IAEA Nuclear Energy Series

No. NP-T-3.5

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Ageing Management of Concrete Structures in Nuclear Power Plants

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AGEING MANAGEMENT OF CONCRETE STRUCTURES IN NUCLEAR POWER PLANTS

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IAEA NUCLEAR ENERGY SERIES No. NP-T-3.5

AGEING MANAGEMENT OF CONCRETE STRUCTURES IN NUCLEAR POWER PLANTS

INTERNATIONAL ATOMIC ENERGY AGENCY
VIENNA, 2016

Printed by the IAEA in Austria
January 2016
STI/PUB/1654

IAEA Library Cataloguing in Publication Data

Names: International Atomic Energy Agency.
Title: Ageing management of concrete structures in nuclear power plants / International Atomic Energy Agency.
Description: Vienna : International Atomic Energy Agency, 2016. | Series: IAEA nuclear energy series, ISSN 1995–7807 ; no. NP-T-3.5 | Includes bibliographical references.
Identifiers: IAEAL 16-01021 | ISBN 978–92–0–102914–0 (paperback : alk. paper)
Subjects: LCSH: Nuclear power plants — Reliability. | Nuclear power plants — Maintainability. | Nuclear power plants — Safety measures. | Concrete — Service life.
Classification: UDC 621.039.58:666.972 | STI/PUB/1654

FOREWORD

One of the IAEA's statutory objectives is to "seek to accelerate and enlarge the contribution of atomic energy to peace, health and prosperity throughout the world." One way this objective is achieved is through the publication of a range of technical series. Two of these are the IAEA Nuclear Energy Series and the IAEA Safety Standards Series.

According to Article III.A.6 of the IAEA Statute, the safety standards establish "standards of safety for protection of health and minimization of danger to life and property". The safety standards include the Safety Fundamentals, Safety Requirements and Safety Guides. These standards are written primarily in a regulatory style and are binding on the IAEA for its own programmes. The principal users are the regulatory bodies in Member States and other national authorities.

The IAEA Nuclear Energy Series comprises reports designed to encourage and assist R&D on, and application of, nuclear energy for peaceful uses. This includes practical examples to be used by owners and operators of utilities in Member States, implementing organizations, academia, and government officials, among others. This information is presented in guides, reports on technology status and advances, and best practices for peaceful uses of nuclear energy based on inputs from international experts. The IAEA Nuclear Energy Series complements the IAEA Safety Standards Series.

There are currently over 400 operational nuclear power plants (NPPs) in IAEA Member States, and over 60 under construction. Operating experience has shown that ineffective control of ageing degradation of major NPP components can jeopardize plant safety and reduce plant life. Ageing in NPPs must be effectively managed to ensure availability of design functions throughout plant service life. From the safety perspective, this means controlling, within acceptable limits, ageing degradation and wear-out of plant components so that adequate safety margins remain.

This publication is one in a series of reports on the assessment and management of ageing of major NPP components. This report is an update of IAEA-TECDOC-1025, Assessment and Management of Ageing of Major Nuclear Power Plant Components Important to Safety: Concrete Containment Buildings, published in 1998. It describes current practices for assessment of safety margins (fitness for service) and inspection, monitoring and mitigation of ageing degradation of selected concrete structures related to a variety of NPP designs. The implications of the differences between new reactor designs are discussed to provide information to better ensure the safe operation of NPPs and to provide a common technical basis for dialogue between plant operators and regulators when dealing with age related licensing issues.

The IAEA officer responsible for this publication was J.H. Moore of the Division of Nuclear Power.

CONTENTS

1. INTRODUCTION

1.1. BACKGROUND

1.1.1. Safety aspects of concrete structures

Containment structures are designed to withstand loadings that may result from postulated events and act as the final physical barrier against radioactive material release to the outside environment.

Both external and internal events are considered in nuclear power plant (NPP) designs. External events include earthquake and severe weather conditions (e.g. floods and tornadoes), as well as potential missile impingement (e.g. aircraft and turbine blades). Critical internal events include loss of coolant accidents (LOCAs) and high energy line breaks.

Potential age related degradation of a concrete containment building (CCB), including its various subcomponents, must be effectively controlled to ensure required leaktightness and structural integrity. Ageing may affect containment structure response under design basis accidents and beyond design basis conditions. Structural margins may be reduced as a result of ageing and environmental effects.

Other concrete structures (e.g. the reactor building (RB), spent fuel pools (SFPs), cooling towers, water intake structures, etc.) can be important to NPP safety, operation or economics as they provide support and shielding functions, or they support key operational functions. SFPs are exposed to the adverse effects of water (e.g. borated or demineralized water), elevated temperatures and radiation. Cooling towers, water intake structures and underground structures may be susceptible to ageing due to their specific service conditions. Ageing may affect these structures' ability to meet functional and performance requirements.

1.1.2. Need for ageing management of concrete structures

The service life for most NPPs was chosen to be 30–40 years. However, the economic benefit for utilities to extend plant service life (with 60 years or more total being a quoted target), has resulted in delayed construction schedules and/or decommissioning strategies that involved the use of containment as a 'safe store' for periods of up to 100 years. This means that containment buildings and other concrete structures often have to perform for a time period significantly greater than their initial service life. Because of the economic benefit for extended plant service life, newer plants often have design lives of 60 years or more.

Concrete is a durable material and its performance as part of the containment function in NPPs has been good. However, experience shows that ageing degradation of concrete structures, often caused or accelerated by factors such as faulty design, use of unsuitable or poor quality materials, improper construction, exposure to aggressive environments, excessive structural loads and accident conditions, can impair the safety functions of concrete structures and can increase risks to public health and safety. Effective ageing management of concrete CCBs and other concrete structures is required to ensure their fitness for service throughout plant service life and during decommissioning.

Several existing NPP programmes, such as periodic inspection, testing, surveillance and preventive and corrective maintenance, contribute to proper ageing management of concrete structures. The effectiveness and efficiency of ageing management can be improved by integrating and modifying these existing programmes, as appropriate, within a systematic ageing management programme (AMP).

Figure 1 from Ageing Management for Nuclear Power Plants, IAEA Safety Guide (NS-G-2.12) [1] provides a systematic and integrated approach to ageing management. The development of a systematic AMP and the interaction of key programme elements are discussed later in this publication.

1.1.3. IAEA programme on safety aspects of NPP ageing

Since 1989, the IAEA has been assisting Member States to understand and effectively manage the ageing of systems, structures and components (SSCs). Details of these activities are described in Appendix I.

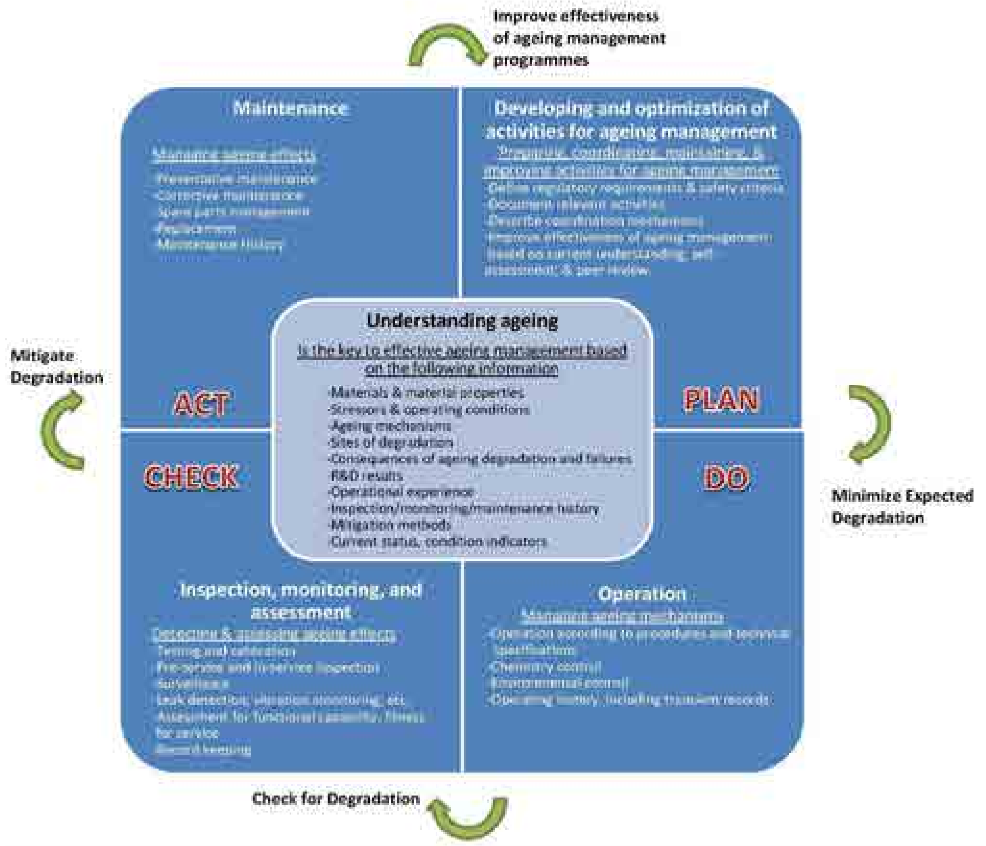


FIG. 1. Systematic approach to managing ageing of a structure or component (adapted from Ref. [1]).

1.2. OBJECTIVE

This publication provides information regarding good practices for assessment and management of ageing related to concrete structures within an NPP. It supersedes IAEA-TECDOC-1025, Assessment and Management of Ageing of Major Nuclear Power Plant Components Important to Safety: Concrete Containment Buildings [2], which was published in 1998. Significant ageing related events have occurred since then that could impact future plant operation, such as alkali–silica reactivity in concrete, delamination events, larger than anticipated loss of prestressing force, SFP leakage, corrosion of steel in water intake structures, etc. These events led to new ageing management actions by both NPP operators and regulators.

Specifically, this publication includes:

(a) State of the art information regarding ageing management of concrete structures in NPPs throughout their entire service life;
(b) Background material indicating the importance of AMPs;
(c) Current practices and techniques for assessing fitness for service and for inspection, monitoring and mitigation of ageing related degradation of CCBs and other concrete structures important to the safe and reliable operation of NPPs;
(d) A technical basis for developing and implementing a systematic AMP;
(e) Guidelines to help ensure ageing management is taken into account during different NPP lifecycle phases, that is, design, fabrication, construction, commissioning, operation (including long term operation and extended shutdown) and decommissioning;
(f) Research material related to ageing and lessons learned.

This report is intended for NPP owners/operators, designers, engineers and specialists to:

(a) Establish, implement and improve AMPs for NPPs;
(b) Facilitate dialogue between owners/operators and regulators when dealing with age related licensing issues;
(c) Consider ageing in new plant design, in modifications, and in approaches to mitigate ageing effects.

1.3. SCOPE

This publication deals with CCBs and other concrete structures (SFPs, cooling towers, water intake structures and underground structures, among others) that are included as part of an NPP.

Topics discussed in this publication include potential ageing mechanisms, age related degradation and ageing management (i.e. inspection, monitoring, assessment and remedial measures) as well as condition assessment. The term condition assessment, as most commonly used, refers to both condition assessment and life assessment. The following materials and components of concrete structures are discussed in this publication:

(a) Concrete;
(b) Mild steel reinforcing;
(c) Prestressing systems;
(d) Penetrations;
(e) Liners (metallic and non-metallic);
(f) Anchorages;
(g) Foundations and piles;
(h) Waterstops, seals and gaskets;
(i) Protective coatings.

1.4. STRUCTURE

This publication follows the structure of the generic AMP defined in NS-G-2.12 [1] as illustrated in Fig. 1. Section 2 introduces the generic AMP as it relates to concrete structures. Section 3 describes where concrete is used in NPPs. The process of an effective AMP is described in Sections 4–8, and includes the following:

(a) Understanding ageing (Section 4);
(b) Developing and optimizing the AMP (Section 5);
(c) Plant operation (Section 6);
(d) Inspection, monitoring and assessment of structures (Section 7);
(e) Maintenance and repair of ageing effects (Section 8).

Each section includes steps in the process for the development of an AMP and information specific to concrete structures, which supplements the generic information provided in NS-G-2.12 [1].

Section 9 summarizes the conclusions of this publication. The appendices cover IAEA activities on ageing management (Appendix I), describe containment structures (Appendix II) and ageing management practices and operating experience in selected Member States (Appendices III and IV), describe ageing management practices in non-nuclear industries (Appendix V) and provide details of the coordinated research project (CRP) conducted in the 1990s (Appendix VI) and case studies and example applications of modelling of concrete structures (Appendix VII).

1.5. TERMINOLOGY

Common terminology for ageing management in the context of NPPs is derived from industry sources such as those found in Refs [3, 4]. Ageing management is defined in the IAEA Safety Glossary [5]. Annex I of NS-G-2.12 [1] points to similar publications.

The IAEA Safety Glossary [5] defines ageing management as:

“Engineering, *operations* and *maintenance* actions to control within *acceptable limits* the *ageing degradation* of *structures, systems and components*.

ⓘ Examples of engineering actions include *design, qualification* and *failure analysis*. Examples of *operations* actions include surveillance, carrying out operating *procedures* within specified *limits* and performing environmental measurements.

ⓘ ***Life management*** (or ***lifetime management***) is the integration of *ageing management* with economic planning: (1) to optimize the *operation, maintenance* and *service life* of *structures, systems and components*; (2) to maintain an acceptable level of performance and *safety*; and (3) to maximize the return on investment over the *service life* of the *facility*.”

Figure 2 shows the relationships among the terms used to describe the lifetime of structures, systems and components. This publication focuses on the service life of SSCs.

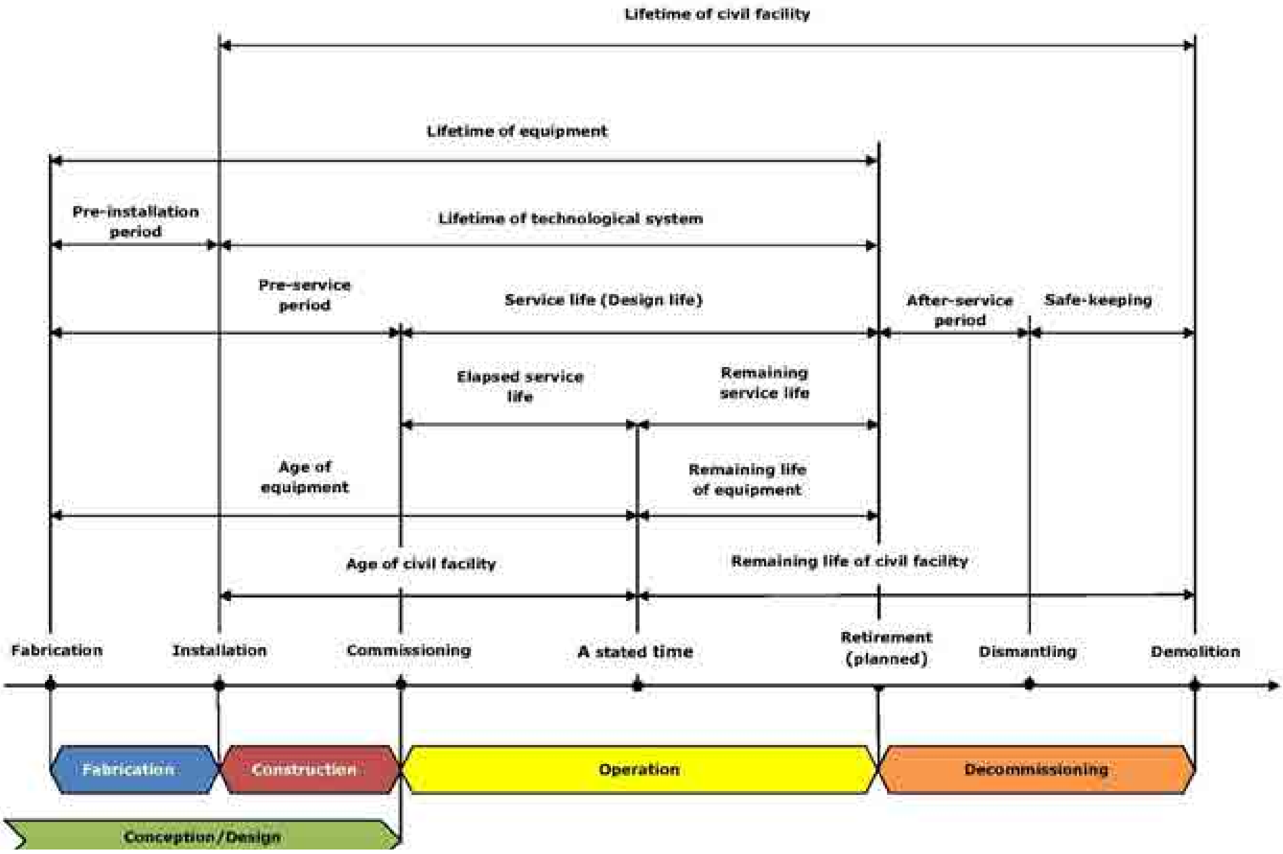


FIG. 2. Visual representation of the terms associated with the lifetime of structures, systems and components [4].

2. AGEING MANAGEMENT BASICS

This section describes the five components of an AMP for concrete structures. Subsequent sections address each component in more detail to facilitate preparation of plant specific AMPs.

IAEA Safety Requirement publication SSR-2/2, Safety of Nuclear Power Plants: Commissioning and Operation [6] contains a requirement that NPP operating organizations ensure an effective AMP is implemented so that “required safety functions of systems, structures and components are fulfilled over the entire operating lifetime of the plant.” IAEA Safety Guide NS-G-2.12 [1] provides recommendations for effective ageing management of such SSCs. IAEA Safety Standards, The Management System for Facilities and Activities (GS-R-3) [7] and The Managament System for Nuclear Installations (GS-G-3.5) [8], provide information on setting up an appropriate management system, including a corrective action programme.

The primary objective of an AMP for concrete structures is to ensure timely detection and mitigation of degradation that could have an impact on safety functions. When applied to non-safety related structures, AMPs can provide economic benefit by detecting structural degradation that could lead to dangerous situations or plant unavailability.

An AMP takes a systematic, comprehensive and integrated approach aimed at ensuring the most effective and efficient management of ageing. A comprehensive understanding of concrete structures, their ageing degradation mechanisms and degradation impacts on a structure’s ability to perform design functions, are the fundamental elements of an AMP. This understanding is derived from knowledge of:

- Plant design basis;
- Applicable codes and regulatory requirements;
- Plant design;
- Plant fabrication and construction methods;
- Material properties;
- Service conditions;
- Operation and maintenance history;
- Commissioning;
- Surveillance methods;
- Inspection results;
- Generic operating experience;
- Research results.

Table 1 lists the generic attributes of an AMP (upon which programme effectiveness can be evaluated). Table 1 also lists the sections of this publication applicable to each generic attribute of an AMP.

A systematic AMP consists of a feedback loop that includes the following:

(a) Understanding of the structure/component that is ageing;
(b) Development and optimization of activities for ageing management;
(c) Operation of a plant within design limits to minimize age related degradation (in particular, error induced accelerated degradation);
(d) Inspection, monitoring and assessment to detect and characterize significant component degradation before fitness for service is compromised;
(e) Maintenance to manage ageing effects.

Such an AMP needs to be implemented in accordance with guidance prepared by an interdisciplinary concrete structure ageing management team that is organized at the corporate or owner’s group level. Criteria to assess the results of such a concrete structure AMP are as follows:

- The physical condition of structures is satisfactory in terms of the required safety margins (i.e. integrity and functional capability are retained).

- Structure condition and/or functional indicators (provided by surveillance, in-service inspection testing or condition monitoring) and their trends conform to acceptance criteria (relevant condition indicators are satisfactory);
- Ambient environment and system parameters (e.g. humidity, temperature and pressure), and their trends are within specified limits.

— Structure contribution to plant or system unavailability has been satisfactory.

TABLE 1. GENERIC ATTRIBUTES OF AN EFFECTIVE AMP AND APPLICABLE SECTIONS OF THIS PUBLICATION (*adapted from Ref.* [1])

Attribute	Description	Applicable sections of this publication
1. Scope of the AMP based on understanding ageing	Structures (including structural elements) and components subject to ageing management Understanding of ageing phenomena (significant ageing mechanisms, susceptible sites): – Structure/component materials, service conditions, stressors, degradation sites, ageing mechanisms and effects – Structure/component condition indicators and acceptance criteria – Quantitative or qualitative predictive models of relevant ageing phenomena	Sections 3, 4, Table 8, Section 7.4.1
2. Preventive actions to minimize and control ageing degradation	Identification of preventive actions Identification of parameters to be monitored or inspected Service conditions (i.e. environmental conditions and operating conditions) to be maintained and operating practices aimed at slowing down potential degradation of the structure or component	Sections 6, 7, 8.2
3. Detection of ageing effects	Effective technology (inspection, testing and monitoring methods) for detecting ageing effects before failure of the structure or component	Sections 7.2, 7.3
4. Monitoring and trending of ageing effects	Condition indicators and parameters monitored Data to be collected to facilitate assessment of structure or component ageing Assessment methods (including data analysis and trending)	Table 8, Sections 7.3, 7.4, 7.5
5. Mitigating ageing effects	Operations, maintenance, repair and replacement actions to mitigate detected ageing effects and/or degradation of the structure or component	Table 8, Section 8
6. Acceptance criteria	Acceptance criteria against which the need for corrective action is evaluated	Section 7.4
7. Corrective actions	Corrective actions if a component fails to meet the acceptance criteria	Sections 5.4, 8.3, 8.5
8. Operating experience feedback and feedback of research and development results	Mechanism that ensures timely feedback of operating experience and research and development results (if applicable), and provides objective evidence that they are taken into account in the AMP	Sections 2, 4.9
9. Quality management	Administrative controls that document the implementation of the AMP and actions taken Indicators to facilitate evaluation and improvement of the AMP Confirmation (verification) process for ensuring that preventive actions are adequate and appropriate and that all corrective actions have been completed and are effective Record keeping practices to be followed	Sections 2, 5, 6.4, 7.5, 8.6

Existing AMPs for concrete structures have generally focused on managing degradation effects. AMPs are typically based on periodic inspection or structure monitoring, with remedial measures being implemented to deal with any observed degradation before serviceability is lost.

