinforcement bars and anchorages
- Irradiation: Irradiated Concrete Creep; Higher irradiated concrete creep would limit/prevent irradiation-induced cracking
- Creep and Relaxation: It is still not well understood what physical phenomena create the Pickett effect (desiccation creep, section 4.12.2.1)
- Pitting corrosion: The impact of features of the steel-concrete interface on pitting corrosion are largely unknown and need to be further studied.

- Pitting corrosion: The effect of steel surface condition and steel microstructure on the pitting corrosion processes in concrete is understudied and requires more attention.
- Crack corrosion: Parallel cracks that pose a more serious threat to reinforcement than transverse cracks, as they sustain higher corrosion rates
- Crack corrosion: Behaviour of cracks submitted to dynamic loading inducing a variation in the crack widths. Indeed, the variation in the crack width may inhibit the sealing effect of the corrosion products and inhibit thus the repassivation
- Crack corrosion: Comparison between real cracks shape on structures with those obtained in laboratory (by flexion) in order to verify if they are in accordance: shape, profile, damage length along the rebar
- AAR: All three stages of progression are poorly understood: (1) The exact features resulting in aggregate dissolution remain ambiguous, (2) the influence of both aggregate and cement paste chemistry on resulting AAR gel mechanical properties remains ambiguous, and (3) the mechanism and kinetics of resulting swelling and damage (dependent on moisture and mechanical properties) also remains ambiguous. These are all compounded by a lack of standardization in assessing and cataloguing relevant data, both in standard practice and laboratory studies.

Even for processes that are well understood, specific gaps consist

- Consequences of degradation effects on safety relevant SSCs in NPP is not well-documented (or not being identified) in literature – as such it is not recognized that and under which conditions the consequences can cause severe deterioration of concrete structures. This is relevant for
    - Microbiological processes
- Understanding the process for the very specific conditions in a NPP:
    - Deterioration of concrete under high temperature in the steam exhaust rooms with very high humidity. As a result, these concrete structures will be damaged more easily, under the influence of temperatures of up to 150°C, compared to structures located in a dry environment.
- To formalize this understanding in a context where it interacts with other processes:
    - Acid-induced dissolution of calcium-bearing cement phases with other processes
    - The coupling between AAR and creep/shrinkage especially in the case of bi-axially loaded structures (such as the NPP containment building)
    - The coupling between DEF and creep/shrinkage especially in the case of bi-axially loaded structures (such as the NPP containment building)
    - Effect of coupled deterioration mechanism on freeze-thaw loading, for example linked to alkali aggregate reactions or leaching
    - The drying, creep, and shrinkage behaviours at high temperatures
    - The synergistic effect of carbonation and chloride ingress is not well studied and little is known how both processes effect corrosion at the same time
    - Crack corrosion and the behaviour of cracks exposed to icing and thawing and chemical attacks

- The combination of several factors that influence degradation processes must be considered with comprehensive and satisfactory studies on this complexity e.g. for
    - AAR
    - DEF
    - General corrosion

Measuring

Some processes lack well-established measurement methods:

- Microbiological processes: Beside visual inspection of biofilm production, and in case of sever attack by sulphuric acid formation of gypsum – not much information on

assessment methods has been found for microbiological processes; it is expected that methods identified for acid attack and leaching could be used. Note that identification of concrete degradation fronts in case of physical degradation might be even more difficult because of the heterogeneous nature of the material (e.g. thermal stresses).
- Selective detection of crevice corrosion in concrete

Measuring degradation and degradation fronts with non-destructive techniques is a current gap, specifically for massive structures. Besides the development of in-situ and non-invasive measurement devices, both for laboratory experiments as for structure monitoring, their reliability for the examination of massive concrete structures should be demonstrated. In case sensors are the most preferred option (e.g. monitoring in-situ pH for general corrosion), their long-term durability should be addressed. In case inspections and repairs can be done only during plant downtime, methods should be executed fast and efficient.

Further research on non-destructive (in-situ) testing is relevant for:
- Measuring the leaching depth
- AAR
- DEF
- abrasion, erosion or cavitation investigation
- pitting corrosion
- general corrosion

Monitoring the development of the degradation process in massive structures is relevant for:
- AAR
- DEF

The requirement for fast and efficient measurement techniques is relevant for:
- Elevated temperatures; the concrete that is exposed to high temperatures (up to 150°C) is situated in the steam exhaust rooms (short access at limited times)

Modelling

There is a lack on computer models for some processes:
- concrete abrasion
- prediction of the emergence and evolution of ageing zones (degradation, destruction) of concrete due to temperature gradients

For some processes, a number of conceptual models exists to describe the process. Straightforward application is not always possible, e.g. by lacking quantitative information, and applicability for NPP has not been always demonstrated as well.