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.

Feedback from operating experience, results from research and development and results from self-assessments and peer reviews need to be used to improve the AMP via the NPP's corrective action process.

3. CONCRETE STRUCTURE DESCRIPTION

3.1. INTRODUCTION

Concrete makes up a large portion of a typical NPP and is used extensively in structures and as biological shielding. This section describes where concrete is typically used and provides some additional detail regarding key structures such as those related to containment.

Table 2 below provides a list of where concrete is typically used, with details related to its accessibility for inspection. Concrete structures may be safety related or non-safety related, and may be of varying importance with respect to plant safety and/or production. Concrete containment is covered in Section 3.2, major balance of plant structures are covered in Section 3.3, and related components are discussed in Section 3.4.

TABLE 2. TYPICAL NPP SAFETY RELATED CONCRETE STRUCTURES AND THEIR ACCESSIBILITY FOR VISUAL EXAMINATION (*adapted from Ref.* [9])

Structure	General accessibility
Primary containment	
Containment dome/roof	Internal surfaces (liner)/complete external
Containment foundation/basemat[a]	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 structure	Generally accessible
Reactor shield wall (biological)	Typically lined
Ice condenser dividing wall (PWR[b]: ice condenser)	Lined or hard to access
Nuclear steam supply system equipment supports/vault structures	Partially accessible (design dependent)
Weir and vent walls (BWR[c] Mark III)	Lined with limited access
Pool structures (BWR Mark III)	Lined
Diaphragm floor (BWR Mark II)	Lined with limited access
Drywell/wetwell slabs/walls (BWR Mark III)	Internal liner/partial external access
Secondary containment/reactor buildings/pressure relief ducts/vacuum buildings	
Slabs, columns and walls	Accessible on multiple surfaces
Foundation	Top surface
Sacrificial shield wall (metal containments)	Internal lined/external accessible

TABLE 2. TYPICAL NPP SAFETY RELATED CONCRETE STRUCTURES AND THEIR ACCESSIBILITY FOR VISUAL EXAMINATION (*adapted from Ref.* [9]) (cont.)

Structure	General accessibility
Pressure relief ducts	Generally accessible
Vacuum buildings	During vacuum building outage only
Fuel/equipment storage pools	
Walls, slabs and canals	Internal lined/partial external
Other building structures (walls, slabs, etc.)	
Auxiliary building	Generally accessible
Control room or building	Generally accessible
Diesel/standby generator building	Generally accessible
New fuel storage building	Generally accessible
Spent dry fuel storage building	Generally accessible
Heavy water upgraders, tritium removal facilities	Generally accessible
Plant-specific structures	
Piping or electrical cable ducts or tunnels	Limited accessibility
Radioactive waste storage building	Generally accessible
Radioactive spent resin storage/liquid waste vaults	Partially accessible
Stacks	Partial internal/external above grade
Intake structures (intake structure, piping and canal)	Internal accessible/external above grade and waterline
Pumping stations	Partially accessible
Cooling towers	Accessible above grade exterior/partially accessible interior
Plant discharge structure	Internal accessible/external above grade and waterline
Emergency cooling water structure	Limited accessibility
Dams	External surfaces above waterline
Tsunami/flood protection barriers	External surfaces above waterline
Water wells, water storage vaults	Limited accessibility
Turbine buildings	Generally accessible

[a] The term ‘basemat’ has been used throughout this publication, replacing similar terms such as base slab, mat foundation or foundation mat.
[b] PWR — pressurized water reactor
[c] BWR — boiling water reactor

3.2. CONTAINMENT STRUCTURES

3.2.1. Containment designs

Although some early thermal reactors (e.g. gas cooled and graphite moderated light water reactors) were designed and built without containment, all current generation reactors include containment, by design. The primary purpose of containment is to prevent the release of fission products to the environment in the case of a design basis accident, such as a LOCA, and to provide radiation shielding under normal operation (i.e. to separate the reactor and other safety significant systems and equipment from the outside). In addition to being designed to withstand internal pressures and temperatures that may result from design basis accidents, containment structures are designed to provide protection against severe external events (e.g. floods, tornadoes and missile impact). It is essential to ensure that structural capacity and leaktight integrity of containment has not unacceptably deteriorated due to ageing or environmental stress. Such knowledge of the status of containment will feed into reliable continued service evaluations and informed ageing management decisions.

With respect to containment functions, various concepts exist that depend on the reactor system used (e.g. PWR, BWR, pressurized heavy water reactor (PHWR), gas cooled reactor, water-water power reactor and modular reactor). Containment is typically constructed from reinforced concrete, prestressed concrete and steel. Containment liners are typically made up of either steel or an epoxy coating, depending on the design.

Information on the design of concrete containment and its performance parameters (e.g. dimensions, capacity, volume and design pressure) and service conditions is provided in Ref. [10]. Detailed summary descriptions of several of the more commonly used Generation II, III and III+ containment designs, and some related structures, are provided in Appendix II. More detailed information is available in Refs [11] and [12].

3.2.2. Containment liners (metallic or non-metallic)

Metallic or non-metallic liners are provided on inside surfaces of many single wall containment structures to provide a barrier against gas leakage.

Typical metallic liners are composed of carbon steel plate up to 25 mm thick, joined by welding and anchored to the concrete by studs, structural steel shapes or other steel products. PWR containment, and dry-well portions of BWR containment, are typically lined with carbon steel plates. Liners of wet-wells for BWR containment, as well as light water reactor fuel pool structures, are typically stainless steel plates. Although the liner's primary function is to provide a leaktight barrier, it also can act as part of the formwork during concrete placement and may be used to support internal piping/equipment. The liner does not contribute to structure strength.

Liners for SFP structures are discussed in Section 3.3.1 and non-metallic liners for containment and other structures are discussed in Section 3.4.4.

3.2.3. Containment penetrations

Access to CCB interiors is provided through penetrations in the containment shell (e.g. electrical penetrations, airlocks and piping penetrations).

In most cases, electrical penetration assemblies, in addition to serving as a closure, have to retain electrical functions during a severe accident [13] and need to meet all requirements for cable performance during both normal operation and operation under accident conditions. Electrical penetration assemblies (Fig. 3) are used to provide a leaktight feed-through into containment for power, control and instrumentation cables. Neither hot, high pressure steam nor intense gamma irradiation is to cause damage to the operation of electrical penetration assemblies. They also need to withstand the decontamination process (e.g. cleaning with acidic and alkaline solvents). The service life of electrical penetration assemblies needs to be consistent with the service life of the reactor and needs to meet the requirements for hermetically sealed containment walls. To prevent the escape of radioactive by-products into the environment, the rate of gas leakage needs to remain below the levels that are defined in the relevant standards. Electrical penetration assemblies also need to be fire resistant, earthquake proof and have radiation screening as effective as that of the concrete wall.

Personnel airlocks and emergency airlocks are similar in principle. They both include two pressure seating doors in series [13]. Only one door is normally allowed to open at a time to maintain the containment function.

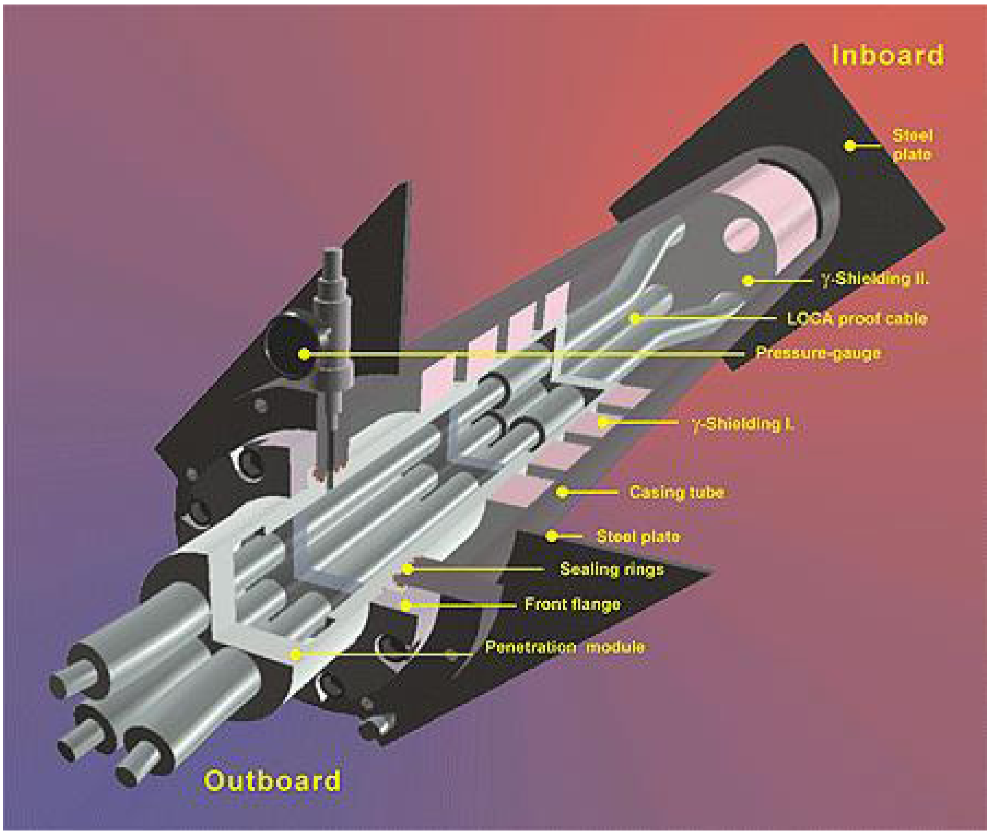


FIG. 3. Example of low voltage electrical penetration assembly for a WWER NPP [14].

These doors are supported from flat (often stiffener reinforced) bulkheads, which are in turn welded to a circular sleeve that is either embedded in concrete or welded to the shell for steel containment. Occasionally, a personnel airlock is integrated, with the equipment hatch and has an airlock sleeve concentric with the equipment hatch spherical cap. Figure 4 presents an example of a personnel airlock for a PWR type plant. Double ethylene propylene diene monomer (EPDM) gaskets provide the seal between each door and the bulkhead. The airlock is designed for an external pressure of 0.41 MPa and a maximum temperature of 170°C. Other sealing arrangements are possible, including double tongue and groove configurations with recessed rectangular elastomeric seals, partially exposed double circular cross-section seals with a flat mating flange, or inflatable seals. Both doors must be opened or their seals bypassed for containment boundary leakage to occur. During accidents, the exterior door is expected to be much cooler than the inner door (since the heat source is behind the inner door) [15].

Pipe penetrations may be either rigidly attached to containment, or bellows may be provided as the pressure boundary with the pipe being allowed to move relative to containment at the penetration. Rigid penetrations typically include a cylindrical sleeve with a concrete shear anchor for concrete containment structures (CCSs), or with the sleeve welded to the (often locally thickened) steel containment vessel. Closure plates may be either a forged (flued) head or a flat plate welded to the sleeve and process pipe spool [15].

Containment piping penetration bellows (Fig. 5) are typically constructed of stainless steel and are primarily used in steel containment structures, with only limited use in reinforced and prestressed structure CCSs [16]. The bellows minimize the load imposed on containment that is caused by differential movements between containment and the pipe to which the bellows are attached.

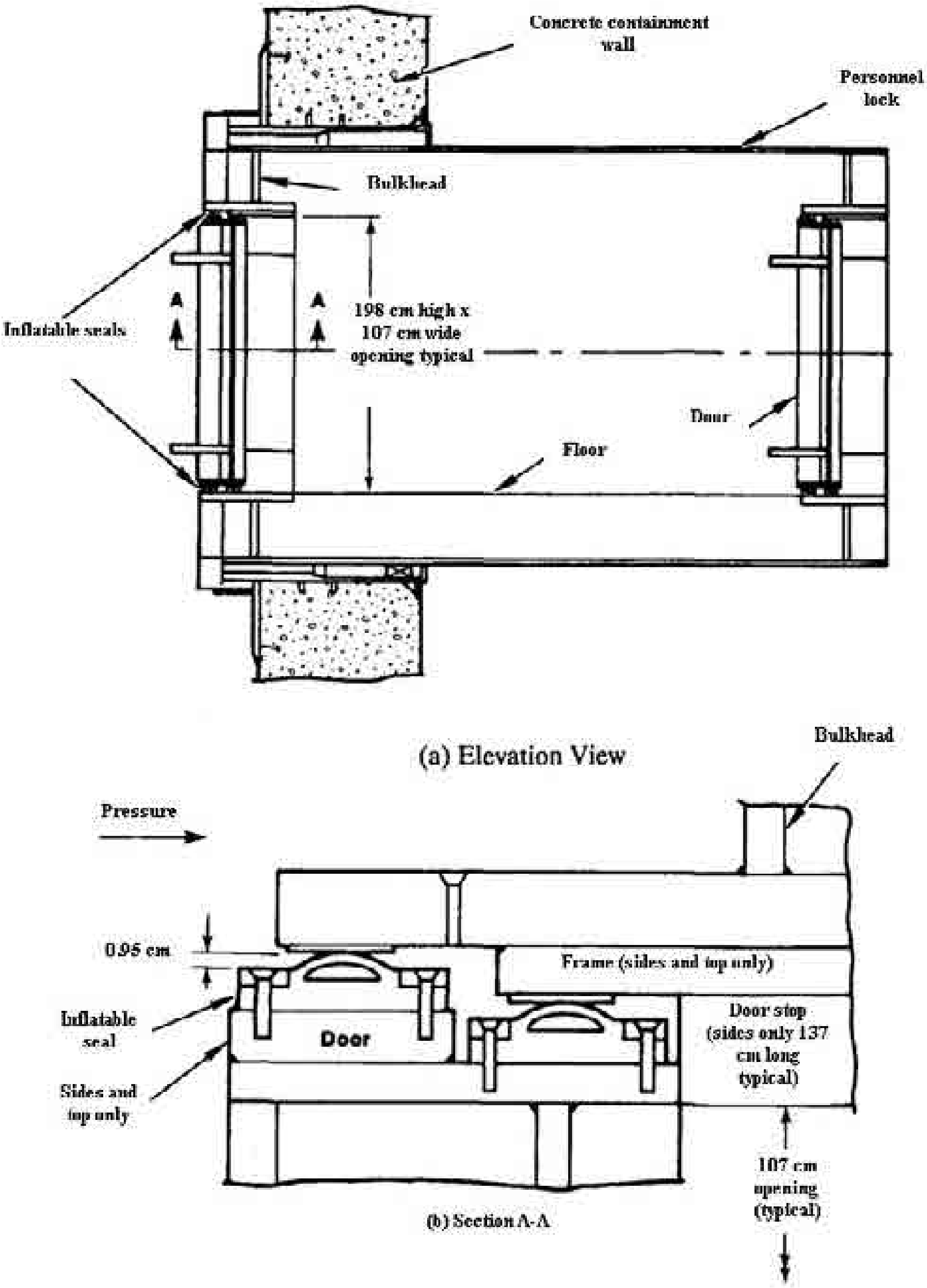


FIG. 4. Example of a personnel airlock and inflatable seal for a PWR: (a) is elevation (cross-section) view; (b) shows detail of door sealing arrangement from (a) (adapted from Ref. [13]).

3.3. BALANCE OF PLANT CONCRETE STRUCTURES

3.3.1. Spent fuel pools

Fuel storage facilities provide for receiving, storing, shielding, shipping and handling of new and spent fuel. They are a part of the RB of most BWR Mark I and Mark II containment structures. For BWR Mark III and many PWRs and PHWRs, fuel storage facilities are separate structures; however, in some plants the facility is part of an auxiliary bay or building (Fig. 6). The fuel storage structure is a multistory reinforced concrete structure supported on bearing walls and/or a basemat.

SFPs are used to store used fuel assemblies, fuel bundles, or to control and/or adjust rods for a period of time until they are sufficiently cool and can be transferred to dry cask or other storage. A SFP typically has

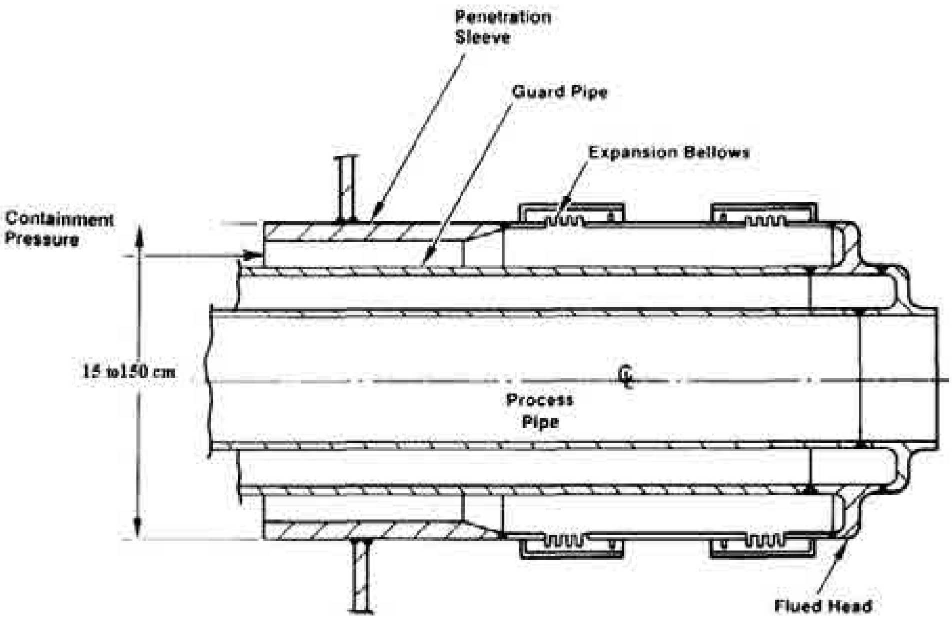


FIG. 5. Example of process piping bellows showing pipe penetration through containment (adapted from Ref. [13]).

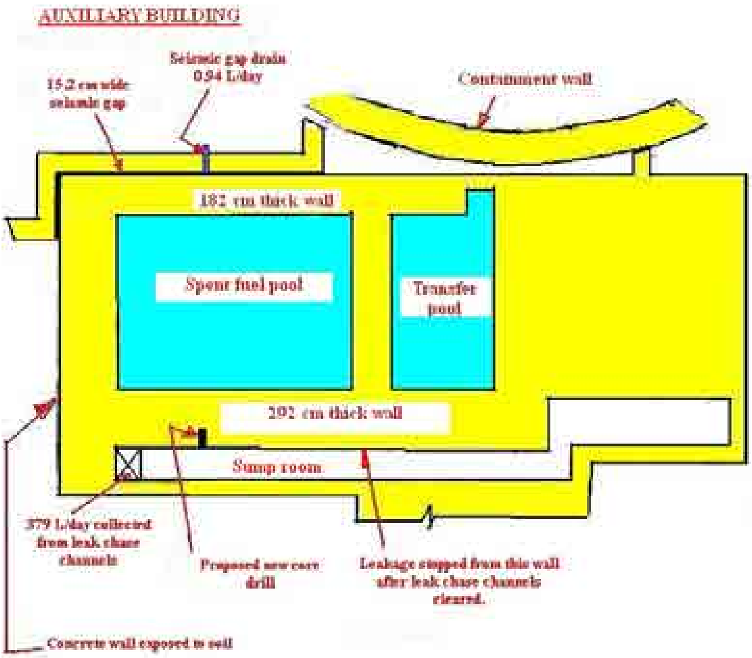


FIG. 6. A fuel handling building showing the relative location of the SFP, transfer pool and containment building (adapted from Ref. [17]).

reinforced concrete walls and floors for radiation shielding and may have a stainless steel or non-metallic liner to enhance leaktightness.

The main design differences between SFPs are at the liner to concrete interface, which can be constructed in two ways. The wallpaper method involves allowing poured concrete to harden before placing the liner. The formwork method involves placing the liner first and using it as the form for concrete.

A typical SFP for a PWR can be about 12 m deep and 12 m or more in each horizontal direction. Walls are constructed using reinforced concrete with a typical thickness of 0.7–3 m. Cover concrete for rebar is typically 3.8–7.6 cm. The inside surfaces of PWR SFPs are typically lined with stainless steel plates that are about 6–13 mm thick and are joined by full penetration seam welds. Between the seams, liner plates may also be plug welded to studs that are embedded in the concrete.

Leakage collection systems, consisting of channels embedded in the concrete at weld seam locations, are provided to permit monitoring and collection of leakage that might occur at the welds. The leakage collection system channels are made of carbon or stainless steel and may be seal welded to the back of the liner. The channels lead to a series of telltales piped to a collection system and routed to the liquid radwaste system.

SFPs may contain either demineralized water or borated water, depending on the reactor type and where the SFP is located with respect to the reactor. Reference [18] provides detailed information on the water type and chemistry for various reactor types.

SFPs for Canadian natural deuterium uranium (CANDU) PHWRs (see Fig. 7) have a variety of pool designs and storage capacities. Single unit stations are usually designed to have a series of interconnected pools with one main pool for spent fuel storage. Multiunit stations usually have primary and secondary storage pools with additional dry storage cask facilities. The water filled, reinforced concrete pools are built in ground or out of ground and are fitted with fibreglass reinforced non-metallic or stainless steel liners.

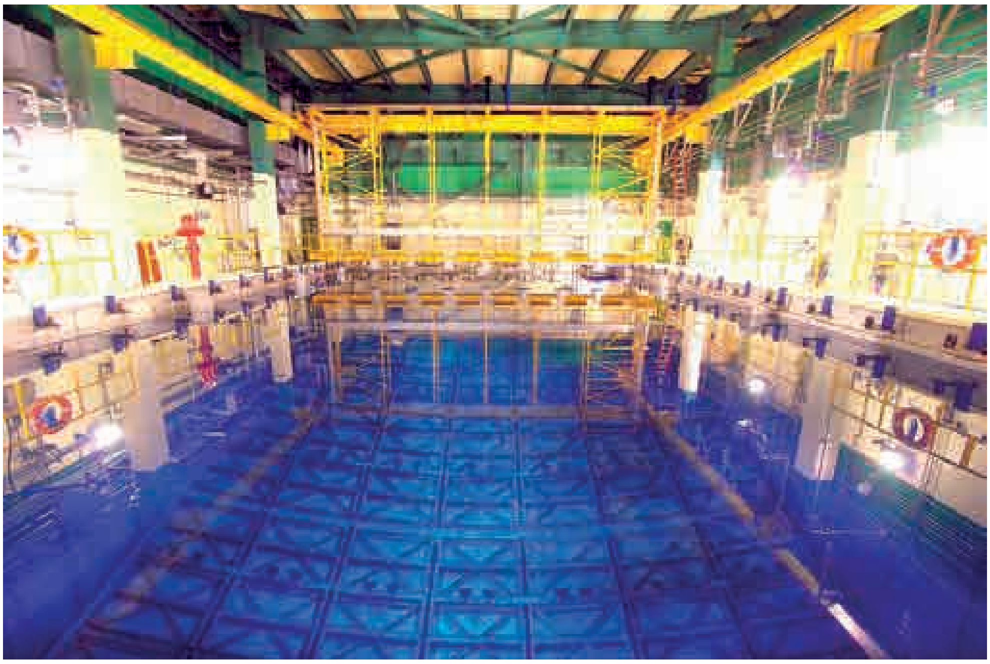


FIG. 7. CANDU 6 SFP.

Russian designed spent fuel bays are typically rectangular reinforced concrete with inner walls and double clad metal insulated dividing walls that are used for detecting leakage. Drain cavities in the walls are divided by insular reservoirs, each with separate drain pipes. A typical arrangement is shown in Fig. 8.

Epoxy/paint liners are used in Magnox pools. Away from reactor storage pools are typically stainless steel lined (e.g. storage pools in Finland, France, Germany, Japan, the Russian Federation and Sweden), with the exception of the UK, where most are only lined at the wind/water line, and the rest is painted [18].

3.3.2. Cooling towers

Cooling towers are used in closed cycle water systems to remove waste heat from the main condenser loop. Cooling towers used as primary heat sinks are considered Class I structures and are designed to withstand natural phenomena (e.g. earthquakes and tornadoes). Concrete cooling towers are divided into two categories, atmospheric or natural draft cooling towers, and mechanical draft cooling towers.

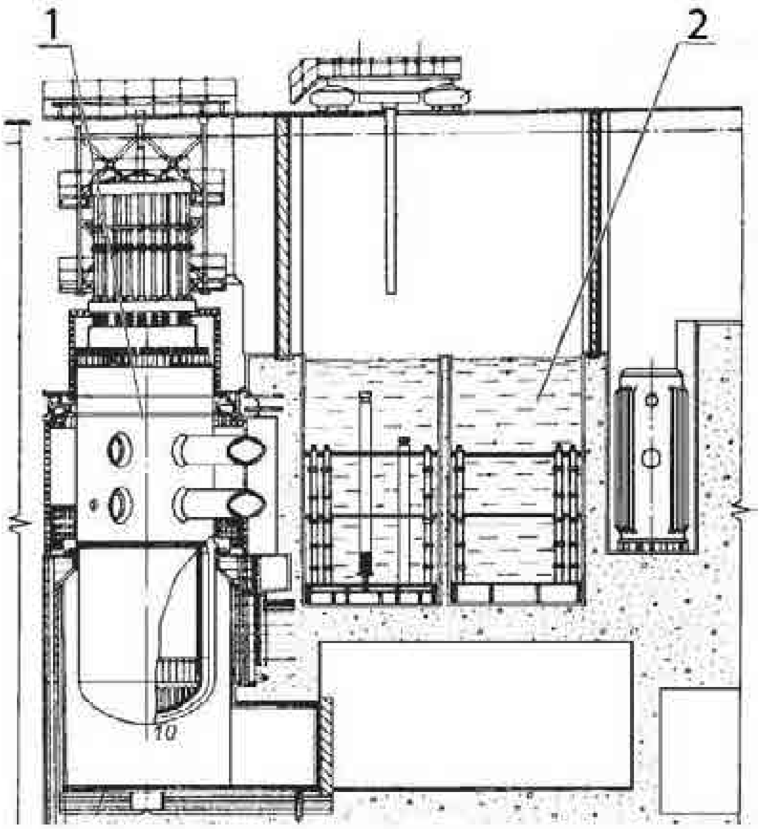


FIG. 8. Typical Russian design SFP (1 – reactor, 2 – SFP) [19].

3.3.2.1. Natural draft cooling towers

Natural draft cooling towers rely on a chimney effect in which air circulation is naturally driven by density differences between heated air (less dense) and cooled air (denser). Therefore, these structures operate better in areas of high relative humidity. These structures tend to be large and are constructed on-site [20]. Figures 9 and 10 present examples of natural draft cooling towers.

FIG. 9. Natural draft cooling towers at Cattenom NPP, France (courtesy of Hamon Thermal Europe SA).

Concrete structures in a natural draft cooling tower include:

— *A hyperbolic shell*. Reinforced concrete structures with typical heights between 120 and 200 m, shell thickness from 25 (neck) to 40 cm, and crown/cornice diameter about 80 m. The crown is generally wider for

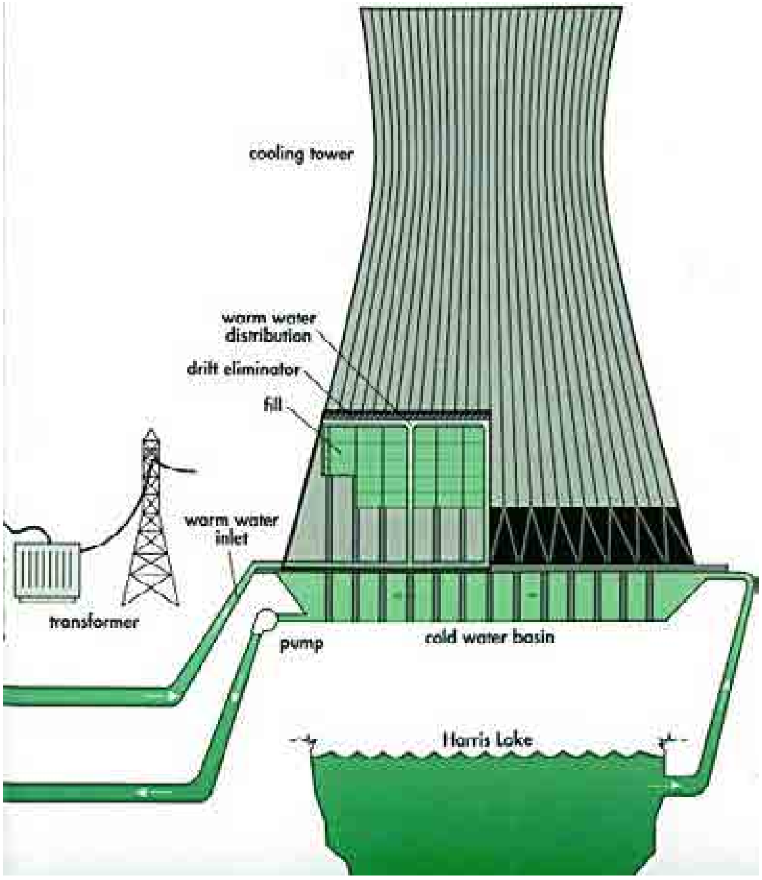


FIG. 10. Example of natural draft cooling tower (courtesy of Progress Energy).

mechanical stiffening and to allow walking access. The shell lintel is supported by X/W/A shaped columns and, usually, a shallow annular foundation.

— *Packing support*. Reinforced beams supporting dispersion fills.
— *Pool.* Reinforced structure used to collect intake water and to dispatch it through a metallic piping network that sprays the water through sprinklers.
— *Intake structures (including piping).* Piping leading to the tower may be made up of a variety of materials: concrete, steel, fibre reinforced piping (FRP), etc.
— *Cold water basin.* After water is cooled, it is returned to a (usually concrete) reservoir called the cold water basin for routing back to the condenser.

3.3.2.2. Mechanical draft cooling towers

Mechanical draft cooling towers are called mechanical because they have fans, usually located at the top of the structure, that move air across the hot water. Towers may be cross-flow or counter-flow types. Cross-flow types allow air to travel perpendicular to the water flow direction. Water is pumped into an open inlet basin and metering orifices (i.e. holes) in the inlet basin allow water to fall by gravity onto the cooling tower fill. These orifices may incorporate shapes at orifice outlets to better distribute water for cooling. Fans are used to induce air flow through the fill. Typically, for power plants, these fans are large axial fans. The air enters horizontally, often directed by louvers over and through the wet fill surface. Drift eliminators prevent excessive water droplets from being carried through the fan and creating a large plume. The air then turns vertically and is exhausted through the fans [21].

Mechanical draft cooling towers are usually prefabricated and can be assembled on-site or at the factory. Different materials are used, with concrete being preferred for water basins. Field-erected towers (typically large units) are increasingly using concrete for columns, beams, supports and decks because of their load carrying capacity and higher resistance to fire. Different configurations can be seen in these towers depending on air flow direction (cross-flow vs. counter-flow), fan location (forced draft vs. induced draft) and tower shape (rectilinear vs. round) [20]. Figure 11 shows a mechanical draft cooling tower cross-section and related information.

Concrete structures in a generic round-shaped mechanical draft cooling tower include the following:

— *Fan deck.* Located at about 15.2 m and supported by a network of beams and slender columns. The deck may support 16 different fans and mechanical equipment.
— *Distribution flume and basin*. Located at the fan deck periphery, they distribute the water before dropping through the fill.
— *Radial and circumferential panels.* Frames support the distribution flume and basin, as well as the fill (splash bars) and concrete fill beams.
— *Cold water basin.* Similar to a natural draft cooling tower.

Concrete elements can be reinforced or prestressed, precast elements. One plant reports the use of epoxy coated rebar. During operation, the concrete inside the mechanical draft cooling tower shell is subjected to temperatures around 40°C and 100% relative humidity.

Natural intake water chemistry varies from one plant to another. Some plants use municipal retreated water containing chlorides or brackish water. Chemicals are often added to the water for condenser cleaning. In the USA, acid/phosphate/oxidizing biocides, such as sodium hypochlorite, are used to dissolve packing deposits or are used as a biocide. Calcium deposits are often observed on packing beams. The deposit threshold to trigger cleaning is about 120 kg/m^3 in the USA and 30 kg/m^3 in France.

3.3.3. Water intake structures

Water intake structures are that portion of the circulating water system that facilitates intake of cooling water to the main condenser [22]. Typical structures where concrete may be installed are the intake structure itself, circulating pump supports, baffle/skimmer walls, concrete ponds and canals.

Two types of circulating water systems are used, open and closed cycle. In open cycle systems, cooling water is taken from a river, lake or ocean. The intake structure for an open cycle circulating system is separated into two parts, the screen well and the pump well, which are combined into one structure at most plants. Water

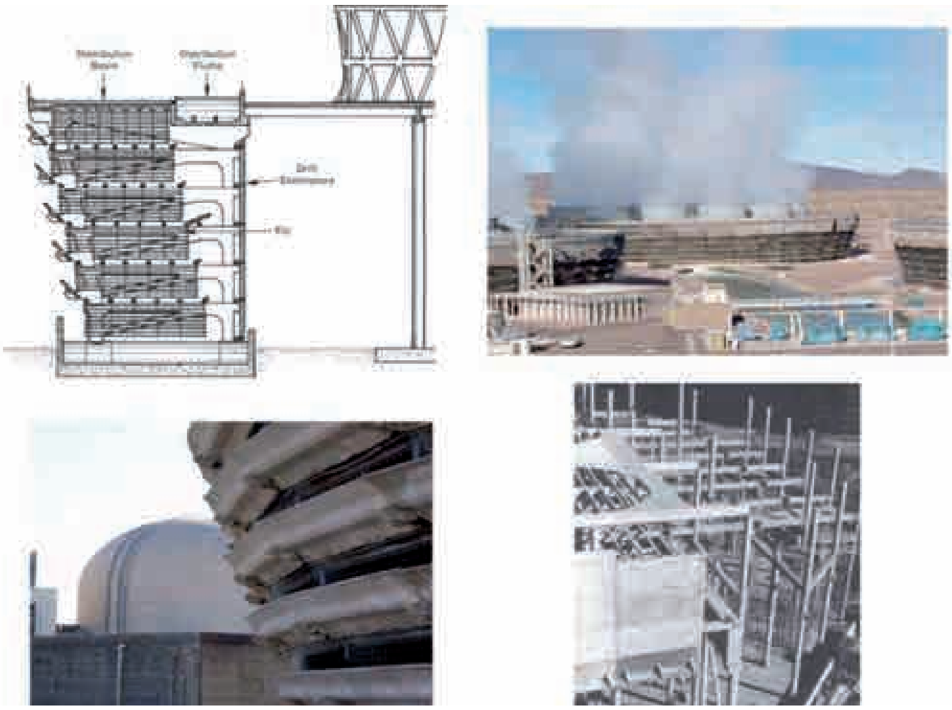


FIG. 11. Example of mechanical draft cooling tower: cross-section (upper left), tower in operation (upper right), louver (bottom left), and cooling tower modular construction (bottom right) (courtesy of APS and SPX Cooling Technologies).

from the source flows first through the screen well in open channel flow where it encounters bar racks and trash screens needed to filter debris before it reaches the pumps. A system for cleaning these screens is normally also located in this area. After passing through the screen well, water is directed to the pump well, the structure of which is dependent on the size, type and number of pumps. Both the screen well and the pump well are of reinforced concrete construction. Most of the intake structure is below grade. For open cycle systems, the intake structure is mostly below the water table level (Fig. 12).

In closed cycle systems, water is cooled by cooling towers or cooling ponds and recirculated. Intake structures for these systems consist of circulating water pumps mounted in a reinforced concrete pump well with inlets pulling water directly from cooling towers or ponds. Bearing walls are reinforced concrete and function as shear walls to resist lateral loads. Reinforced concrete or reinforced masonry block walls separate different equipment divisions or trains to minimize potential for common mode events. The floor and roof are constructed of reinforced concrete and are supported by steel beams. They are designed as diaphragms to transmit lateral loads to shear walls. Where intake structures are part of the ultimate heat sink, they are designed to withstand natural phenomena such as earthquakes, seiches and tsunamis.

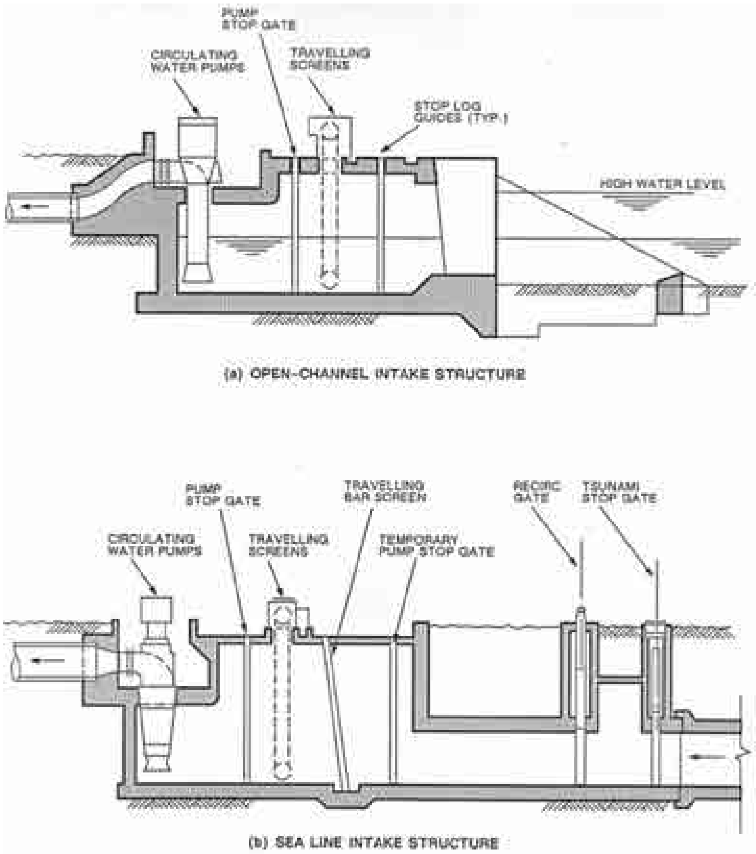


FIG. 12. Typical open cycle intake structure [22].

3.3.4. Concrete pipe

As described in Ref. [23], concrete pipes usually fall into the types listed in Table 3. Such piping is typically used in large diameter underground water and sewage systems. An example of prestressed concrete cylinder pipe (PCCP) is shown in Fig. 13.

TABLE 3. TYPES OF CONCRETE PIPE

Type	History	Scope of use	Typical available diameter	Typical available length
Reinforced concrete pipe (non-cylinder)	Manufactured since 1900 due to sanitary sewer system issues.	Designed for low internal pressure (up to 379 kPa). Limited in external load capacity.	300–3600 mm	2.5–7 m
Reinforced concrete cylinder pipe (RCCP)	Manufactured since 1940. Steel cylinder core provides watertightness while outer layer made of reinforced concrete provides mechanical strength.	Designed for high internal pressure (over 2758 kPa). Limited in external load capacity.	600–3600 mm	4–7 m
Prestressed concrete cylinder pipe (PCCP)	An evolution of reinforced concrete cylinder pipe that has been manufactured since 1942. Consists of a concrete core, a thin steel cylinder, high tensile prestressing wires and a mortar coating. Concrete core is the main structural load-bearing component with the steel cylinder acting as a water barrier between concrete layers. Prestressing wires produce uniform compressive pressure in the core, which offsets tensile stresses in the pipe. The mortar coating protects the wires from physical damage and corrosion.	Designed for high internal pressure (over 2758 kPa) and external load capacity (earth cover in excess of 30 m).		5–7 m
Concrete bar wrapped cylinder pipe	Manufactured since 1942. While the cross-section of a bar wrapped pipe looks like that of a PCCP, their design and materials are significantly different. PCCP is a concrete pipe that remains under compression because of prestressing wires, whereas in bar-wrapped pipe, the cylinder plays a key role in structural integrity.	Designed for high internal pressure (over 2758 kPa) or external load capacity.		7–12 m
Polymer concrete microtunnelling pipe	Developed in the 1960s in Germany, but was not in general use until the 1980s. Made from thermosetting resin, sand, gravel and mineral fillers. Has the load carrying capability of reinforced concrete with anti-corrosive characteristics (pH1–10).		15 mm–3 m	0.9–3 m

3.3.5. Concrete masonry

Concrete masonry structures are used for structural walls, pipe and equipment supports, partitions and shield walls [22]. When masonry walls are used to support safety related piping, raceways and equipment, they are considered safety related structures [24]. Masonry structures may be in single or multiple widths and can be designed as bearing walls, shear walls and piping or equipment support walls. Concrete masonry structures

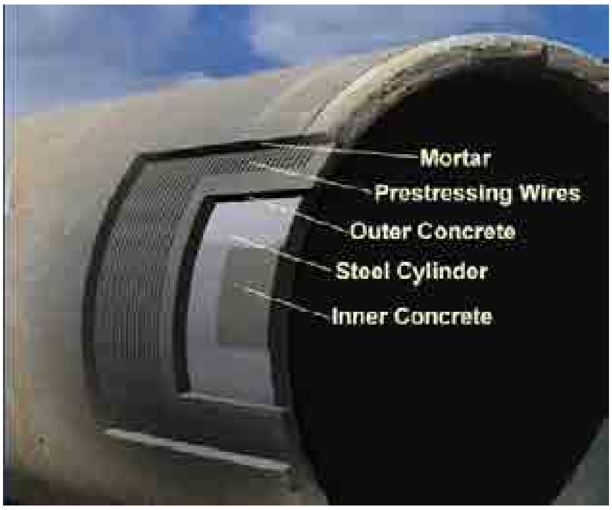


FIG. 13. PCCP typical cross-section (courtesy of Pure Technologies Ltd).

are generally constructed from concrete and grout and their cells may or may not contain horizontal or vertical reinforcing steel to provide additional structural strength. For structural block walls, the extent of grouted cells varies with specific wall design requirements (e.g. for shield walls, all cells are grouted to provide the desired shielding effect).

3.3.6. Others

Concrete may also be used in many other parts of the power plant, such as for spent fuel storage canisters, for reservoirs for water storage, as part of flood or tsunami protection barrier systems, or as part of security related intrusion delay or physical protection systems.