Prediction of degradation kinetics (for particular concrete properties and environments) is not straightforward, such as for
- Carbonation
- Microbiological processes

In addition, other challenges are
- To build and adapt universal models including all occurring phenomena or relevant processes, such as for:
    - Microbiological processes
    - Freeze-thaw damage both in the presence or not of slat solution
- To couple different processes such as
    - chemical (e.g. leaching) with electro-chemical (e.g. corrosion) processes, and even more coupling with mechanical degradation due to reaching.

- To have readily applicable models that can be used for the case of accidental situations in concrete containment buildings for:
  - Creep and Relaxation

Specific studies to improve the mechanistic modelling:

- General corrosion:
  - Monitored more parameter than electrochemical data for instance resistivity of the material, presence/amount of water separately and specifically.
  - Study more specifically the influence of the pore size distribution of the carbonated mortar or concrete samples that can change with the type of binder and w/b ratio, on all the coupled parameters influencing the corrosion: water content, oxygen diffusion to the steel, electrical resistivity.

Although some models perform well for standard ordinary Portland cement systems, models could not predict well for blended systems with supplementary cementitious materials. This is relevant for:

- Carbonation

To increase confidence in modelling, benchmarks of one or different models using experiments are required, relevant for

- AAR
- DEF
- Validation of irradiated concrete models against test-reactor data
- General corrosion

Demonstration of application of e.g. complex coupled reactive transport models for conditions relevant in NPP, e.g.

- Microbiological processes

Assessment

Even for rather some well understood processes, assessments often hamper because of insufficient details on parameters relevant to a concrete’s properties in relations to its service environment:

- acid-induced dissolution of calcium-bearing cement phases
- general corrosion

A step forward is the development of a database of affected concrete based on concrete parameters and environmental conditions. This is relevant for

- AAR
- DEF

Assessing concrete performance by testing is sometimes difficult, because of the characterization is a very lengthy process. This hampers optimization during the preparation phases of new construction techniques. It would be useful to optimise the characterization technique to decrease this time. This is for example relevant for:

- Creep and relaxation

Practical prediction models for engineering phenomena through standardization in internationally recognized codes are for some processes lagging behind compared to some other engineering phenomena, for example for

- Carbonation

Re-assessment of existing structures with new data or changing standards :

- Vibration/seismic when new reference requirements are adopted

Assessment of migitating actions:

- Crack corrosion - tests of protective coating start in laboratory but need to be address on long time
- Crack corrosion - Effect of cover thickness especially in case of low thickness (less than 20 mm)

Performance indicators and acceptance criteria

For some processes, performance indicators and acceptance criteria are not established:

- Microbiological processes; it is expected that they could be similar as those that are defined for acid attack and leaching. However, this has to be confirmed.
- Methodological guideline to assess the performance of irradiated concrete in LWRs

Developing of realistic testing for evaluating performance:

- Freeze-thaw for aged concrete

Risk evaluation

- Evaluation of the crevice corrosion risks in comparison with other localized corrosion process in concrete

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7. Appendix

7.1 Appendix Chapter 2

7.1.1 Figure 3 in [12]

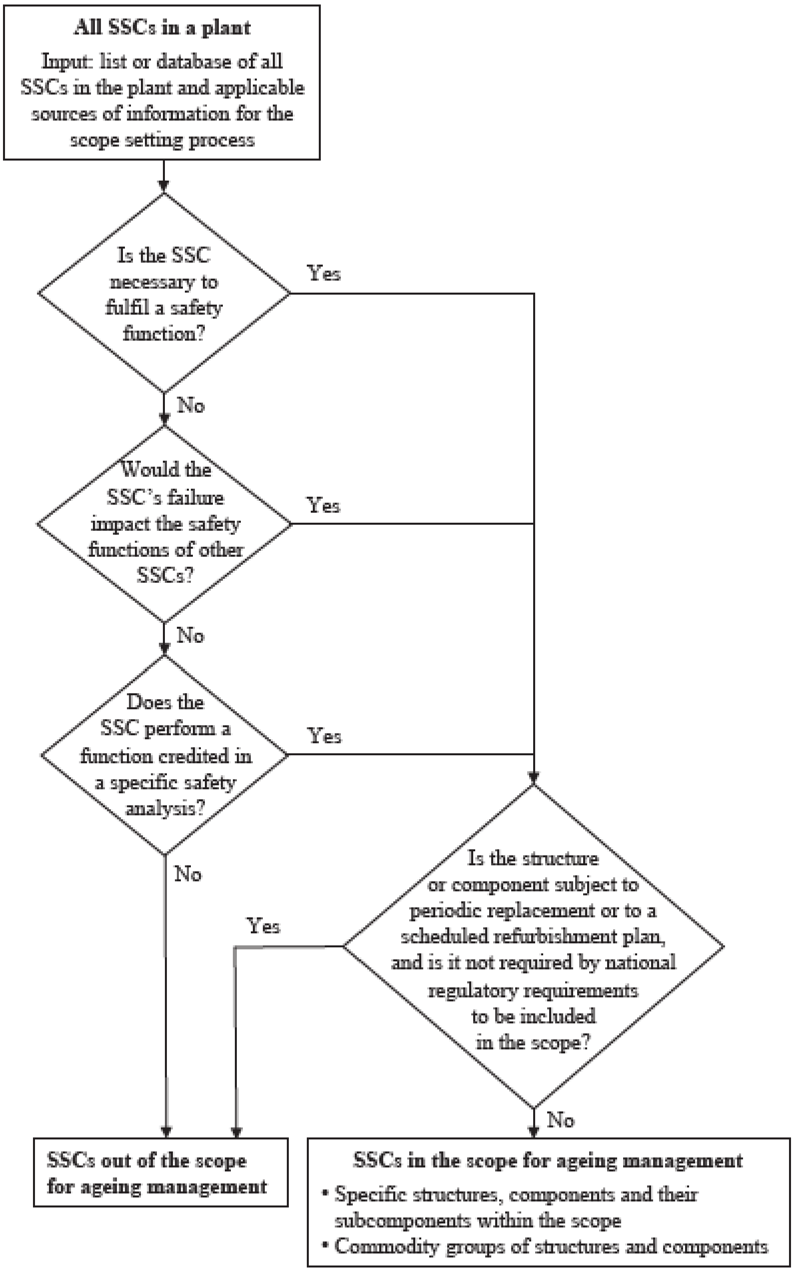

FIG. 3. Scope setting process for ageing management.

7.1.2 Table 1 in [11]

Table 1. Typical safety-related concrete structures in LWR (light water reactors) plants and their accessibility for visual examination [6].

Concrete structure	Accessibility
Primary containment	
Containment dome/roof	Internal liner/complete external
Containment foundation/basemat	Internal liner (not embedded) or top surface
Slabs and walls	Internal liner/external above grade
Containment internal structures	
Slabs and walls	Generally accessible
Reactor vessel support structure (or pedestal)	Typically lined or hard to access
Crane support structures	Generally accessible
Reactor shield wall (biological)	Typically lined
Ice condenser dividing wall (ice condenser plants)	Lined or hard to access
NSSS equipment supports/vault structures	Generally accessible
Weir and vent walls	Lined with limited access
Pool structures (reactor, fuel, condensation)	Lined with limited access
Diaphragm floor	Lined with limited access
Drywell/wetwell slabs and walls	Internal liner/partial external access
Secondary containment/Reactor buildings	
Slabs, columns, and walls	Accessible on multiple surfaces
Foundations	Top surface
Sacrificial shield wall (metallic containments)	Internal lined/external accessible
Emergency cooling water structures	
Cooling water channels	External surfaces above waterline
Water wells/pools	Limited accessibility
Turbine building	Generally accessible