3.4. RELATED COMPONENTS

3.4.1. Structural anchorages

Concrete anchors or screws are required for heavy machinery, structural members, piping, ductwork, cable trays, towers and many other types of structures and components in NPPs. Anchorage requirements may relate to ease of installation or inspection, load capacity, resistance to vibration, preload retention, temperature range, corrosion resistance, stiffness, and the timing when the embedded anchor is installed (i.e. pre-installation or post-installation) [25]. It is preferred to use embedded anchors wherever possible. Where embedded anchors have not been provided during construction (for example in case of modifications, or following installation misplacement), post-installed anchorage systems can be used.

Post-installed concrete anchors are of two primary types, mechanical and bonded. Figures 14–16 present examples of mechanical anchors. The embedment depth, hole diameter and torque applied are critical to proper installation. Bonded anchorage systems can be grouted (i.e. bonded to concrete using cementitious grout) or an adhesive can be used, which chemically bonds the anchor to the substrate.

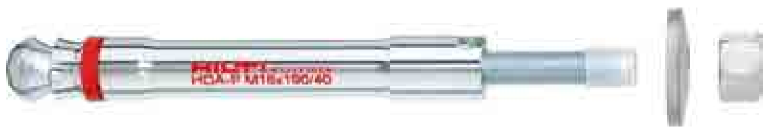


FIG. 14. Example of post-installed undercut anchor for NPP applications (keying effect provides secure hold even in cracked concrete under dynamic loading) [26].

FIG. 15. Example of post-installed, heavy-duty expansion anchor (for use in cracked or uncracked or lightweight concrete) [26].

FIG. 16. Example of post-installed expansion anchor (for use in cracked or uncracked or lightweight concrete, and with lightweight concrete over metal decks) [26].

3.4.2. Foundations, piles and underground structures

Machine foundations are usually made of reinforced concrete. In NPPs, these foundations serve as support structures for machines with rotating masses, such as turbines, generators, pumps or blowers. Foundations are also used to support heavy equipment such as electrical power transformers. Such equipment is often safety related or has significance in power production. Foundations can be divided into table, spring, raft or floor foundations [27].

As a rotating machine and foundation form an integral dynamically loaded system, changes in foundation properties may influence the ability to prevent impermissible vibrations. Changes may, for example, alter natural frequencies of the vibrating entity and affect shaft train alignment, or may impact machine anchoring. With respect to dynamic behaviour, ageing management needs to include both the foundation and the machinery resting on it.

In extreme situations, ageing of machine foundations may affect load-bearing capacity. Typical machine induced extreme loadings can be caused by loss of bucket (turbine blade failure), short circuits, or faulty synchronization, which typically are considered in foundation design. Equipment foundations may have special requirements, for example, to act as a catch basin in the case of oil leakage or spills from transformers. In these cases, the tightness/permeability of the structure is important.

Foundations for containment are fabricated from reinforced concrete and can range in thickness from approximately 2.7 to 4.1 m, or more. The basemat may consist of a simple mat foundation on fill, natural cut or bedrock, or may be a pile/pile cap arrangement. Most plants have used the simple mat on fill or bedrock design.

Either concrete or steel piles are used to transfer foundation loads to greater depths below grade to support safety related structures in NPPs [22].

Concrete foundations for outdoor electrical transmission related components are typically partially embedded in soil with the top portion accessible for visual inspection.

Underground structures are typically fabricated from cast-in-place wall and slab reinforced concrete.

3.4.3. Waterstops, joint sealants and gaskets

Waterstops are static seals used between joints in concrete structures to prevent passage of water or other fluids. Waterstops are embedded in concrete or across and/or along joints. Waterstops are used in containment, SFPs, wastewater treatment plants, secondary hazardous fluid containment structures, tunnels, pools, water reservoirs, etc.

Joint sealants are used for caulking and sealing joints subject to concrete movement to provide a firm, flexible, weathertight seal. They may be used to enhance leaktightness within containment structures.

To select the right waterstop and/or joint sealant for an application, one needs to consider structure type, joint type, joint movement, environmental exposure and chemical containment requirements. Methods for securing waterstops in place (hog rings, grommets, etc.) and duration of ultraviolet exposure during construction must also be considered.

Gaskets are used for sealing mating surfaces between concrete pipe and structures such as culverts or manholes. They are typically made of elastomeric materials depending on service requirements.

3.4.4. Non-metallic liners, sealers and coatings

Non-metallic containment liners typically use epoxy or polyurethane based materials applied to concrete surfaces. Epoxy liners used for concrete surfaces in contact with water (e.g. dousing tanks or spent fuel bays) are often reinforced with fibreglass.

Protective coatings are applied to metallic liners, structural steel and concrete to provide barriers to environmental stressors. Common interior coatings applied to carbon steel surfaces are inorganic zinc primer and top coat, inorganic zinc primer and a modified phenolic or epoxy enamel top coat, and modified phenolic primer and top coat. Inorganic zinc primers and top coats have been applied to exterior carbon steel surfaces.

Sealers are liquids applied to hardened concrete to either prevent or decrease penetration of liquid or gaseous media (e.g. water, carbon dioxide or aggressive chemicals) [28]. Coatings or membranes are different from sealers in that they are applied to concrete in some thickness, generally measured in hundredths of a millimetre and generally do not penetrate concrete. Typical sealers, coatings and membranes that have been applied to concrete are identified in Section 8.2.1.

Materials selection needs to ensure non-interference with any normal operations or safety functions, for example, by deteriorating and causing clogging of the filters of sumps or as a result of the formation of organic iodine [29].

3.4.5. Cathodic protection systems

Cathodic protection systems can be installed to help prevent or mitigate corrosion of concrete rebar, other concrete reinforcement or concrete pipe. Systems can either use sacrificial anode cathodic protection (SACP), impressed current cathodic protection (ICCP), or both. SACP systems connect the protected metal to a more easily corroded sacrificial metal to act as the anode. The sacrificial metal then corrodes instead of the protected metal. SACP systems have limited current capability and where available protection current is not adequate, ICCP systems using an external electrical power source are used to provide sufficient protective current.

For concrete buildings, bridges and similar structures, anodes and reference electrodes are usually embedded during construction when concrete is poured. For pipes, the anodes are connected directly to the pipe (typically through a test post) and buried relatively close to the protected pipe in specially prepared anode beds.

SACP systems are simple, easy to retrofit, and do not require electric power or control systems. They do, however, require anode replacement at regular intervals (anodes typically are designed to have 10 to 15 year service lives). ICCP systems are more complex but have the advantages of better control, increased protective currents and potential for remote monitoring and surveillance. ICCP systems are more common at NPP sites due to the typically large current requirements needed to overcome impacts of large local grounding grids and interference from other metal structures and pipes.

The use of ICCP for PCCP pipe is similar to that for common steel pipelines. Applied potentials, however, must be limited to prevent damage to prestressing wires. Overprotection of PCCP wires can cause hydrogen embrittlement leading to their failure. Lower current SACP systems can be used in some locations to minimize this potential for embrittlement. The National Association of Corrosion Engineers (NACE International) has produced the recommendations document SP0100 for PCCP cathodic protection [30].

4. UNDERSTANDING AGEING OF CONCRETE STRUCTURES

4.1. BACKGROUND

Understanding relevant ageing mechanisms and their potential impact on concrete structures is the key to an effective, optimized AMP. As Fig. 1 shows, an understanding of ageing mechanisms will affect all areas of an AMP. For example, it helps to define the:

— Parts of concrete structures that are susceptible to degradation;
— Key degradation mechanisms, their symptoms and potential rates of action;
— Impact of degradation on a structure's ability to perform safety functions;
— Appropriate remedial action.

Plant condition evaluation always calls for informed judgment. Developing appropriate understanding is a continuous process and builds on experience. Informed judgment is often based on a review of the generic body of knowledge and is supported by a detailed understanding of the mechanisms involved, all put into a plant specific context.

Subsequent parts of this section cover the typical knowledge required to help individuals involved in concrete AMPs to make informed judgements. This includes an understanding of material properties, methods of construction, structure models, stressors, operating conditions, ageing mechanisms, sites of degradation, degradation consequences, relevant research and development, relevant operating experience (OPEX), maintenance history, mitigation measures and current structure status.

This section is based on and expands upon information found in NUREG/CR-6927 [9].

4.2. MATERIALS AND CONSTRUCTION

Nuclear safety related concrete structures are composed of several constituents that perform multiple functions (e.g. load carrying, radiation shielding and leaktightness). These can include concrete, conventional rebar, prestressing steel and liners. Material quality is established through regulations, qualification tests and certification, followed by checks during construction. More information on materials of construction is available in Refs [31–33].

4.2.1. Concrete

Concrete is a composite material made of water, coarse or fine aggregate filler particles and a binder known as cement. Cement is a very fine powder made of limestone and other minerals, which absorbs water and acts as a binder to hold the concrete together. While cement is a construction material in its own right, concrete cannot be made without cement (see Fig. 17).

Cement is made by grinding crushed limestone, clay, sand and iron ore together to form a homogeneous powder. The powder is then heated at very high temperatures ranging from 1400–1600°C to form a clinker [32]. The term clinker refers to lumps or nodules, usually 3–25 mm in diameter. After cooling, the clinker is ground and mixed with gypsum to regulate setting and to facilitate placement. This produces general purpose Portland cement, which is mixed with water to produce cement paste that binds the aggregate particles together. The term Portland cement originated in the 19th century, and refers to the similarity of the cement to Portland stone found on the Isle of Portland in Dorset, England.

Current generation cements have higher C_3S content and are ground finer than previous cements. They attain most of their compressive strength within a 28 day period, whereas previous generation cements continue to gain strength after 28 days [34, 35].

Cement is the most expensive ingredient in concrete, and its production causes environmental impacts in the form of CO_2 and heavy metal emissions. It is, therefore, desirable to use the minimum amount of cement necessary to produce the desired properties and characteristics of concrete.

FIG. 17. Concrete constituent components (adapted from Ref. [9]).

Portland cements are composed primarily of four chemical compounds: tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A) and tetracalcium aluminoferrite (C_4AF). The type of Portland cement produced depends on relative amounts of four basic chemical compounds and their fineness. Mixes can be customized to adjust properties such as sulphate resistance, heat of hydration and early strength. Calcium silicate hydrates constitute about 75% the mass of Portland cement. The calcium silicate hydrate gel structure is made up of three types of groups that contribute to bonds across surfaces or in the interlayer of partly crystallized tobermorite material: calcium ions, siloxanes and water molecules. The bonding of water within layers (gel water) with other groups, via hydrogen bonds, determines the strength, stiffness and creep properties of the cement paste.

Alternative cementing agents have been used in conjunction with or as a partial replacement for Portland cement. These include pulverized fly ash, ground granulated blast furnace slag and silica fume. Concrete mixes containing these materials are workable and, depending on the material used and its amount, may generate less heat of hydration and exhibit less shrinkage and reduced cracking potential. In some applications, up to 35% of Portland cement has been replaced with these materials. Fly ash is collected from the exhaust flow of furnaces burning finely ground coal, and reacts with calcium hydroxide and water to form cement compounds consisting of calcium silicate hydrate. Ground granulated blast-furnace slag is a by-product of iron-making and is formed by taking hot slag, rapidly chilling or quenching it and grinding it into a powder. When mixed with water in the alkaline environment provided by Portland cement, ground, granulated blast-furnace slag hydrates to form cementing compounds consisting of calcium silicate hydrate. Silica fume is the condensed vapour by-product of ferro-silicon smelting. Silica fume reacts with calcium hydroxide in the presence of water to form cementing compounds consisting of calcium silicate hydrate. High alumina cement, consisting mainly of calcium aluminates, has been used as a cementitious material because of its rapid set and rapid strength gain characteristics and resistance to acidic environments, seawater and sulphates. Under certain conditions of temperature and humidity, calcium aluminate cement converts, over time, to a different hydrate (i.e. increased porosity and reduced strength), therefore it is recommended that calcium aluminate cements not be used for structural applications (particularly in wet or humid conditions above 27°C) [36].

Several NPPs have used high alumina cement to produce porous concrete subfoundations. Although some erosion of cementitious materials has occurred in at least one plant, the amount of material removed has been insignificant and the plants are monitored for signs of excessive settlement [37].

The selection of proper concrete water content is critical. Too much water reduces strength and durability, while too little water causes the concrete to be unworkable and may result in limited hydration. Hydration is a chemical reaction in which major cement compounds form chemical bonds with water molecules and become hydrates, which leads to concrete hardening. Hardened cement paste consists of calcium silicate hydrates, calcium hydroxide and lower proportions of calcium sulphoaluminate hydrate either as ettringite or monosulphate. About 20% of hardened cement paste volume is calcium hydroxide. The pore solution is normally a saturated solution of calcium hydroxide in which high concentrations of potassium and sodium hydroxides are present. Proper curing is essential as it affects concrete durability, strength, watertightness, abrasion resistance, volume stability and resistance to freezing and thawing.

Concrete contains about 60–75% aggregate by volume, 10–15% cement, 15–20% water and air (5–8% if entrained). The aggregate strongly influences chemical, physical and thermal properties, mix proportions, construction costs and concrete durability. Aggregates come in various shapes, sizes and materials ranging from fine sand to coarse rocks, with selection determined in part by desired concrete characteristics. Materials are available ranging from ultra-lightweight (e.g. vermiculite and perlite) to lightweight (e.g. expanded clay shale or slate-crushed brick) to normal weight (e.g. crushed limestone or river gravel) to heavyweight (e.g. steel or iron shot). Chemical or mineral admixtures can be added during mixing to enhance durability (air entrainment), improve workability (enhanced placement and compaction), modify hardening and setting characteristics, aid in curing, reduce heat evolution, or provide other property improvements [38]. Heat of hydration typically has to be considered for massive nuclear concrete structures.

Concrete typically used in NPPs consists of moderate heat of hydration or sulphate resistant cement, fine aggregates (e.g. sand), water, various mineral or chemical admixtures for improving properties or performance of concrete, and either normal weight or heavyweight coarse aggregate. Both water and aggregates are normally acquired from local sources and are subjected to material characterization testing prior to use. Coarse aggregate can consist of gravel, crushed gravel or crushed stone. Chemical (e.g. air entraining or water reducing) or mineral (e.g. fly ash or ground granulated blast furnace slag) admixtures, typically referred to as supplementary cementing materials, have been used in many of the mixes to improve concrete characteristics or performance. Heavyweight or dense aggregate materials, such as barites, limonites, magnetites and ilmenites, can be used in (biological) radiation shielding applications.

Hardened concrete typically provides the structure's compressive load carrying capacity. Specified concrete unconfined compressive strengths typically range from 13–55 MPa, with 35 MPa being a typical value achieved at 28 days of age. According to requirements and special applications, concretes with higher compressive strengths of up to 60 MPa (high strength concrete) are being specified for some NPPs, with 120 MPa compressive strength concretes having been used in commercial high-rise construction [39]. The American Concrete Institute (ACI) defines high strength concrete as concrete with a compressive strength of greater than 41 MPa [40].

Recently, other concrete types have become candidates for use in NPP construction. Examples are:

- Precast concrete for repetitive structures (e.g. modular), for manholes or utility vaults designed and manufactured for simple connection.
- Short, discontinuous, natural and synthetic fibres (e.g. carbon, fibreglass and steel) incorporated into concrete to increase cracking resistance and improve tensile strength and toughness.
- Self-consolidating (or self-compacting) concrete, a flowing concrete mixture able to consolidate under its own weight. Self-consolidating concrete is used for placement in difficult conditions and in sections of congested rebar and requires less time to place large sections. Care needs to be taken when using self-consolidating concrete as its high fluidity makes it prone to segregation, which is aggregate separation from cementitious material causing concrete proportions to be different than designed.

4.2.2. Conventional steel reinforcement (rebar)

Concrete tensile strength is about one-tenth to one-fifth its compressive strength and cannot be relied upon to withstand very high tensile stresses. This is overcome by embedding rebar (Fig. 18) within the concrete so that a composite material is formed with both compressive and tensile strength. Bonded rebar resists the tensile loads, controls the extent and width of cracks, resists inclined tensile stresses caused by shear forces and assists in resistance of compressive forces, especially where it is desirable to reduce member cross-sections. Rebar is also used in compression members to guard against unanticipated bending moments that can crack or fail a member. The effectiveness of reinforced concrete as a structural material depends on interfacial bonding between steel and concrete, the passivating effect of the highly alkaline concrete to inhibit steel corrosion and similar coefficients of thermal expansion of concrete and steel [9].

Reinforced concrete has been used in all NPPs. Most mild, or conventional, rebar that is used in NPPs consists of plain carbon steel bar with surface deformations (lugs or protrusions). This material has a minimum yield strength that ranges from 280 MPa to 520 MPa.

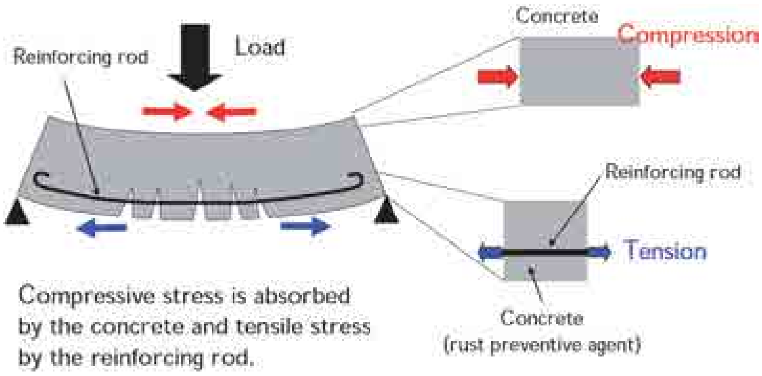


FIG. 18. Steel reinforcement [41].

More recent construction has incorporated advanced installation techniques, such as modularized rebar (installed in prefabricated solid modules or rolled out similar to a carpet), deck plate methods and improved connection methods [39].

4.2.3. Prestressing steel

Prestressed concrete is concrete that has had internal stresses introduced to counteract, to the degree desired, tensile stresses that will be imposed in service. Post-tensioning uses high strength steel wire, strands or bars referred to as tendons (Fig. 19) to apply such stress. The tendons resist tensile loadings and are used to apply compressive force to the concrete, which provides increased resistance to applied loads. Tendons are installed, tensioned and anchored to concrete after it has hardened, hence the term post-tensioning. Such systems are generally used in conjunction with conventional (non-tensioned) rebar. Conventional rebar is considered passive, whereas a post-tensioning system can be considered active.

Tendons are installed within preinstalled ducts in containment structures and post-tensioned from one or both ends after concrete has achieved sufficient strength. After tensioning, tendons are anchored by buttonheads, wedges or nuts. Corrosion protection is provided by filling ducts with wax, corrosion-inhibiting grease (unbonded) or Portland cement grout (bonded). Some prestressed concrete pressure vessels are circumferentially prestressed by wrapping wire or strands under tension around the vessel circumference. Supplemental conventional reinforcing is also used to minimize shrinkage or temperature effects and to provide local load carrying capacity or load transfer [9].

Materials used for these systems typically have minimum tensile strengths ranging from 1035–1860 MPa. Typical NPP tendon systems group sufficient numbers of wires, strands or bars to have minimum ultimate strengths ranging from 2000–10 000 kN. The trend has been to increase individual tendon strength and to reduce the total numbers required. For example in the early 1970s a typical tendon had a capacity of 3000 kN, while by the early 2000s this had progressed to capacities of 10 300 kN and 15 300 kN [42].

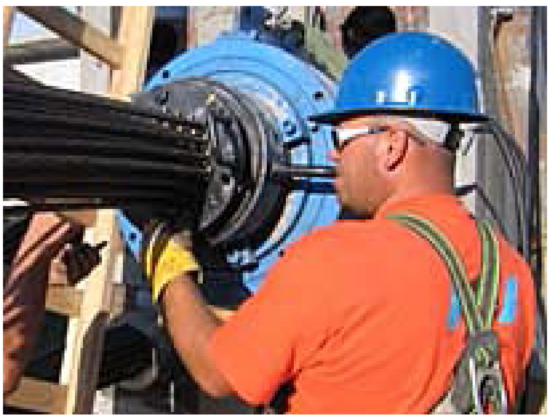


FIG. 19. Steel tendons (courtesy of Oak Ridge National Laboratory).

Relaxation is a property of prestressing steel material whereby the force required to hold the highly stressed steel wire at a given elongation will reduce with time. There are two major types of tendons: low relaxation tendons and stress relieved (normal relaxation) tendons. As per standard A416 of ASTM International (formerly known as the American Society for Testing and Materials) [43] relaxation of low relaxation tendons is limited to 2.5% when initially loaded to 70% of specified minimum breaking strength, or to 3.5% when loaded to 80% of specified minimum breaking strength of the strand after testing for 1000 hours at 20°C, and other conditions as listed in the standard. Relaxation of stress relieved wires is greater than the relaxation of low relaxation wires and is generally about 6–10% when tested in accordance with ASTM A421 [44].

4.2.4. Metallic liners

Liner plate material (Fig. 20) is typically mild steel, such as ASTM A516 [45], with large ductility. Corrosion of the exterior of such steel is expected to proceed at a very slow rate due to stabilization of a passive oxide film on the steel in the alkaline concrete environment. Nevertheless, as discussed later in Section 4.5.3.1, there have been incidents of corrosion affecting containment liners.

4.2.5. Non-metallic liners, sealers and coatings

Typical non-metallic liners, sealers, coatings and membranes that have been applied to concrete were previously discussed in Section 3.4.4.