7.1.3 Table 11 in [2]

TABLE 11. POTENTIAL PLANT DATA SOURCES

Type of data	Sources	Information
Baseline	Design calculations	Service life Design philosophy Design codes/standards Material design properties Design stresses/strains Static design loading Dynamic design loading Hazard design loading Environmental assumptions
Construction and commissioning	Construction and record drawings	Substructure (foundations) Superstructure Fabric and finishes Construction details Construction sequence
	Specifications	Construction standards Material sources Material properties Level of QA/inspection/testing Construction sequence Construction methods
	Designers/contractors	Design variations Specification variations Temporary works Temporary loads Construction history Levels of supervision
	Quality control records	Certified material test records Performance test results for prestressing tendons Liner acceptance test results Jacking data for prestressing tendons
	Pre-operational test records	Structural integrity test records Leakage test records Polar crane test records
Operational history	Plant operating procedures	Service loading Environmental conditions Fault loading Safety procedures Maintenance procedures
Inspection and surveillance	Inspection records	Visual inspection data Leakage rate tests Ultrasonic thickness tests for liner Prestressing tendon metallurgical tests Prestressing loads Monitoring instrumentation data
	Plant management/operations	Plant history Maintenance history Environmental condition history

7.1.4 Figure 5 in [12]

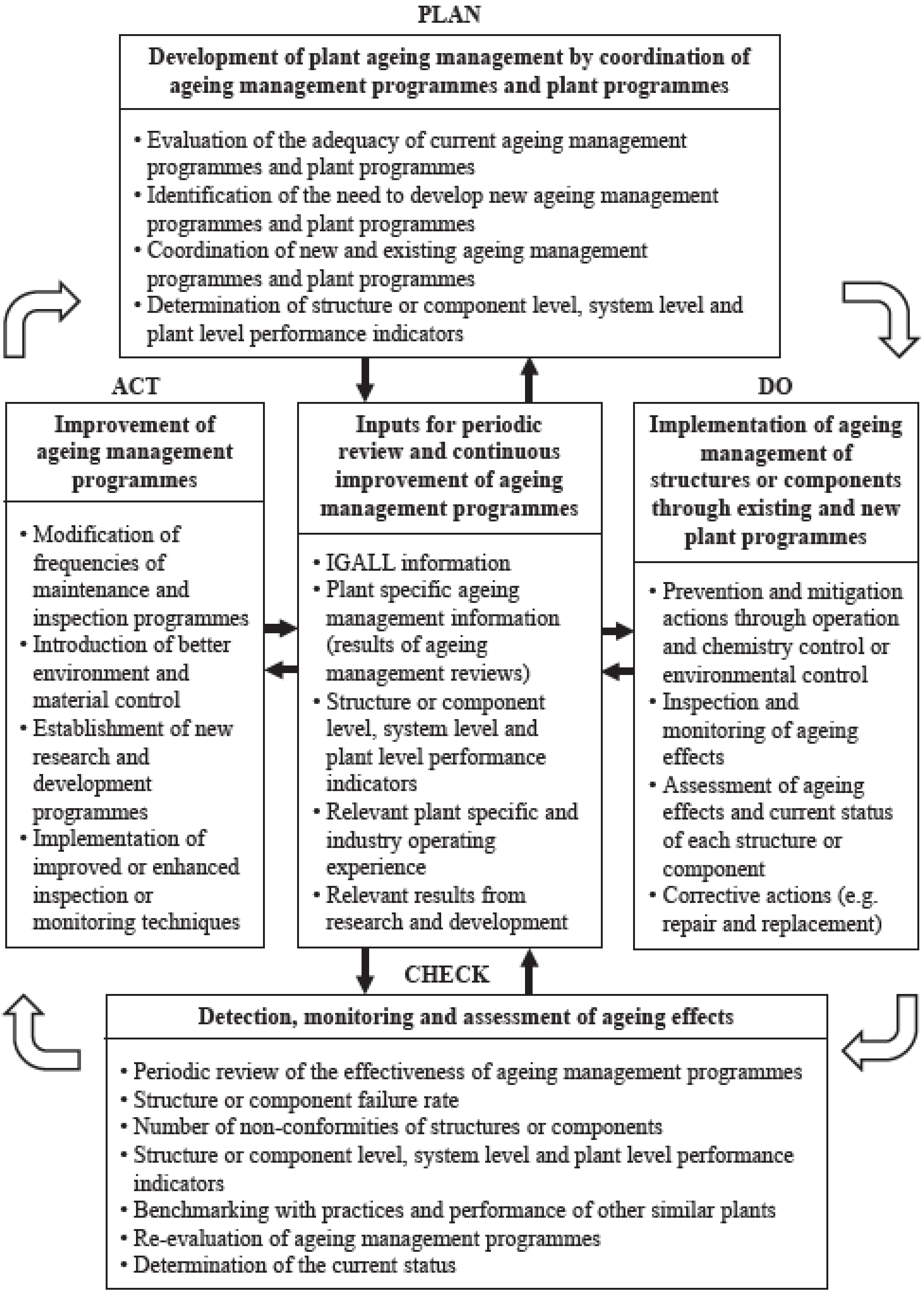

FIG. 5. Development, implementation, review and improvement of ageing management programmes.