4.2.6. Materials databases

Detailed qualitative assessments may be required to evaluate the likely performance of an inaccessible structure, or in response to detecting symptoms of degradation. These assessments benefit considerably from access to a generic database of material performance over time in a given environment. Such databases exist, but they often contain proprietary information.

Published data typically relates to a single mechanism. When applying this data to a plant, care needs to be taken to understand the potential synergies between degradation processes.

One example of a materials property database is the web based Nuclear Concrete Materials Database (NCMDB) developed at Oak Ridge National Laboratory (ORNL), which can be accessed globally

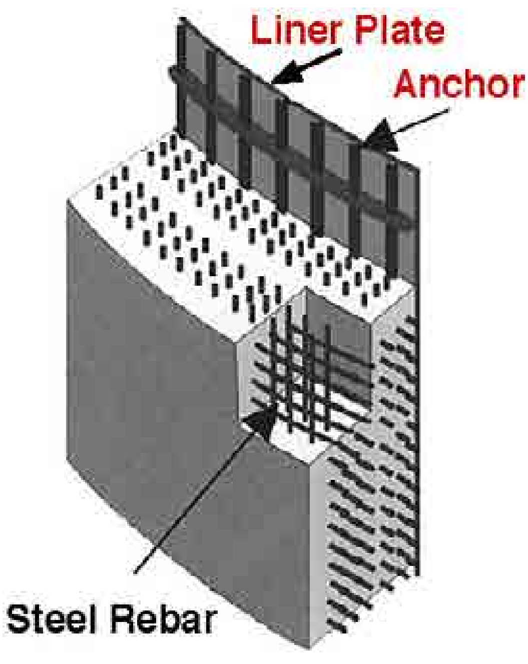


FIG. 20. Metallic liner plate adjacent to reinforced concrete.

(http://ncmdb.extranet.ornl.gov). This database contains historical and new data for concrete and concrete related materials used in design and construction of nuclear systems. The advanced materials property information management system developed for the Gen IV Nuclear Energy Systems Program, the Gen IV Materials Handbook System [46], is being used to develop the NCMDB to take advantage of the information management infrastructure already in place. The development of the NCMDB has been divided into two phases.

In Phase I, a document database was designed and constructed to store and manage historical data and information on concrete materials. The focus of Phase I development was on maintaining the characteristics of original historical data and information documents that are familiar to scientists and engineers who have been working with such documents. Information and historical data have been uploaded into a well-organized database structure with rudimentary search abilities.

In Phase II, a digitized database is to be designed to store historical and newly generated data, with a focus on enabling advanced data processing functionalities. Information will be stored and managed in a digitized form with search, report, tabulation, plot, comparison and other desirable data processing and information management capabilities.

The database will not only store data and information, but will also register relationships between data and information to enable traceability to satisfy pedigree and prediction research needs. Both the previous Structural Materials Information Center (SMIC), developed under the Nuclear Regulatory Commission (NRC) Structural Aging Program, and the NCMDB use the same format as the Nuclear Systems Materials Handbook, which was developed in the 1970s. Initial data and information input into the NCMDB was provided by the SMIC [47].

In the Republic of Korea, degradation mechanisms were investigated and systematic inspection procedures were developed for NPP concrete structures between 1993 and 1996. A structural ageing management system was developed to manage imaging data related to these inspections [48]. During the application of this system for NPP structures, problems were identified associated with the accuracy of computerized inspection data as a result of large data storage requirements. This resulted in the development of a web based Structural Life Management System (SLMS) database that encompasses the design, construction and maintenance phases of NPPs. The database contains general information about structures, field inspection data and repair data. SLMS focuses on the management of degradation data obtained during periodic inspections to predict structure or subelement service life based on field inspection data, primarily involving carbonation and chloride penetration. SLMS allows for the input and review of inspection results using numerical coordinates and structure management by means of user-friendly graphical windows. Periodic inspection data include results related to cracking, spalling, efflorescence, corrosion, chloride content and carbonation depth. Input data (e.g. design and construction information, and inspection results) is digitized and checked for reliability prior to storage in the main database. During data input, the SLMS provides an integrity assessment function that computes an integrity index based on the extent of degradation and state (level) of degradation for a structure or subelement. SLMS also provides an index management tool to address indices necessary for integrity assessments. The integrity management tool controls the member's importance factors, weights of integrity loss factors and integrity indices of the structure and subelements used in integrity assessment. The data search programme provides search functions by NPP unit, structure, degradation occurrence by structural member and level of degradation. Following development in 1998, the SLMS has been applied to Korean NPPs to evaluate ageing, assess structure integrity and to plan repairs.

In Finland, public concrete material databases are related to concrete façades and highway applications and are connected to research at Aalto University or the VTT Technical Research Centre of Finland. For example, the façade repair database includes deterioration parameters of condition surveys and provides an informative and flexible means of accessing data from condition surveys of concrete façades [49]. At the beginning of the 21st century, a series of long term durability tests with different concrete and rebar types began under both laboratory and outdoor conditions [50]. Field testing is ongoing under Nordic climate highway conditions [51]. Investigated phenomena include carbonation rebar corrosion frost deterioration and chloride penetration. In Finland, concrete will be an engineered barrier for final disposal of nuclear waste. As service life of these structures will be hundreds of years, some long term tests of concrete samples started in 1997. These samples have continually been followed and used in research [52].

4.2.7. Design considerations

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 [53]. Inadequate structural design occurs when structures are exposed to loads greater than they are capable of carrying or they sustain more than their strain capacity. Attention to detail errors include inadequate consideration of temperature changes, concrete creep, accidental impacts, or poor structural detailing (which can cause local stress concentrations). Typical symptoms of inadequate design include spalling and cracking of concrete. Cracking can in turn permit water or chemicals to have access to the concrete, or may result in ponding of water, both of which may produce saturated concrete. Poor detailing does not generally lead directly to concrete failure, but can contribute to other causes of concrete failure [53]. Examples of inadequate structural design include: insufficient concrete cover over rebar, improper sizing and placement of rebar, inadequate section geometry, inadequate provision for drainage, abrupt changes in section, improperly selected material, material incompatibility and inadequate provision for deflection [9].

Designers need to consider that concrete structures may need to function for longer than the planned service life of 30–40 years that was typical of Generation II NPPs. Longer service life, life extensions, long term storage and decommissioning periods all need to be taken into account.

Materials need to be properly selected considering anticipated exposure conditions as well as longevity. The compatibility of materials needs to be ensured. Materials specified for use in nuclear structures 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. For example, materials such as non-metallic liners and coatings are applied as a system and need to be qualified as such including elements such as substrate (e.g. concrete or steel), number and consistency of layers, presence of primer, film thickness of each layer, ambient conditions and time of application and curing of each layer.

Technical specifications need to include requirements for durability parameters that are measurable within a reasonable time frame (a few months) after construction. Examples of typically measured parameters would be compressive strength and air entrainment content. Depending on the exposure environment, structure geometry and loading, some parameters such as heat of hydration development, permeability of concrete cover and crack size may be specified to ensure long term durability of the structure.

Codes and standards that exist for civil structures need careful review to determine their applicability in NPP environments, as the requirements for NPPs can differ substantially from general civil construction. For example, models for creep and shrinkage prediction are typically based on data collected in average ambient environments that are generally not the case for CCSs.

Instrumentation to facilitate ongoing concrete monitoring and assessment is now available to be incorporated as part of structure design. This is more fully discussed later in Section 7.3.2.

In some jurisdictions there are environmental preferences for recirculating water supplies, such as cooling towers or reclaimed water sources (municipal waste water or brackish water) as opposed to once through cooling [54, 55]. This will lead to an increased use of cooling towers and increased use of aggressive water sources in such towers (preferential use of degraded water sources) [56–58]. This will result in a higher risk of rebar corrosion and perhaps an increased use of structural materials other than concrete (e.g. fibreglass) in cooling tower designs.

4.2.8. Construction considerations and quality assurance

4.2.8.1. General

Experience has shown that over 50% of ageing problems in concrete structures are associated with construction defects (Fig. 21). Flaws such as 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.

Results of a 2010 Electric Power Research Institute (EPRI) study [60] showed that while the general performance of concrete civil infrastructure in US NPPs was good, several instances of degradation were identified, the majority of which were the result of poor construction practices. This study is a literature review based on available information mostly from plants of up to 40 years of operation.

FIG. 21. Where problems arise [59].

Results of a survey of the performance of NPP concrete structures, including about half of the commercial NPPs in the USA, are documented in Ref. [9]. These results indicate that most concrete structure problems were a result of design and construction errors, that have since been corrected. The problems resulted from not following specified procedures and a lack of attention to detail. Foreign material has been shown to initiate concrete liner corrosion, signifying that quality control and quality assurance programmes at NPPs are necessary to ensure the production of good quality concrete (e.g. material selection, batching, mixing, placing and curing).

Poor construction practices (e.g. increasing water in concrete mix to facilitate placement or finishing, improper mixing and curing, improper consolidation and improper location of rebar, etc.) do not lead directly to failure or deterioration of concrete, but can cause defects that lead to concrete cracking. Cracks can be formed due to plastic shrinkage, plastic settlement, early thermal contraction, crazing and long term drying shrinkage. The cracking enhances the adverse impacts of degradation mechanisms and leads to further concrete degradation ('degradation' throughout this publication refers to 'concrete degradation'). Poor construction practices are best addressed through quality assurance/quality control in conjunction with an aggressive inspection programme. Processes need to be implemented to monitor the quality of fresh and hardened concrete, concrete placement and vibration processes [9].

To assist with construction considerations and quality assurance, EPRI has developed a field guide to conducting quality inspections and tests of concrete placement for nuclear facilities [61].

Careful material selection and specification is also important for long term structure durability. Factors that may have negative effects on durability include:

— Improper cement content;
— Use of poor quality or contaminated aggregates;
— Incorporation of additives that can produce corrosion, such as calcium chloride accelerators;
— Incorrect water to cement ratios [9].

As per Safety Guide NS-G-2.12 [1], the following needs to take place during the construction stage:

— Known factors affecting ageing management of SSCs provided to manufacturers and constructors;
— Current knowledge about relevant ageing related degradation mechanisms (ARDMs), their effects, degradation and possible mitigation measures addressed during fabrication and construction;
— Collection and documentation of baseline data;
— Surveillance specimens for specific ageing monitoring programmes made available and installed (e.g. construction samples where applicable).

4.2.8.2. Lessons learned from construction experience

To address known factors that affect ageing management, lessons learned from construction experience needs to be considered when planning for and constructing nuclear concrete structures.

As per NRC Information Notice 2008-17, Construction Experience with Concrete Placement [62], a number of issues were identified during US plant construction over 20 years ago, as well as more recently in Europe. Issues identified during recent construction at three facilities, Olkiluoto 3 (Finland), a Mixed Oxide Fuel Fabrication Facility (USA) and Flamanville 3 (France), included the following:

— Inconsistencies between different truckloads of concrete caused by use of plasticizer;
— Water to cement ratio outside the design requirements;

— Changes in concrete composition made by the contractor outside approved procedures;
— Concrete slump being outside design values, caused at the batch plant by adding chemical plasticizer and water exceeding design specifications;
— Problems with bent rebar;
— Rebar placement not in accordance with approved procedures and design requirements;
— Improperly spliced and tied rebar;
— Rebar placement not following construction specifications and drawings.

The main issues identified were a lack of contractor oversight and poor quality control in concrete placement. The importance of adequate quality control and quality assurance from the beginning of the construction project cannot be overemphasized.

Specifically, based on lessons learned from construction experience documented in IN 2008-17 [62], necessary controls need to be ensured during the following activities:

— Concrete batch plant inspections and truck mixer quality checks (e.g. slump tests);
— Rebar bending;
— Concrete placement and consolidation using vibrators;
— Quality control checks of rebar installation (prior to concrete placement);
— Visual inspection of concrete placement activities;
— Concrete testing (e.g. testing of cylinder samples and compressive strength tests).

The UK's Royal Academy of Engineering published a study, Nuclear Construction Lessons Learned, which documents lessons learned as they apply to concrete for new nuclear plant construction [63]. The study is generally applicable to the broader nuclear industry. The study's key lessons learned are as follows:

> "In conventional reinforced concrete construction, significant emphasis is placed on the assessment of concrete quality following placement through inspection, cube strength tests and cover meter surveys. This approach operates on the traditional basis that if quality is not achieved, the contractor may be asked to 'break it out and start again'. However, the massive sections that are typical of the nuclear industry make this approach impractical for much of new build nuclear construction. More crucially, beyond the impracticality of such measures, failing to meet the concrete quality requirements will inevitably cause programme delay, dissatisfaction in client and contractor teams and critically, a loss of both regulator and investor confidence.
>
> "Therefore, for nuclear construction, the emphasis must be moved from post-placement verification of quality, to pre-placement quality of design, specification and training to minimise the potential for defects. Details such as cover and reinforcement position must be verified before placing concrete and all items must be secured to ensure that nothing occurs during the pour to change this."

Furthermore, the study detailed 14 recommendations [63]:

(1) "**Need for pre-placement quality assurance** – given the key role of concrete in nuclear structures and the difficulty of modifying it later in the construction or operational phases, licensees should move the focus from the traditional post placement verification of quality to pre-placement quality of design, specification and training."
(2) "**Licensee oversight** – the licensee must manage the core capabilities of the integrated design and construction programmes to ensure that the arrangements for oversight of work carried out by the supply chain guarantee quality work that meets the licensee's ultimate responsibility for quality and safety."
(3) "**Early contractor engagement** – early contractor engagement should be undertaken to ensure comprehensive integration of the design with the construction approach for complex areas."
(4) "**Integrated design and construction programmes** – design and construction programmes must be realistic and fully integrated across all disciplines with appropriate allowance for approvals and contingency. In particular; mechanical, services and process plant design should be sufficiently well advanced as to

allow design of encast items penetrations and equipment. Co-location of the design and construction teams is desirable."

(5) "**Specification** – the specification should be comprehensive, achievable and well understood across the supply chain. This will ensure that it can be enforced as mandatory."

(6) "**Need for understanding across the team** – concrete has an important role to play in ensuring nuclear safety. Everyone involved in the concrete process must understand the importance of producing high quality durable concrete and the procedures and specifications for concrete works."

(7) "**Designers need practical experience** – design and technician staff should gain practical site experience of constructing heavily reinforced concrete structures to understand the contractor's challenges and constraints. Such experience should be an integral part of a formal training programme for all design and detailing staff."

(8) "**Importance of Technician role** – the role of the lead technician should be recognised as key to the successful delivery of the construction design, as part of a core team of experienced staff specifically identified, trained and supported appropriately."

(9) "**Integrated and visible quality assurance process** – to achieve the required long-term durability properties the licensee should ensure that there is an integrated approach to quality management, achieved by creating an attitude of teamwork among all parties involved."

(10) "**Concrete mix design** – All concrete mixes should be designed for all relevant properties, tested and approved. Any changes to the approved set of mixes should be formally controlled."

(11) "**Concrete placement** – good preparation will ensure that concrete can be place[d] efficiently and correctly. Any problems should be resolved before the pour, no[t] during or after it."

(12) "**Treatment of non-conformance** – licensees should give consideration to how non-conformances will be addressed and should make this clear to all parties. They must then follow that process if non-conformances arise."

(13) "**Feedback processes** – processes should be put in place across all developers to allow for the collection, analysis and implementation of lessons learned and experience feedback."

(14) "**Knowledge transfer over plant lifetime** – the licensee should ensure that mechanisms are in place for the transfer of knowledge and experience at each stage of the project and the management of ageing from the conceptual stage onwards."

The Finnish Radiation and Nuclear Safety Authority (STUK) produced an investigation report [64] regarding the Olkiluoto 3 project. This report assessed compliance with safety requirements during construction. STUK's conclusions were that quality control was implemented largely as planned, but that the quality control organization may not have had enough opportunities to intervene in deviations in order to demand repair [65]. STUK further concluded that time and resources needed for detailed design of Olkiluoto 3 were clearly underestimated, which created extra project challenges. The large number of subcontractors without previous experience in NPP construction was identified as a challenge. Chaining of contracts created a situation where the manufacturer of the containment steel liner may not have been aware of quality and construction supervision requirements early enough [65]. In 2007, STUK performed postwelding inspections in Olkiluoto 3, revealing problems in weld quality and deformations in the containment steel liner [66]. Figure 22 shows an example of observed defective deformations. Corrective action included the removal and replacement of defective liner sections. These quality deviations emphasize the necessity of third party quality control during NPP construction.

4.2.8.3. Construction technologies

Refer to IAEA Nuclear Energy Series document NP-T-2.5, Construction Technologies for Nuclear Power Plants [39] for various methods of containment construction including jump forming and slip forming.

4.2.8.4. Construction documentation

All design changes implemented during construction need to be documented in as-built drawings to aid ageing management activities, enable condition assessments and facilitate modifications and repairs. Construction history dockets need to be created and maintained over the plant life (including decommissioning) and need to include:

FIG. 22. Problems revealed by weld inspections of containment steel liner during Olkiluoto 3 construction (courtesy of STUK).

— Materials and construction specifications and procedures;
— Design changes;
— Records of inspection and testing of materials and components (including concrete batching records);
— Records of environmental conditions during construction;
— Construction sequence;
— Any issues during construction and their resolution;
— As-built drawings.

4.3. MODELLING AND ANALYSIS

4.3.1. Modelling and analysis tools

Models (e.g. for service life prediction) can be used to supplement materials databases for predicting structure response to ageing mechanisms. With sufficient data and an understanding of degradation processes, generic models may be derived that can be used for quantitative structural assessments. Data sources include laboratory tests and experience with both nuclear and non-nuclear structures. Data are often reported in journals and at technical conferences.

The use of known structural material properties (obtained via testing and inspections) with finite element structural models in combination with non-destructive testing (NDT) methods can provide an improved approach to monitoring concrete structure condition. The finite element model can indicate areas of high tensile stress or areas where surveillance or NDT would be focussed. Figure 23 shows an example of such a model. NDT results are fed back into the model to further refine it.

A consortium initiated by the ACI has developed a concrete service life prediction model called Life-365 [67]. The Life-365 software models and compares a number of corrosion protection strategies and estimates life cycle costs based on ASTM E917-05, Standard Practice for Measuring Life Cycle Costs of Buildings and Building Systems [68]. A guide to service life predication is also available [69].

4.3.2. Time limited ageing analysis tools

A time limited ageing analysis (TLAA) is an engineering analysis of the suitability of a safety related structure or component that is developed on the basis of an explicitly stated length of plant life. These analyses would typically be done as part of original plant design, or may be developed later (e.g. as part of a periodic safety

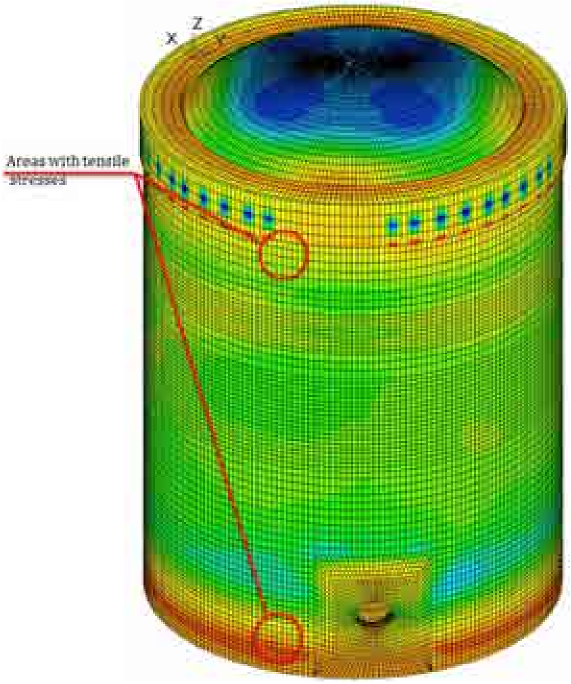


FIG. 23. Finite element model showing areas with high tensile stress at a maximum temperature gradient (courtesy of CMSLM Ltd).

review). These need to be revalidated as required as new operating history is obtained, or whenever original service life assumptions are revisited, for example, in preparation of plant life extensions/long term operation.

For licence extensions, TLAAs typically must:

— Be verified to bound the renewal period;
— Be reanalysed (recalculated) to determine if they will bound the renewal period;
— Show that ageing effects encompassed by the calculation will be adequately managed.

Methodologies for such analyses are documented in Ref. [70]. Some examples of TLAAs performed by NPPs relating to concrete structures include:

(a) Concrete containment tendon prestress analysis [71]

Prestressing tendons are used to impart compressive forces in prestressed concrete containment to resist internal pressure inside the containment that would be generated in the event of a LOCA. Prestressing forces generated by tendons diminish over time, primarily due to relaxation of prestressing material and concrete shrinkage and creep. The concrete containment tendon prestress TLAA ageing management programme provides reasonable assurance of the adequacy of prestressing forces in prestressed concrete containment tendons during the period of extended operation. The programme consists of an assessment of inspections performed in accordance with the requirements of Section XI, Subsection IWL of the American Society of Mechanical Engineers (ASME) Code. The assessment related to the adequacy of the prestressing force establishes acceptance criteria in accordance with NRC Regulatory Guide 1.35.1 [72], and trend lines based on the guidance provided in NRC Information Notice 99-10 [73]. The estimated and measured prestressing forces are plotted against time, and the predicted lower limit, minimum required value and trending lines are developed for the period of extended operation. Regulatory Guide 1.35.1 provides guidance for calculating the predicted lower limit and minimum required value. The trend line represents the trend of prestressing forces based on the actual measured forces. The prestressing force is acceptable when the trend line is above the minimum required value throughout the period of extended operation. If acceptance criteria are not met, then either systematic retensioning of tendons or a reanalysis of the containment is warranted to ensure design adequacy of the containment.