7.1.5 Figure 4 in [1]

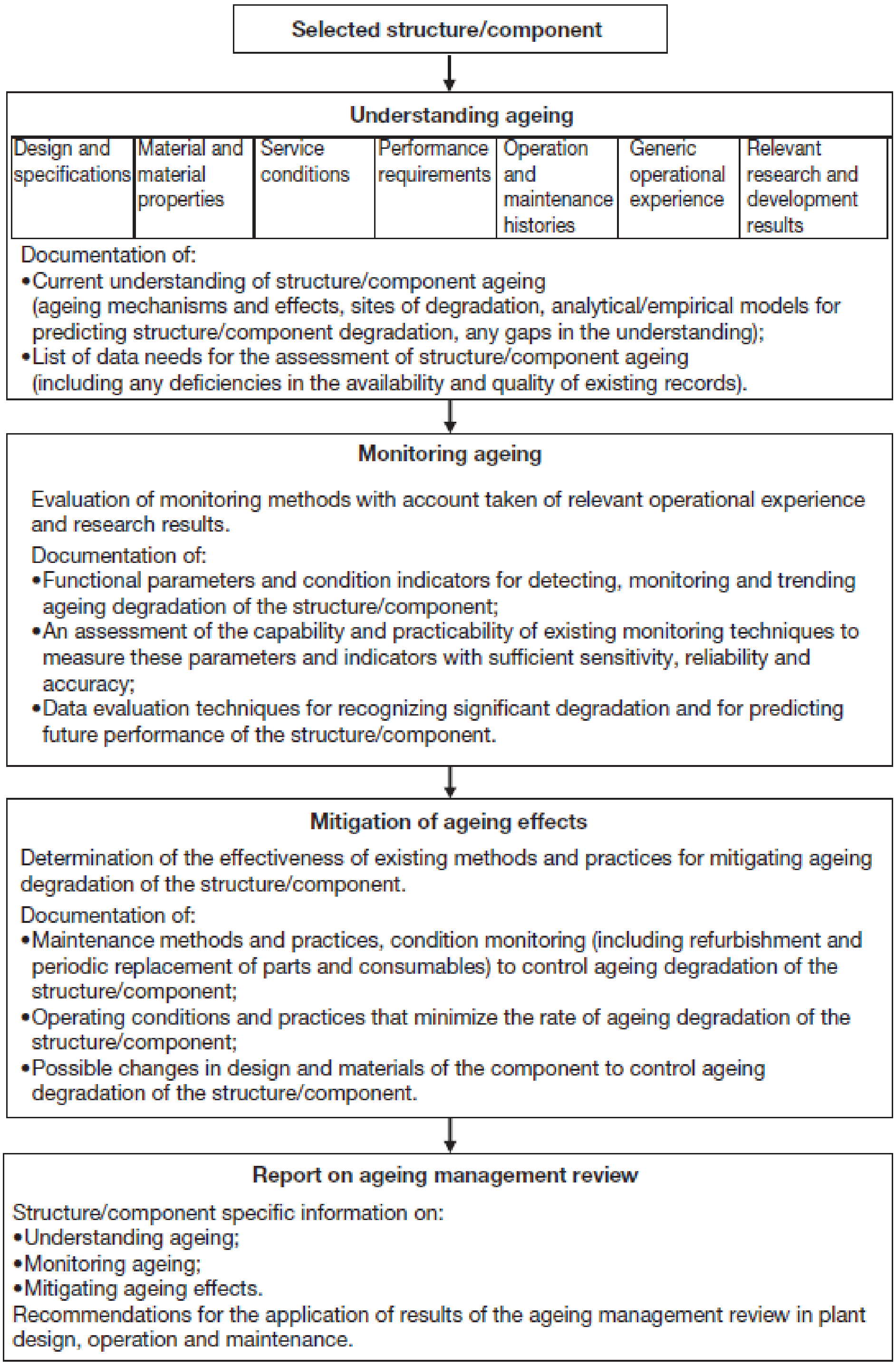

FIG. 4. Illustration of the review of the management of ageing.