A similar TLAA analysis is defined as part of the IAEA Programme on International Generic Ageing Lessons Learned (IGALL) (TLAA 301) [74].

(b) Containment liner plate, metal containment and penetration cumulative fatigue analysis [71]

Fatigue of liner plates, metal containment and penetrations may be considered in the design based on an assumed number of loading cycles. Cyclic loads include, for example, RB interior temperature variation during heat-up and cooldown, outdoor temperature variations, leak tests, high energy penetration piping lines and seismic loads. This equipment may be required by its code of record to have a fatigue analysis and if so, then this analysis may be a TLAA and is to be evaluated to ensure that the effects of ageing on the intended functions are adequately managed for the period of extended operation. The ASME Section III fatigue analysis requires the calculation of a cumulative usage factor based on the fatigue properties of the materials and the expected fatigue service of the component. The ASME Code limits the cumulative usage factor to a value less than or equal to one for acceptable design fatigue. Three acceptance criteria for fatigue of containment liner plates are available:

(i) Existing calculations remain valid because the number of assumed cyclic loads will not be exceeded during the period of extended operation;
(ii) Cumulative usage factor calculations are re-evaluated based on an increased number of assumed cyclic loads to cover the period of extended operation and the new cumulative usage factor does not exceed one;
(iii) The AMP provided by the applicant demonstrates that the effects of ageing on the component's intended functions will be adequately managed during the period of extended operation.

A similar TLAA analysis is defined as part of the IGALL initiative (TLAA 303) [75].

(c) CANDU/PHWR concrete strength reduction due to creep and shrinkage [76]

IGALL TLAA 302 is defined as follows:

Time dependent phenomena such as creep and shrinkage of concrete induce additional strains in the concrete structural elements. These secondary strains are generally accounted for in design in terms of reduced strength and modulus of concrete available for resisting the primary loads. The creep strain and shrinkage strain of concrete depend on several aspects and site specific data. The common factors affecting both creep strain and shrinkage strain are the age of concrete, type of cement used, compressive strength of concrete, relative humidity, ambient temperature and the ratio of structural element cross-section to the element surface area exposed to atmosphere.

Creep is the increase in strain under sustained stress. To take the effect of creep in design, the value of modulus of elasticity of concrete is modified using a creep coefficient. Besides the common factors stated above, the creep coefficient also depends on mean compressive stress due to sustained loading, age of concrete at loading and short term modulus of elasticity of concrete. Effect of shrinkage is defined in terms of a shrinkage strain value which also involves the additional factor of the age of concrete at the beginning of shrinkage, that is, the concrete curing period.

TLAA for concrete strength reduction due to creep and shrinkage is required to ensure the acceptable level of margin in concrete strength to resist all induced stresses during long term operation. The time dependent parameters for the evaluation of the reduction in concrete strength are the induced strains due to creep and shrinkage respectively. The analysis parameters are the corresponding stress values.

(d) Cumulative fatigue damage of containment liners and penetrations [75]

IGALL TLAA 303 is defined as follows:

Fatigue analyses are considered in the design of containment liner plate, metal containment and penetrations using the cumulative usage factor (CUF). This is done to assess the likelihood of initiating a fatigue crack, and this analysis is consistent with national Member State requirements, codes and standards. Because the evaluation of CUF considers the number of thermal and pressure cyclic transients imposed on the analysed component, CUF evaluations are typically TLAAs that consider the number of cycles for the analysed operating time of the component.

For CUF evaluations, the time dependent parameter is usually the number of transient cycles, and the analysis parameter is the CUF value. For a typical CUF evaluation, multiple transients are considered in the analysis, and the number of cycles that apply to each transient would represent the individual time dependent parameters to be evaluated.

(e) Foundation settlement due to soil movement [77]

IGALL TLAA 304 is defined as follows:

Ground movement is a phenomena that occurs around structures and systems of some NPPs.

In general, during construction and the early years, structures undergo ground settlement. Since most NPP foundations rest on firm rock or other non-cohesive material, settlement occurs in the earliest months after construction and tends to disappear or reduce substantially thereafter. The AMPs for in-service inspection for concrete containment, structures monitoring and water control structures (AMP 302, 306 and 307, respectively) can be used effectively to manage ageing of such structures due to settlement. However, for NPP structures that are founded on clay and silty soils or on structural weathered rock, actual settlement may not match the predicted values due to a lack of adequate knowledge of the foundation material properties.

At other NPPs, a phenomenon of continued elevation or up-heave of the terrain has been detected. This phenomenon of ground movement occurs due to the swelling of marl (a calcium carbonate or lime rich mud or mudstone that contains between 35–65% clays and silt) in contact with water. Excavation of the overburden during construction and changes in the soil (i.e. groundwater level) leads to ground movements throughout the life of the NPP.

For ground movement, the time dependent parameter is the expected movement considered in the design. It needs to not affect or influence the intended functions of the SSC.

(f) In-service local metal containment corrosion analysis [71]

Corrosion in inaccessible areas may be of concern, and may necessitate additional analysis. Examples of inaccessible areas are below grade exterior walls and foundations with below grade aggressive environments that might impact embedded steel or concrete.

(g) SFP liner fatigue analysis [78, 79]

Liner plates may be attached to concrete containment walls by stud anchors, structural rolled shapes or by other means. Design processes sometimes have assumed that liner plates do not carry load, however anchorage systems transfer normal loads, such as those from concrete shrinkage, creep and thermal changes that are imposed on containment structures to the liner plates. Internal pressure and temperature loads are also directly applied to the plates. Thus, under design basis conditions, liner plates can experience significant strain. The design may consider liner plate fatigue based on an assumed number of loading cycles. Cyclic loads include:

— Containment building interior temperature variation during heat-up and cooldown;
— LOCAs;
— Outdoor temperature variations;
— Thermal loads because of high-energy containment;
— Penetration piping lines (e.g. steam and feedwater lines);
— Seismic loads;
— Pressurization from periodic integrated leak rate tests.

If a plant's code of record requires a fatigue analysis, then this analysis may be a TLAA and needs to be evaluated to ensure that ageing effects will be adequately managed for the period of extended operation.

4.4. STRESSORS AND OPERATING CONDITIONS

Stressors and operating conditions related to concrete, mild reinforcing steel, post-tensioning systems and liners can impact structure degradation performance. Sample stressors include:

— Percolation of fluid, flowing gases or liquids and potential abrasives;
— Exposure to alkalis, sulphates, acids, bases and dissolved salts;
— Cyclic loads and vibration;
— Long term loading;
— Impact loading;
— Thermal cycles;
— High or low temperatures;
— Irradiation;
— Foundation consolidation/movement;
— Corrosion due to chloride ions;
— Loss of material due to corrosion;
— Electrochemical reactions.

It is important to consider these stressors and operating conditions during the NPP design phase so they do not lead to subsequent unacceptable degradation. Stressors may not be explicitly defined in codes and standards and may need to be addressed in design/technical specifications.

During operation, it is important to monitor operating conditions to ensure that design assumptions regarding applicable stressors are valid.

4.5. AGEING MECHANISMS

4.5.1. Background

Concrete structure degradation is a function of many factors, including constituent materials, location (e.g. coastal or inland), climatic conditions (e.g. temperature and moisture) and the presence of external agents (e.g. aggressive ionic species) [9]. More detailed information to that summarized below is available in Ref. [10] and Refs [80–83].

4.5.2. Concrete material

Concrete material durability is negatively impacted when its cement–paste matrix or aggregate constituents come under attack. The attack processes on both may occur concurrently and reinforce each other. Transport mechanisms for gases, ions and water within concrete pores and cracks dominate the physical and chemical processes that influence concrete durability. Transport characteristics do not provide information on rates or the extent of reactions or total amounts of substances reacting with aggressive material; they only provide an indication of a material’s durability [84].

Transport mechanisms important for concrete durability include [9]:

— Diffusion of gases, CO_2, O_2 and water vapour through empty pockets, microcracks and component interfaces;
— Diffusion of ions (e.g. chlorides or sulphates) in concrete pore solution and dissolved gases;
— Permeation of water or aqueous solutions under hydraulic head (submerged concrete or water control structures) [85];
— Capillary suction of water (water absorption) or aqueous solutions in empty or unsaturated capillaries [84].

Table 4 provides an indication of the influence of moisture (relative humidity) on several deterioration processes in concrete [86].

TABLE 4. INFLUENCE OF MOISTURE STATE ON SELECTED DURABILITY PROCESSES (*adapted from Ref.* [87])

	Relative severity of the deterioration process[a]				
	Carbonation of concrete	Frost attack of concrete	Chemical attack of concrete	Risk of steel corrosion	
				In carbonated concrete	In chloride-rich concrete
Very low (<40%)	1	0	0	0	0[b]
Low (40–60%)	3[c]	0	0	1	1
Medium (60–80%)	2[d]	0	0	3	3
High (80–98%)	1	2	1	2	3
Saturated (>98%)	0	3	3	1	1

[a] 0 = insignificant, 1 = slight risk, 2 = medium risk, 3 = high risk.
[b] Corrosion risk in a chloride-rich environment is high if there are significant humidity variations.
[c] For 40–50% relative humidity, carbonation is medium.
[d] For 60–70 % relative humidity, carbonation is high.

Cracking occurs in virtually all concrete structures and, because of concrete's inherently low tensile strength and lack of ductility, can never be totally eliminated. Cracks and crack patterns have different characteristics depending on underlying causes. Cracks are significant because of the following reasons [9]:

— They can indicate major structural problems (active cracks).
— They can provide an avenue for ingress of hostile environments (active or dormant cracks).
— They may inhibit a component from meeting performance requirements (active or dormant cracks) (e.g. diminished leaktightness or shielding capacity).

Figure 24(a) provides information on types of cracks that can form in concrete structures [86] and Fig. 24(b) provides a description and appearance of several crack forms [88]. Figure 25 presents examples of the most common types of intrinsic concrete cracks, as well as an indication of the potential time of occurrence [86, 89].

4.5.2.1. Physical attack

Physical attack involves concrete degradation due to external influences and generally involves cracking due to exceeding the concrete tensile strength, or loss of surface material. Load induced cracking is not considered as an ageing mechanism, but may provide access to the concrete interior, enabling deleterious mechanisms to operate (e.g. chlorides).

(a) Salt crystallization

Salt solutions (e.g. NaCl, $CaSO_4$ and $NaSO_4$) can move through concrete by capillary action and when they dry and crystallize, their expansive forces can cause damage, often repeatedly via wetting and drying cycles. The mechanism is similar to water freezing and thawing. Figure 26 presents a concrete slab after one year of exposure to cyclic wetting and drying in sulphate solutions [90].

Structures in contact with fluctuating water levels or in contact with groundwater containing large quantities of dissolved salts are susceptible to this deterioration. Above ground moisture is drawn to the concrete surface where it evaporates, leaving salt crystals growing near surface pores. The result is a deteriorated area just above ground level. Salt crystallization problems are minimized with low permeability concretes and where sealers or barriers have been applied to prevent water ingress or subsequent evaporation [9].

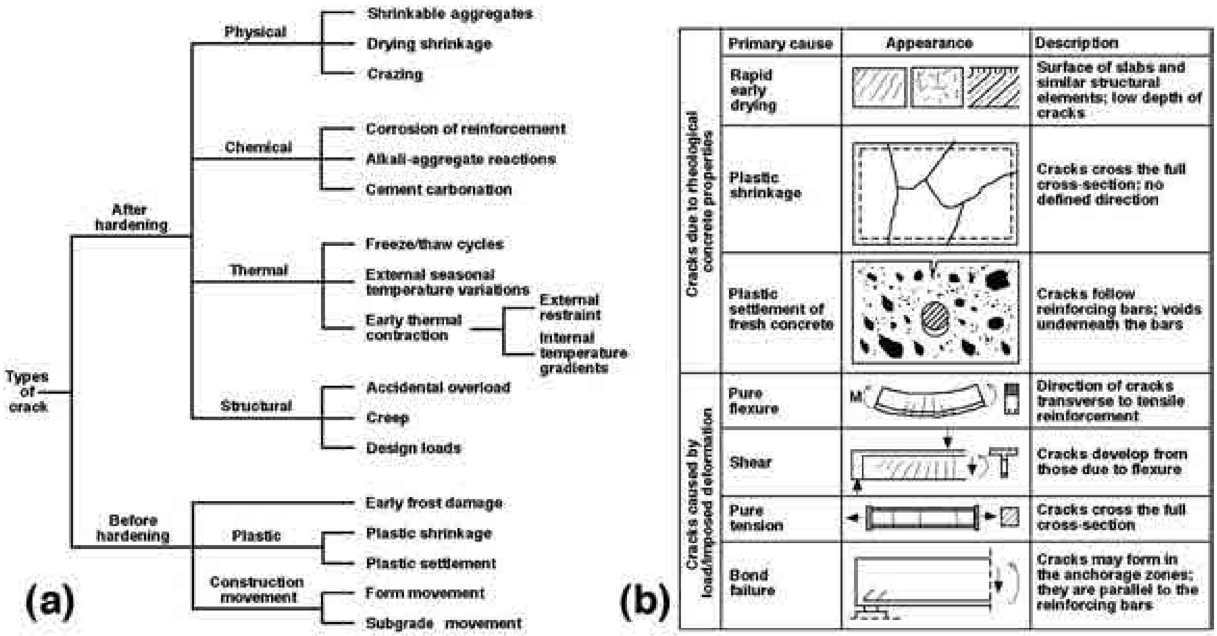


FIG. 24. Relationship between primary causes and types of cracks in concrete (adapted from Refs [88, 89]).

Type of Cracking	Designation	Time of Occurrence
Plastic settlement	A, B, C	Ten minutes to three hours
Plastic shrinkage	D, E, F	Thirty minutes to six hours
Early thermal contraction	G, H	One day to two to three weeks
Long-term drying shrinkage	I	Several weeks or months
Crazing	J, K	One to seven days - sometimes much later
Corrosion of reinforcement	L, M	Several years, but may be sooner
Alkali-aggregate reaction	N	More than five years

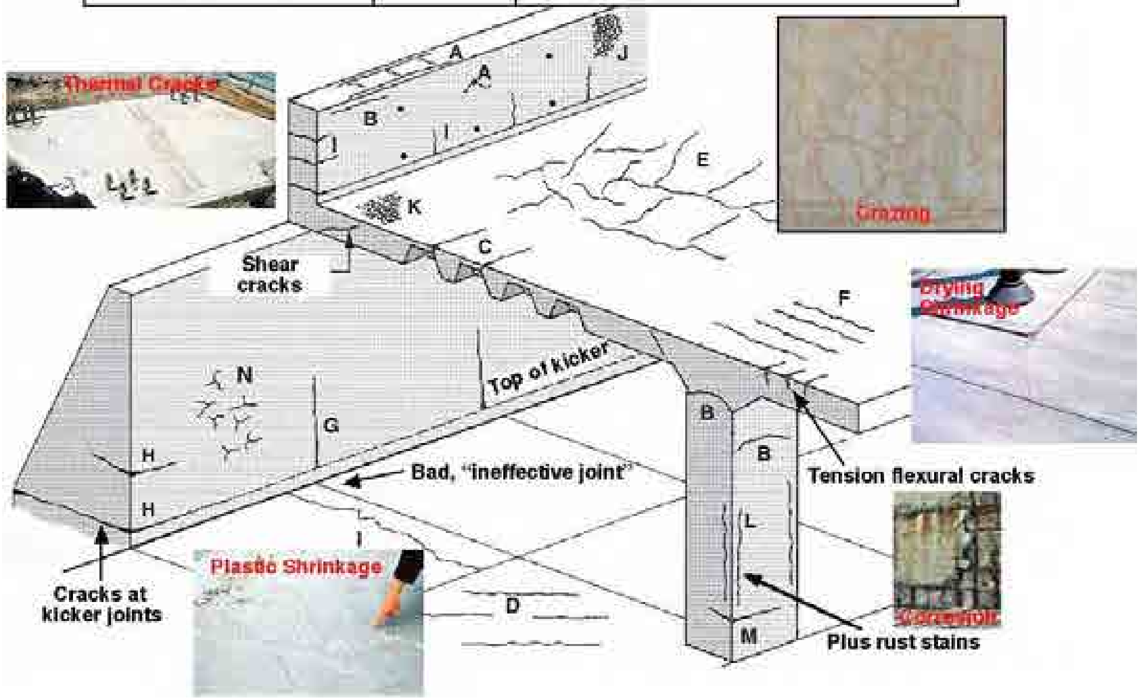


FIG. 25. Examples of intrinsic cracks in a hypothetical structure (adapted from Ref. [89]).

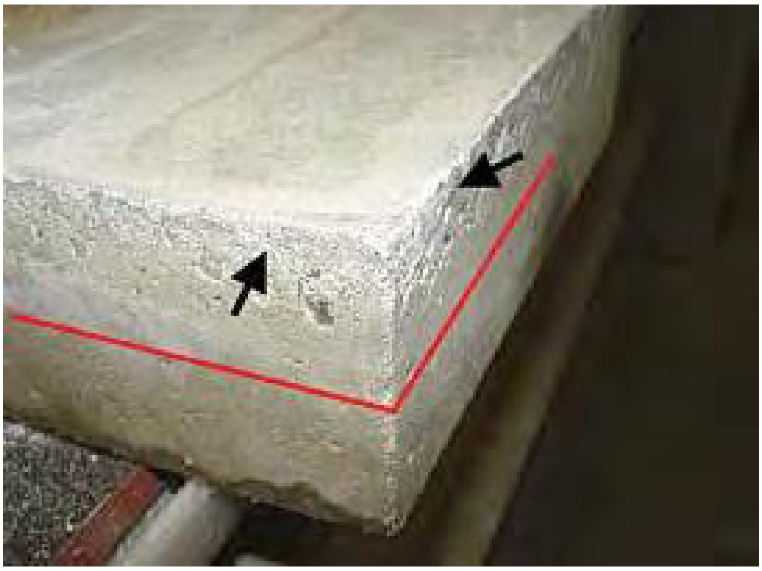


FIG. 26. Concrete slab experiencing deterioration due to salt crystallization; arrows indicate concrete surface deterioration and lines indicate the level of solution in wetting cycle [90].

(b) Freezing and thawing attack

Concrete paste and aggregate can be susceptible to damage during freeze-thaw cycles produced by the natural environment or industrial processes. Structures constructed without adequate air entrainment (i.e. size and spacing of air bubbles, as opposed to entrapped air) and with portions of structures where moisture can accumulate, are at the greatest risk. Other factors include water to cement ratios, aggregate selection, curing practices, concrete strength and degree of saturation.

Damage occurs after a number of freeze-thaw cycles and is observed on exposed structure surfaces. One hypothesis is that damage is caused by hydraulic pressure generated in the cement paste capillary cavities, while critically saturated, as water freezes and expands by about 9%. When pressure exceeds the concrete tensile strength, the cavity dilates and ruptures [9].

Damage to concrete resulting from freeze and thaw attack can include:

— Expansion;
— Internal cracking;
— Spalling;
— Scaling (associated with salt application);
— Pop-outs [91, 92].

Scaling is concrete surface delamination. Weaknesses may exist at surfaces due to excessive water, excess mortar or mistreatment during construction. Surface layers may detach if stresses exceed the tensile or bond strength holding the layer to the substrate. Scaling may develop from a shallow surface feature into internal damage. This often is associated with the application of de-icing chemicals that cause concrete surface temperatures to change rapidly, thereby inducing a thermal shock that can cause cracking and scaling. Also, if salts are present in the pore solution, osmotic pressure is increased since moisture tends to move toward zones of higher salt concentrations [9].

A pop-out is a small volume of concrete that has separated from the concrete body leaving a roughly conical depression. The most common cause of pop-outs is stress resulting from the freeze-thaw action within a coarse aggregate particle, causing particle cracking and concrete fracture between the particle and the nearest concrete face. Aggregates producing pop-outs are generally sedimentary (e.g. cherts, sandstones, shales and limestones), but can be calcareous or siliceous gravel or crushed rock with high porosity [93]. Internal damage that occurs by cracking is primarily confined to mortar and is associated with the freeze-thaw damage of young concrete or mature cement paste, which do not have the pore structures necessary to resist developed stresses [9]. Figure 27 presents types of freeze-thaw damage [91].

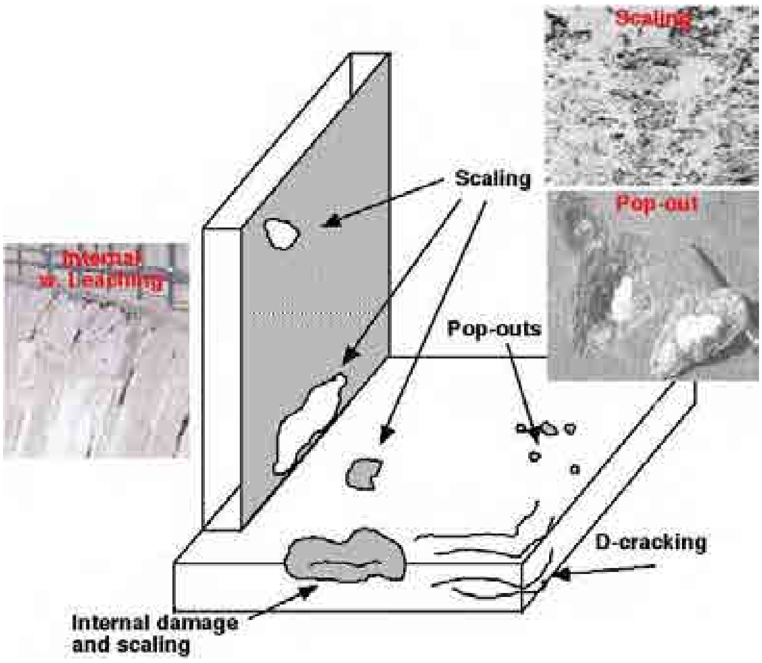


FIG. 27. Types of freeze-thaw damage [91].

Guidelines to evaluate if concrete was produced to provide resistance to freeze-thaw attack are available in Refs [94–96].

(c) Abrasion/erosion/cavitation

Loss of material at concrete surfaces can progressively occur due to abrasion, erosion or cavitation. Abrasion is dry attrition due to rubbing or grinding of aggregate or other debris on the surface, while erosion is abrasive wear by the action of fluids containing suspended particles, for example, water-transported silt, sand, gravel, ice or debris. Mechanical abrasion is usually characterized by long shallow grooves and spalling along monolithic joints. Concrete surfaces eroded by water-borne debris are generally smooth and may contain localized depressions.

Cavitation is a particular form of erosion caused by the implosion of gas bubbles on a surface. It is often associated with sudden variations in pressure related to the hydrodynamic parameters of a fluid. In hydraulic structures, the liquid is water and the cavities are filled with water vapour and air. Cavities form where local pressure drops to a value that causes water to vaporize at the prevailing temperature. The formation of cavities is usually triggered by concrete surface irregularities that are subjected to high velocity water flow. Cavitation bubbles grow and travel with flowing water to an area where the pressure field causes collapse. When a bubble collapses or implodes close to or against a solid surface, extremely high pressure is generated, which acts on an infinitesimally small surface area for a very short time. A succession of these high-energy impacts will damage almost any solid material [97].

Structures that are susceptible to degradation due to abrasion and cavitation are those that are exposed to running water, such as cooling towers, intake structures and spray ponds. Figure 28 presents examples of concrete abrasion-erosion [97, 98].

Concrete resistance to abrasion and erosion depends on concrete quality (low porosity, high strength) and, in particular, the aggregate particles used in the mix. While good quality concrete may show resistance to abrasion and erosion, it may still suffer severe loss of surface material due to cavitation. The best way to guard against cavitation effects is to eliminate their causes [9].

(d) Thermal exposure/thermal cycling

Portland cement paste experiences physical and chemical changes that contribute to shrinkage, transient creep and changes in strength when subjected to elevated temperatures. Key material changes brought about include [9]:

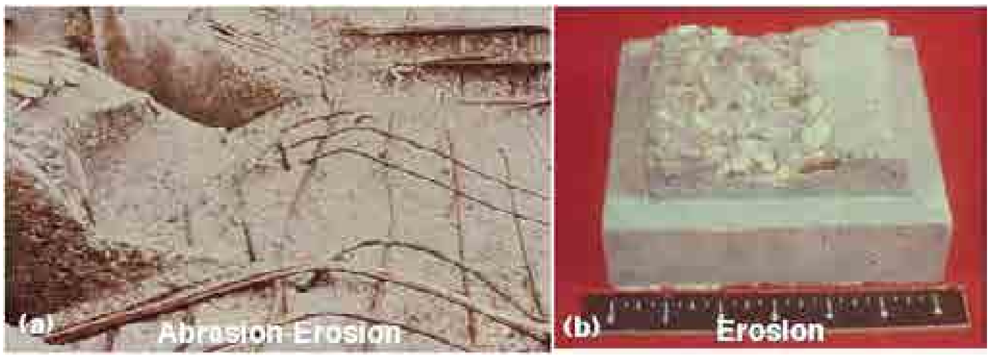


FIG. 28. Abrasion-erosion of concrete: (a) abrasion-erosion damage in a concrete stilling basin [98]; (b) erosion of conventional concrete [97].

— Moisture state (i.e. sealed or unsealed);
— Chemical structure (i.e. loss of chemically bound water from calcium silicate hydrates in the unsealed condition, CaO to SiO_2 ratio of the hydrate in the sealed condition and amount of $Ca(OH)_2$ crystals in sealed or unsealed conditions);
— Physical structure (i.e. total pore volume including cracks, average pore size and amorphous/crystalline structure of solid) [99].

Concrete at elevated temperature is sensitive to temperature level, heating rate, thermal cycling and temperature duration (as long as chemical and physical transformations occur).

Changes in mechanical properties and durability occur at high temperatures. Non-linearities in material properties, variation of mechanical and physical properties with temperature, tensile cracking and creep effects affect buildup of thermal forces, load carrying capacity and deformation capability (i.e. ductility) of concrete structural members. Property variations are due to changes in moisture conditions of concrete constituents and progressive deterioration of cement paste–aggregate bonds, which is especially critical where thermal expansion values for cement paste and aggregate differ significantly. The bond region is affected by aggregate surface roughness and its chemical and physical interactions [99]. Chemical interaction relates to chemical reactions between aggregate and cement paste that can be either beneficial or detrimental. Physical interaction relates to dimensional compatibility between aggregate materials and cement paste [9].

Concrete in the temperature range of 20–200°C can show small strength losses. Between 22–120°C, any strength loss that occurs is attributed to thermal swelling of physically bound water causing disjoint pressures. Regained strength is often observed between 120–300°C and is attributed to greater van der Waals forces as a result of cement gel layers moving closer to each other during heating. Between 200–250°C, residual compressive strength is nearly constant. Beyond 350°C there can be a rapid decrease in strength. Figure 29 illustrates the effects of elevated temperature on compressive strength for several unsealed ordinary concretes. These were made with various normal weight aggregate materials and tested at room temperature after heat treatment [100]. Some conclusions relative to the impact of elevated temperature on concrete compressive strength are that the presence of preload improves strength retention, the strength of unsealed concrete is higher than sealed (mass) concrete and specimens heated and then permitted to cool to room temperature lose more strength than those tested while hot. Codes and standards such as the ASME Boiler and Pressure Vessel Code, British Standard (BS 4975) [101] and Canadian Standards Association (CSA) N287.3 [102] recognize that concrete compressive strength tends to decrease with increasing temperature and these codes and standards specify concrete temperature limits to ensure predictable concrete behaviour.

Thermal exposure can also result in cracking and, when heating rate is high and concrete permeability low, surface spalling. Elevated temperatures diminish bonds between concrete and rebar [103–105]. Elevated temperatures also affect volume change and concrete creep [106].

Thermal cycling, even at relatively low temperatures (i.e. 65°C), can negatively impact concrete mechanical properties (i.e. compressive, tensile and bond strengths and modulus of elasticity are reduced) [106]. Daily thermal cycling due to ambient environmental temperature changes is accounted for in designs (e.g. by adding rebar concrete). At higher temperatures (200–300°C), the first thermal cycle causes the largest percentage of damage,

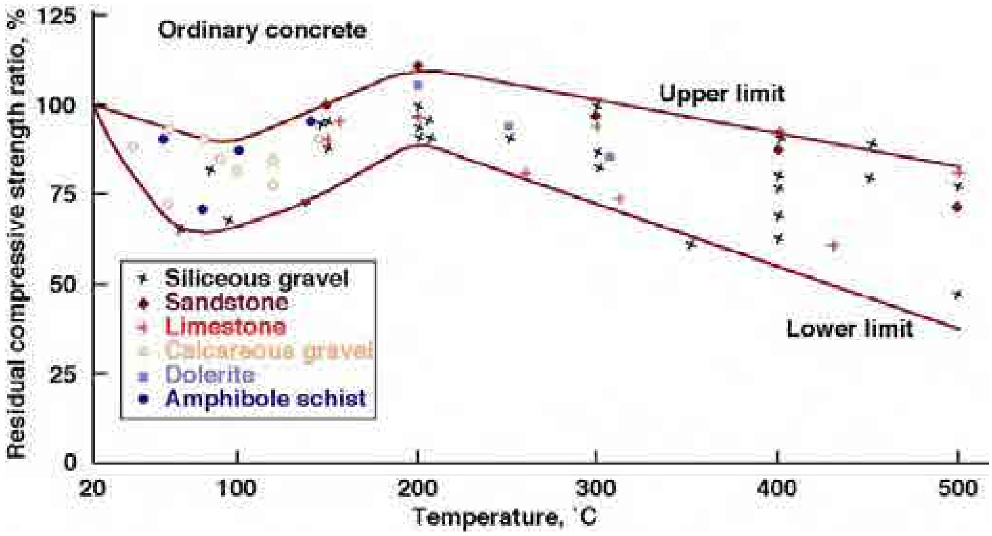


FIG. 29. Effect of temperature on residual compressive strength: unsealed specimens [100].

with damage extent markedly dependent on aggregate type and associated with loss of bond between aggregate and matrix [107]. Thermal cycles also can become important if structure deformation resulting from temperature variations is constrained.

(e) Irradiation

Irradiation can affect concrete. Typical sources are fast or thermal neutrons emitted by the reactor core or gamma rays produced as a result of members (particularly steel) capturing neutrons in contact with concrete.

Table 5 provides a summary of effects of gamma and neutron irradiation on cement paste, aggregate and concrete materials. Changes in concrete properties appear to depend primarily on the aggregate used, which can change in volume when exposed to radiation [108]. Fast neutrons are mainly responsible for expansion caused by atomic displacements in certain aggregates (e.g. flint). Quartz aggregates containing crystals with covalent bonding should be more greatly affected by radiation than calcareous aggregates containing crystals with ionic bonding [109]. When nuclear radiation is attenuated or absorbed in the concrete, almost all absorbed radiation is converted into heat. Nuclear heating occurs as a result of energy, introduced as neutrons or gamma radiation, interacting with concrete molecules. Heat generated may have detrimental effects on concrete physical, mechanical and nuclear properties. Reference [110] indicates that nuclear heating is negligible for incident energy fluxes less than 10^{10} MeV·cm^{-2}·s^{-1}. Determining whether concrete deterioration is due to radiation damage or thermal effects can be difficult. Prolonged exposure of concrete to irradiation can result in decreases in tensile and compressive strengths and modulus of elasticity. Literature [9, 108] indicates the following:

— For some concretes, neutron radiation of more than 1×10^{19} n/cm^2 or 10^8 Gy of dose for gamma radiation may cause a reduction in compressive strength.
— Concrete tensile strength is significantly reduced at neutron fluences exceeding 1×10^{19} n/cm^2 with the decrease of tensile strength caused by neutron radiation more pronounced than the decrease of compressive strength.
— Resistance of concrete to neutron radiation depends on the type of neutrons (slow or fast) involved, but the effect is not clear.
— Resistance of concrete to neutron radiation depends on mix proportions, type of cement and type of aggregate.
— Effect of gamma radiation on concrete's mechanical properties requires clarification.
— Deterioration of concrete properties associated with a temperature rise resulting from irradiation is relatively minor.
— Irradiated concrete's coefficients of thermal expansion and conductivity differ little from those of temperature exposed concrete.
— When exposed to neutron irradiation, concrete's modulus of elasticity decreases with increasing neutron fluence.

— Concrete creep is not affected by low level radiation exposure but, for high levels of exposure, creep probably would increase because of the effects of irradiation on tensile and compressive strength. Gamma rays produce radiolysis of water in cement paste that can affect concrete's creep and shrinkage behaviour to a limited extent and also result in the evolution of gas.
— For some concretes, neutron radiation with a fluence of more than 1×10^{19} n/cm^2 can cause a marked increase in volume.
— Generally, concrete's irradiation resistance increases as irradiation resistance of aggregate increases.
— Irradiation has little effect on the shielding properties of concrete beyond moisture loss caused by a temperature increase. Furthermore, there is an indication that nuclear radiation can significantly increase the reactivity of silica rich aggregates to alkali (i.e. alkali–silica reaction) [111].

TABLE 5. EFFECT OF GAMMA RAYS AND NEUTRONS ON CONCRETE AND ITS COMPONENTS (*adapted from Ref.* [112])

Component	Gamma rays	Neutrons
Cement paste	Water Water is decomposed by radiolysis to generate hydrogen and hydrogen peroxide, which in turn decompose into water and oxygen Gamma heating causes additional hydration of unhydrated cement and phase change of hydrates Gamma heating causes severe drying in cement paste	Water Molecular products from water may be the same as those for gamma rays, but the yields are different due to the difference of linear energy transfer
	Solids Si–O bond of calcium silicate hydrates may be slightly decomposed due to the covalent nature of the bond Electrons are ejected by scattering of gamma rays and collision with the cement solid phase materials	Solids Dislocation of atoms in the solid phase of cement paste occurs; the lattice defect will not accumulate due to its original crystalline imperfections
Aggregate	Water Small amounts of water will be released by radiolysis and gamma heating	Water Same effect as that of cement paste
	Solids Siliceous aggregate may be slightly decomposed	Solids Lattice constants are increased due to the dislocation of atoms and lattice defects are accumulated
Concrete	Drying causes large shrinkage, loss of stiffness and strength changes of cement paste resulting in cracks around aggregate and deterioration of stiffness and strength of concrete Additional reaction of unhydrated cement may increase concrete strength Hydrogen peroxide generated in radiolysis process may react with cement paste and may affect concrete strength	Increase of lattice constant and accumulation of defects in aggregate causes expansion, which produces cracking around aggregate and deterioration of stiffness and strength of concrete Siliceous aggregate may show larger expansion than non-siliceous aggregate

Vodak et al. [113] conducted an experimental study on the effect of gamma radiation on the strength of concrete samples representative of Temelin NPP concrete. Concrete specimens were exposed to various levels of gamma irradiation. Results indicated that the compressive strength of concrete exposed to a gamma radiation of 5×10^5 Gy was reduced by about 10% compared to unirradiated samples. In fact, the strength reduction started from 3×10^5 Gy. As stated by the authors, their results were not consistent with others in that the compressive strength showed reductions at exposure levels below the commonly accepted threshold reference value of 2×10^8 Gy. One possible cause for the reduction at lower exposure levels was that no method of shielding neutron radiation was mentioned and, therefore, the reduction in strength may be due to the combined effect of neutron and gamma radiation instead of only gamma radiation. Soo and Milian [114] also found that loss of compressive strength of Portland cement mortar could occur at gamma doses less than the threshold reference value. It was postulated that the loss of strength could be connected with radiolysis of water in the cement as well as pore water. If hydrogen and oxygen radiolytic species are lost during irradiation, this would reduce the level of cement hydration and strength.

It should be noted that the specific behaviour of concrete under long term irradiation, typical of an NPP, is relatively unknown and is the subject of research [112]. Information on the interaction of irradiation with concrete is currently insufficient to adequately determine the impact on concrete properties and performance for service life beyond 60 years of operation. One obstacle to understanding irradiation effects is the lack of available data from which to draw statistically relevant conclusions. Studies are currently funded by Japanese research organizations [112], the Electric Power Research Institute [115] and the US Department of Energy [116] to determine neutron and gamma exposure levels of structures representative of plant service lives of 60 years, 80 years, and beyond 80 years, and to determine if these levels are sufficient to impact performance of pertinent structures (e.g. biological shield of PWRs and reactor pressure vessel supports of BWRs).

(f) Fatigue/vibration

Concrete structures subjected to fluctuations in loading, temperature or moisture content can be damaged by fatigue. Concrete exhibits good resistance to fatigue, so fatigue failure is unusual and concrete structures are designed using codes limiting design stress levels to values below concrete's endurance limit [117, 118]. However, as structures age, there may be instances of local fatigue damage at locations where reciprocating equipment is attached, or at supports for pipes exhibiting flow induced vibration.

Fatigue damage initiates as microcracks in the cement paste, close to large aggregate particles, reinforcing steel or stress risers (e.g. defects). With continued or reversed load application, these cracks may propagate to form structurally significant cracks, exposing the concrete and reinforcing steel or producing increased deflections. Ultimate failure by fatigue will occur as a result of excessive cracking, excessive deflections or brittle fracture. As concrete ages and gains strength, for a given stress level, the cycles to failure will increase. If concrete is reinforced or prestressed, steel properties will tend to control structural performance since the steel carries tensile loads.

(g) Settlement

Settlement in a structure refers to the following:

— Distortion or disruption of parts of a building due to unequal compression of its foundations;
— Shrinkage, such as that which occurs in timber-framed buildings as the frame adjusts its moisture content;
— Undue loads being applied to the building after its initial construction [119].

All structures have a tendency to settle during construction and early life. Settlement may be caused by errors in foundation design either due to incorrect assumptions about properties and distribution of soil and rock below the structure, or errors in structural design of elements such as pile caps [87]. Generally, most settlement occurs within a few months after construction and is negligible after this period [9].

Uniform settlement is quite normal and will not normally cause structural distress, although if excessive it can result in damage or misalignment of connecting services or structures. Differential settlement is more of a concern as it can cause misalignment of structures, equipment, piping, cabling and other components and lead to overstress conditions (e.g. cracking such as illustrated in a building structure in Fig. 30). Settlement amounts are dependent

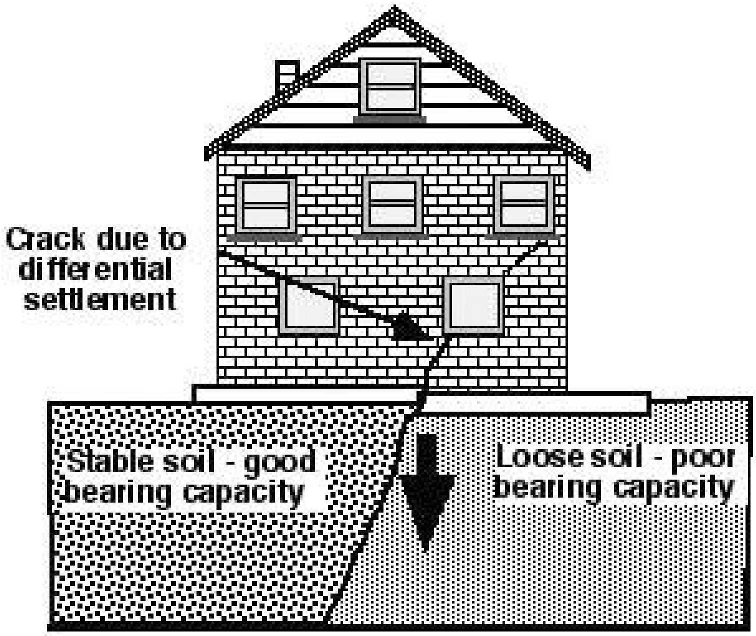


FIG. 30. Example of differential settlement cracking in a building structure due to inadequate foundation design [9].

upon physical properties of foundation material, which may range from bedrock (minimal settlement expected) to compacted soil (some settlement expected). Settlement is generally accounted for in structure designs and is not expected to be significant. Where there is some potential for significant settlement, monitoring programmes may be implemented to confirm that design allowances are not exceeded.

4.5.2.2. Chemical attack

Chemical attack on concrete is possible when the chemicals are in solution above critical concentrations (in their normal state or following evaporation), especially when such solutions are circulated. Concrete is rarely attacked by solid dry chemicals. The degree of attack depends upon the chemical involved, temperature of the aggressive solution, concrete water to cement ratio, type of cement used, degree of consolidation of the concrete, concrete permeability and degree of wetting and drying of the chemical on the concrete [9].

Chemical attack alters the concrete via a chemical reaction with its cement paste, coarse aggregate or rebar. Attack generally occurs on exposed surface regions (cover concrete), but can affect entire structural cross-sections in the presence of cracks or prolonged exposure. Corrosion of embedded steel reinforcement due to concrete carbonation or action of chloride ions is covered under the section addressing mild steel reinforcement (Section 4.5.4).

Chemical causes of deterioration can be any of the following:

— Hydrolysis of cement paste components by soft water;
— Cation exchange reactions between aggressive fluids and cement paste;
— Reactions leading to formation of expansion products [120].

Chemical attack of concrete may occur in several different forms, which are described on the following pages.

(a) Efflorescence and leaching

Efflorescence is a salt crystalline deposit on surfaces of masonry, stucco or concrete. It is whitish in appearance and is sometimes referred to as ‘whiskers’. It follows percolation of a fluid such as water through the material, either intermittently, continuously, or when an exposed surface is alternately wetted and dried. It may occur on a seasonal basis following the rainy season. Denser material is less prone to the effect.

Crystals of dissolved salts, usually calcium, sodium or potassium carbonate, form as a result of fluid evaporation or interaction with atmospheric carbon dioxide. Occasionally, efflorescence may be a symptom of chemical reactions such as sulphate attack, or may indicate leaks in water retaining structures or undesired leakage of moisture or groundwater through a structure [9]. To establish that damage has occurred, it is essential to demonstrate that deleterious reactions have occurred in the concrete interior or at surfaces that are in contact with sulphates in the surrounding soil [121]. Typically, efflorescence is primarily an aesthetic problem rather than a problem that affects mechanical properties or durability. In rare cases, excessive efflorescence deposits can occur within concrete surface pores, causing expansion that may disrupt the surface [122] (See Fig. 31).

FIG. 31. Efflorescence in a water retaining structure [123].

Leaching of cementitious materials involves the transport of ions from a material's interior, through its pore system, and outwards into the surroundings. Solid concrete compounds are dissolved by water and transported away. The transport is driven either by diffusion based on concentration gradients, or by convection through water flow. Water characteristics impact on the leaching process. Pure water containing little or no calcium ions, or acidic groundwater (containing dissolved carbon dioxide gas, carbonic acid or bicarbonate ion) tends to hydrolyse or dissolve alkali oxides and calcium containing products. Water with pH<12.5 destabilizes cementitious hydration products. Leaching rates depend on the amount of dissolved salts in the percolating fluid, the rate of permeation of fluid through the cement paste matrix and temperature. Extensive leaching causes increased porosity and permeability, which lowers concrete strength and makes the concrete vulnerable to hostile environments (e.g. water saturation, frost damage or chloride penetration and corrosion of embedded steel) [9]. Concrete leaching is of three types:

(a) Leaching at concrete free surfaces, which is generally of little importance;
(b) Leaching from concrete interior;
(c) Leaching at concrete crack surfaces, which is difficult to manage and complicated to assess [124].