7.1.6 Table 2-7 in [6]

Table 2-7 Durability tests used during the design phase

Objective	Potential Testing Specifications	Information Obtained
Determine design chloride loading from concrete cores taken from existing/nearby structures	Nordtest Method NT Build 443 / Accelerated Chloride Penetration (Bulk Diffusion Test)[2-4]	C_s or $C_{S,\Delta x}$ and Δx Chloride Surface Concentration, used in chloride deterioration model
	CEN/TS 12390-11 / Testing hardened concrete – Part 11: Determination of the chloride resistance of concrete unidirectional diffusion[2-5]	
	ASTM C1556 / Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion (used with ASTM C1543)[2-6]	
	Nordtest Method NT Build 492 / Chloride Migration Coefficient from Non-Steady-State Migration Experiments (Rapid Chloride Migration, RCM)[2-7]	
Determine design chloride durability resistance properties from trial batch mix designs	Nordtest NT Build 208 / Chloride Content by Volhard Titration[2-8]	$D_{RCM,0}$, Chloride Migration Coefficient used in chloride deterioration model
Determine initial chloride content of concrete from trial batch mix designs	ASTM C1152 / Acid Soluble Chloride in Mortar and Concrete[2-9]	C_0, Initial Chloride Content used in chloride deterioration model
	ASTM D512 / Chloride Ion in Water[2-10]	
Determine design chloride loading in brackish water	ASTM D516 / Sulfate Ion in Water[2-11]	C_{SW}, Natural Chloride Content of Sea Water used in chloride deterioration model
Determine presence of chemicals (sulfates) in soil and water	ASTM C1580 / Water-Soluble Sulfate in Soil[2-12]	Sulfate content (% mass SO_4 in sample)
	RILEM CPC-18 / Measurement of Hardened Concrete Carbonation Depth[2-13]	
Determine carbonation durability resistance properties from trial batch mix designs	Accelerated Carbonation Test (ACC) / DARTS: Durable and Reliable Tunnel Structures: Data European Commission, Growths 2000, Contract G1RD-CT-2000-00476, Project GrD1-25633, 2004[2-14]	$R_{ACC,0}^{-1}$, Inverse Effective Carbonation Resistance used in carbonation deterioration model
	CEN/TS 12390-10 / Testing hardened concrete – Part 10: Determination of the carbonation resistance of concrete at atmospheric levels of carbon dioxide[2-15]	
	Nordtest Method NT Build 357 / Concrete, Repairing Materials and Protective Coating: Carbonation Resistance[2-16]	

7.1.7 Table 2-8 in [6]

Table 2-8 Durability tests used during construction

Objective	Potential Testing Specifications	Information Obtained
Verify chloride durability resistance properties during production*	Nordtest Method NT Build 492 (Rapid Chloride Migration, RCM)[2-7]	As-built $D_{RCM,0}$, Chloride Migration Coefficient used in chloride deterioration model
Determine initial chloride content of concrete during production	Nordtest NT Build 208[2-8]	As-built C_O, Initial Chloride Content used in chloride deterioration model
	ASTM C1152[2-9]	
Verify concrete carbonation durability resistance properties during production	RILEM CPC-18[2-13]	As-built $R_{NAC,0}^{-1}/R_{ACC,0}^{-1}$, Inverse Effective Carbonation Resistance used in the carbonation deterioration model
	Accelerated Carbonation Test (ACC) / DARTS[2-14]	
	CEN/TS 12390-10 Carbonation Resistance[2-15]	
Verify clear concrete cover in completed structure	BS1881:204 Testing concrete. Recommendations on the use of electromagnetic covermeters[2-17]	Calibration of covermeters
	ACI 228.2R-2.51 / ACI Concrete Practices Non-Destructive testing: Covermeters[2-18]	
	The German Concrete and Construction Association, Technical Report, Concrete Cover and Reinforcement per Eurocode 2. Deutscher Beton- und Bautechnik-Verein, DBV-Merkblatt, Betondeckung und Bewehrung nach Eurocode 2 (in German)[2-19]	Statistical evaluation of measured cover dimensions in hardened concrete
Determine potential for reactivity of aggregates in concrete	RILEM TC 191-ARP / Alkali-reactivity and prevention – Assessment, specification and diagnosis of alkali-reactivity.[2-20]	Expansion % of aggregates for classification of reactivity / Class 1 very unlikely / Class 2 uncertain / Class 3 very likely
	AASHTO R80 / Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction[2-21]	Determine risk of ASR from reactivity of aggregates and exposure, and determination of supplemental cementitious materials (SCMs) to provide various levels of prevention
	ASTM C1260 / Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)[2-22]	Average length change in concrete specimens

Determine freeze-thaw and salt scaling susceptibility	CEN/TS 12390-9 / Testing hardened concrete – Part 9: Freeze-thaw resistance – Scaling (pre-standard), 2006[2-23]	Scaling % by weight or mass per volume
	RILEM TC 176-IDC / Recommendation: CIF Test, Test method of frost resistance of concrete (Capillary suction, internal damage and freeze-thaw test) reference method and alternative methods A and B, 2004.[2-24]	
	ASTM C457 / Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete[2-25]	Air/void spacing and percentage
	ASTM C231 / Air Content of Freshly Mixed Concrete by the Pressure Method[2-26]	Air content of concrete sample and/or air content of mortar portion
	ASTM C173 / Air Content of Freshly Mixed Concrete by the Volumetric Method[2-27]	
Verify chloride durability resistance using alternative air-permeability testing	SIA 262/1 / Air-Permeability Site test[2-28]	As-built air-permeability of concrete cover zone
	Romer, M., Final Recommendation of RILEM TC 189-NEC[2-29]	

* The Rapid Chloride Permeability Test (RCPT), ASTM C1202[2-30], is a very popular worldwide standard to assess concrete's ability to resist chloride ion penetration by measuring its electrical conductance (coulombs). The RCPT test is typically faster and less expensive than the RCM test. However, there is not a universally accepted correlation of the RCPT test results to the Chloride Migration Coefficient used in the *Model Code for Service Life Design* (*fib* Bulletin 34)[2-1] deterioration model for chloride ingress. The RCPT test can be performed in conjunction with the RCM test to develop a correlation between electrical conductance (coulombs) and the Chloride Migration Coefficient for specific concrete mix designs. This correlation could allow the RCPT test to be used for verification purposes during construction.

7.1.8 Figure 156 in [2]

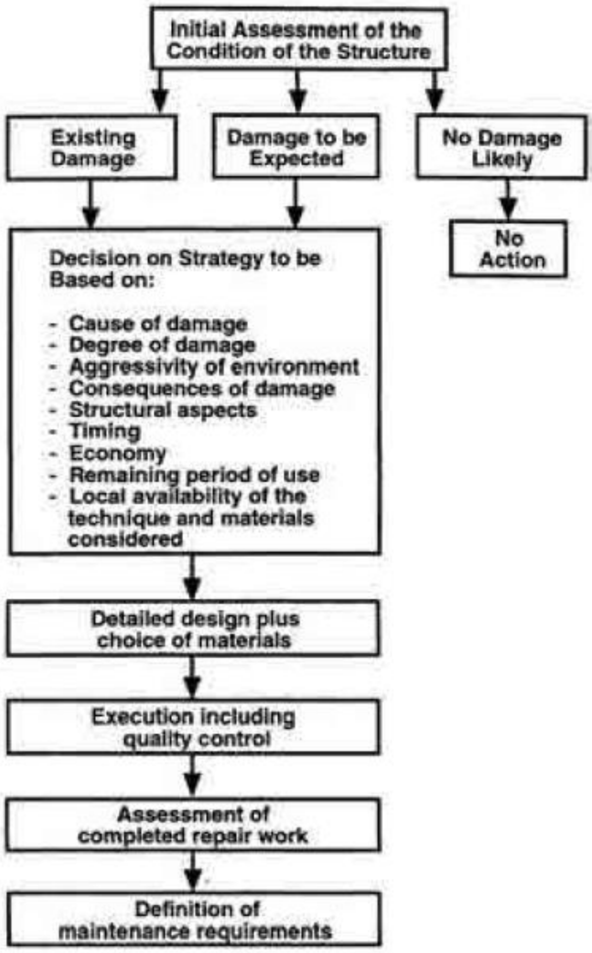

FIG. 156. Steps to be taken in a repair process [518].

[518] TECHNICAL COMMITTEE 124-SRC, Draft Recommendation for Repair Strategies for Concrete Structures Damaged by Reinforcement Corrosion, Materials and Structures 27 171, Int. Union Testing Res. Lab. Mater. Struct. (RILEM), Cachan (1994) 415–436.