Concretes produced using low water to cement ratios, adequate cement content and proper compaction and curing are most resistant to leaching. Examples of leaching in a concrete dam and a tendon gallery of an NPP are shown in Fig. 32.

FIG. 32. Examples of leaching: (a) a dam with freeze-thaw damage and leaching; (b) NPP tendon gallery leaching [9].

(b) Sulphate attack

Sulphates can damage concrete by causing excessive expansion, delamination, cracking and loss of strength. Attacks come from exposure to excessive amounts of sulphate from internal sources (soluble sulphate sources incorporated into concrete at the time of mixing, or naturally gypsum or pyrite being present in the aggregate or admixtures), or from external sources. Figure 33 illustrates an external sodium sulphate attack.

External attack sources include magnesium, sodium, calcium and potassium sulphates present in soils, groundwater and sea water. Anaerobic bacteria in sewers can also produce sulphur dioxide which dissolves in water and then oxidizes to form sulphuric acid. Dissolved sulphates can penetrate concrete, react with calcium hydroxide, and if enough water is present, cause expansion and irregular concrete cracking. The degree of attack depends on water penetration, sulphate salt concentration and type, the means by which salt develops (e.g. crystallization caused by rising and drying) and concrete binder chemistry [9]. Concentrations of about 0.2% sulphate content in groundwater may be enough to cause sulphate attack. Solutions containing magnesium sulphate are generally more aggressive, for the same concentration [125].

Sulphate attack of hardened concrete generally appears either as the expansive formation of ettringite and gypsum (calcium sulphate) in the concrete, or as softened and dissolved cementing compounds due to direct attack on these compounds by sulphate or by their decomposition when calcium hydroxide reacts with sulphates and is removed [53]. Ettringite is a hydrous calcium aluminium sulphate mineral. It is naturally colourless to yellow in colour and was first found near the Ettringer Bellerberg volcano in Germany.

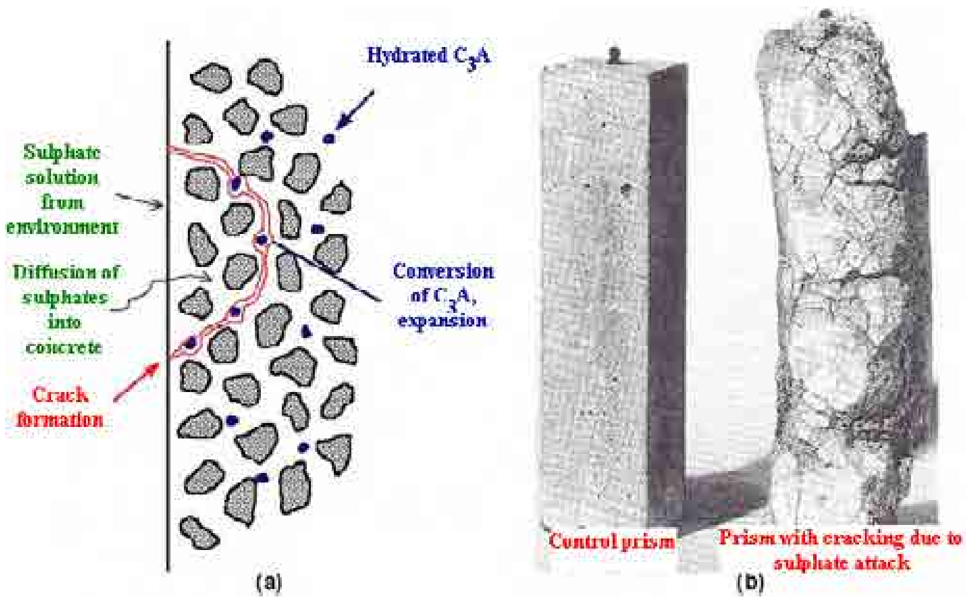
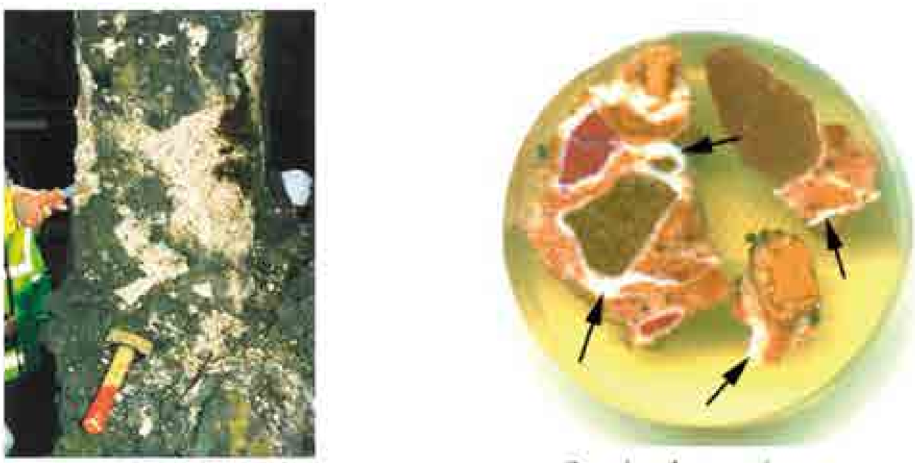


FIG. 33. Concrete cracking due to sodium sulphate attack: (a) mechanism (adapted from Ref. [86]); (b) cracking due to sulphate attack (adapted from Ref. [87]).

Vulnerable concrete structures include footings, foundation walls, retaining walls, piers, culverts, piles, pipes and surface slabs. The most severe attack occurs on elements where one side is exposed to sulphate solutions and evaporation can take place at the other [126]. Structures subjected to sea water are more resistant to sulphate attack because of the presence of chlorides that form chloro-aluminates to moderate the reaction. Concretes using cements low in tricalcium aluminate and those that are dense and of low permeability are most resistant to sulphate attack [9].

A rarer form of sulphate attack involves thaumasite formation due to reactions between calcium silicates in the cement, calcium carbonate from limestone aggregates or fillers and sulphates, usually from external sources [87]. Thaumasite ($CaSiO_3.CaCO_3.CaSO_4.15H_2O$) is a calcium-silicate-sulphate-carbonate mineral. Thaumasite sulphate attack in susceptible concrete typically requires sulphate sources, mobile groundwater, calcium silicate hydrate sources, the presence of carbonate and low temperatures (<10°C) [127]. The attack forms slowly and can destroy a significant part of the calcium silicate hydrate, resulting in a soft, white, pulpy mass that causes total disintegration of the concrete and exposed rebar. Serious damage to concrete or masonry due to thaumasite formation is uncommon. Figure 34 presents a subsurface concrete pier affected by thaumasite sulphate attack in the UK [9].



Subsurface concrete pier affected by thaumasite sulphate attack

Scanning electron microscope image of thaumasite formation.

FIG. 34. Thaumasite sulphate attack ([128] and [129], respectively).

(c) Delayed ettringite formation (DEF)

DEF is a special case of internal sulphate attack. It occurs when internal or external sulphates react with anhydrous or hydrated calcium aluminates and has an expansive character. Such expansion is not a concern in fresh concrete, but can induce cracking in hardened concrete and also increase the risk of secondary forms of deterioration such as freeze/thaw attack or reinforcement corrosion.

DEF is a result of high early temperatures (above 70°C) in concrete which prevents the normal formation of ettringite (a normal product of early cement hydration) or the decomposition of ettringite that has already formed.

Use of cements having high sulphate content, in which the sulphate has very low solubility, can also lead to DEF. In this case, sulphate concentrations in the pore liquid are high for an unusually long period of time in the hardened concrete. Eventually, sulphate will react with calcium- and aluminium-containing phases of the cement paste, expanding the paste and forming cracks around aggregate particles. In one case where this was reported, it was thought that the DEF was due to sulphate formed in the cement clinker being present as anhydrite and as a component of the silicate phases that are slowly soluble [130]. If structures susceptible to DEF are later exposed to water, ettringite can reform in the paste as a massive development of needle like crystals, causing expansive forces that result in cracking. The extent of DEF development is dependent on the amount of sulphate available for late ettringite development in the particular concrete and on water presence during the service life. Elevated temperatures also increase potential for damage due to DEF [9].

Prevention or minimization of DEF can be accomplished by lowering curing temperatures, limiting clinker sulphate levels, avoiding excessive curing for potentially critical sulphate to aluminate ratios, preventing in service water exposure and using proper air entrainment. Limiting internal concrete temperatures to 70°C during very early life can be achieved either by direct specification, or indirectly by limiting the cement content or specifying the use of low or very low heat cement.

Neither the mechanisms involved in DEF, nor their potential consequences relative to concrete durability are completely understood. The DEF process leads to degradation of mechanical properties such as compressive strength, and can promote increased permeability. Figure 35 shows cracking damage due to DEF.

FIG. 35. Cracking damage in a concrete structure due to DEF [131].

(d) Acids and bases

Concrete is not acid resistant and will not last long if exposed to solutions of pH3 or lower [132]. Sulphuric or carbonic acids present in groundwater, sewers and certain plant internal fluids (e.g. boric and sulphuric acids) can combine with calcium compounds in the cement paste (i.e. calcium hydroxide, calcium silicate hydrate and calcium aluminate hydrate) to form soluble materials that are readily leached away, increasing concrete porosity and permeability. The extent of attack is determined more by the solubility of the resulting calcium salt than on the aggressiveness of the acid. Flowing chemical solutions exhibit accelerated deterioration rates.

Nitric, hydrochloric and sulphuric acids are very aggressive, as their calcium salts are readily soluble and removed from the acid front. Organic acids such as formic, acetic, butyric and lactic are also corrosive to concrete. Other acids such as phosphoric, carbonic, tannic and humic are less harmful as their calcium salts have low solubility and inhibit the attack by blocking pathways within the concrete. Oxalic acids have negligible effect on Portland cement concretes. Carbonic, humic and sulphuric acids are the acids most commonly encountered by concrete since they are found in natural groundwater. Acid rain does not have a significant effect due to the large buffering capacity of concrete and relatively small amount of acid contained within the rain [9].

Concrete undergoing acid attack will visually show disintegration in the form of loss of cement paste and aggregate from the matrix. Figure 36 illustrates acid attack and presents an example of acid attack on a concrete wall.

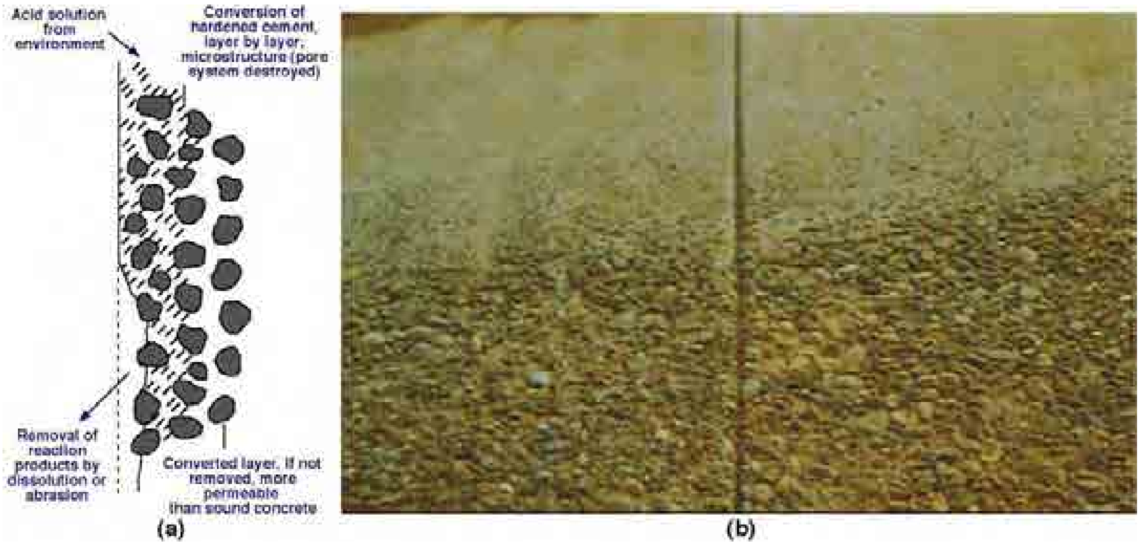


FIG. 36. (a) Mechanism of surface loss due to acid attack [87]; (b) example of acid attack on a concrete wall [98].

Since cement paste is alkaline, high quality concretes made with chemically stable aggregates normally are resistant to bases. Sodium and potassium hydroxides in high concentrations (>20%) can, however, cause disintegration. Dense concretes with a low water to cement ratio can provide suitable resistance to mild attack [9].

As corrosive chemicals can attack concrete only in the presence of water, designs to minimize attack by acids and bases generally involve the use of protective barrier systems. Table 6 presents a listing of reactivity of various chemicals with concrete and steel. References [133] and [134] present additional information on the effects of chemicals on concrete.

TABLE 6. REACTIVITY OF VARIOUS MATERIALS WITH CONCRETE AND STEEL (*adapted from Ref*. [133])

Material	Effect on concrete	Effect on steel
Acetone	Liquid loss by penetration (may cause slow disintegration)	None
Acidic water (less than pH6.5)	Disintegrates concrete slowly	May attack rebar and embedments
Boric acid	Negligible effect unless immersed	Severely corrosive to liner and reinforcing steel
Borated water (and boron)	Negligible effect unless immersed	Very corrosive at high concentration
Chlorine gas	Concrete (moist) slowly disintegrates	Highly corrosive
Demineralized water	Leaching	Slight
De-icing salts	Scaling of non-air entrained concrete	Highly corrosive
Diesel exhaust gases	Disintegration of moist concrete caused by carbonic, nitric or sulphurous acid; minimal effect on hardened dry concrete	Minimal
Hydrochloric acid	Disintegrates concrete rapidly	Highly corrosive

TABLE 6. REACTIVITY OF VARIOUS MATERIALS WITH CONCRETE AND STEEL (*adapted from Ref.* [133]) (cont.)

Material	Effect on concrete	Effect on steel
Hydroxides	At low concentrations, slow disintegration; at high concentrations, greater disintegration	Unknown
Nitric acid	Disintegrates rapidly	Highly corrosive
Lubricating oil	Fatty oils, if present, slowly disintegrate concrete	Minimal
Sea water	Disintegrates concrete with inadequate sulphate resistance	Highly corrosive
Sodium hydroxide	Not harmful below 20% concentration, disintegrates at concentrations above 20%	Minimal
Sodium pentaborate	Disintegrates at varying rates depending on concentration	Dependent on concentration
Sulphates	Disintegrates at varying rates with concentration (concretes with low sulphate resistance such as Type I Portland cement concrete)	Harmful at certain concentrations
Sulphuric acid (sulphurous)	Disintegrates rapidly in concentration between 10–80%	Very corrosive

(e) Alkali–aggregate reactions (AARs)

Some siliceous and dolomitic aggregates can react with the alkali hydroxides in concrete, causing expansion and cracking over a period of many years. This alkali–aggregate reaction has two forms: alkali–silica reaction (ASR) and alkali–carbonate reaction (ACR).

ASRs are more of a concern because aggregates containing reactive silica materials are more common. In an ASR, alkali ions form a calcium alkali-silicate gel. This gel takes up pore solution water due to the attractive forces between polar water molecules and alkali-silicate ions, expands and creates potentially damaging tensile stresses from 4100–11 000 kPa within the cement paste matrix [135]. Figure 37 presents the ASR and the resultant gel. Primary factors influencing ASRs include aggregate reactivity (i.e. amount and grain size of reactive aggregate), alkali and calcium concentrations in concrete pore water, cement alkali content and water presence [9].

ASR can be controlled using certain supplementary cementitious materials. In proper proportions, silica fume, fly ash and ground granulated blast-furnace slag can significantly reduced or eliminated expansion due to ASR.

Expansion reactions can also occur as a result of ACRs (i.e. dedolomitization). Dedolomitization, the breaking down of dolomite, is normally associated with expansion. Aggregates susceptible to this type of reaction are typically dolomitic limestone consisting of a fine grained matrix of calcite and clay in which larger crystals (20–80 μm) of euhedral dolomite rhombohedra are suspended [137]. ACR can be distinguished from the ASR by the lack of a silica gel exudation at cracks [94].

Unlike ASR, the use of supplementary cementing materials does not prevent deleterious expansion due to ACR. It is recommended that ACR susceptible aggregates not be used in concrete.

The most reactive forms of aggregates found in the world are strained quartz, amorphous silica, cryptocrystalline quartz, chalcedony and chert [138]. Aggregates containing crystalline silica are generally stable, while those with amorphous or very fine grained silica are reactive (see Appendix B of Ref. [95]). Although ASRs typically occur within 5–10 years of construction, deterioration has occurred in some structures 15 or even 25 or more years following construction. This deterioration delay indicates that there may also be less reactive forms of silica [92].

Figure 38 presents an example of ASRs in a concrete structure.

ASRs begin with extremely fine cracks, and thus may go unrecognized for years. If the concrete member is unrestrained, damage becomes visible as small surface cracks exhibiting an irregular pattern (or map

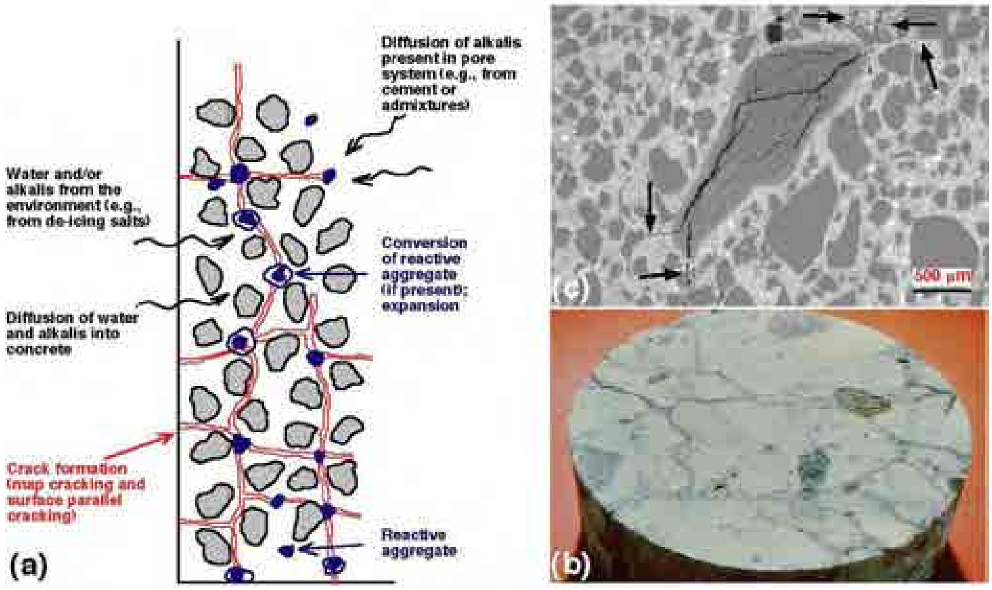


FIG. 37. Concrete cracking due to ASR: (a) mechanism [87]; (b) resulting gel that causes expansion and cracking [98]; (c) polished section of concrete showing chert particle with extensive internal cracks extending from aggregate as noted by arrows [136].

FIG. 38. Example of cracking and gel resulting from ASRs in concrete (courtesy of CTLGROUP) [139].

cracking). When expansive forces are restrained (e.g. by rebar), the pattern will develop in the direction of least constraint (i.e. parallel surface crack patterns propagating inward from the surface for slabs, and cracking parallel to compression forces in columns or prestressed members). Pop-outs and glassy-appearing seepage of varying composition can appear as a result of ASRs. Cracks allow access to the concrete interior, enabling other deleterious mechanisms to operate [9]. ASRs occurring in salt contaminated concrete increase the risk of rebar corrosion [140].

To date, ASRs have been observed in NPP concrete structures in Belgium, Canada, the USA and Japan (turbine generator foundation of the Unit 1, Ikata NPP) [141, 142].

(f) Aggressive marine water

Aggressive water environments such as those found in sea water, brackish water, waste water, and as part of some industrial processes can degrade concrete. The most common damaging ion encountered is sulphate [143]; other damaging aggressive waters contain acids and chemical by-products.

Most sea water has a pH of 7.5–8.4, is fairly uniform in chemical composition and contains about 3.5% soluble salts by weight (Na^+ and Cl^- have the highest ionic concentrations, but Mg^{2+} and SO_4^{2-} are also present) [120]. Magnesium sulphate reacts with cement hydration products and forms ettringite, calcium sulphate and insoluble magnesium hydroxide (brucite), which reduce rates of attack. Attack rates are further reduced by formation of aragonite (calcium carbonate), which forms more readily in tidal zones than on continuously immersed elements [144].

Concrete exposed to marine environments may deteriorate as a result of the combined effects of the following [9]:

— Chemical action of sea water constituents on cement hydration products;
— AAR (if reactive aggregates are present);
— Crystallization pressure of salts within concrete (if a structure face is subject to wetting and others to drying);
— Frost action in cold climates;
— Corrosion of embedded rebar;
— Physical erosion due to wave action or floating objects.

Figures 39 and 40 show an example of seawater attack and deterioration of seawater exposed concrete.

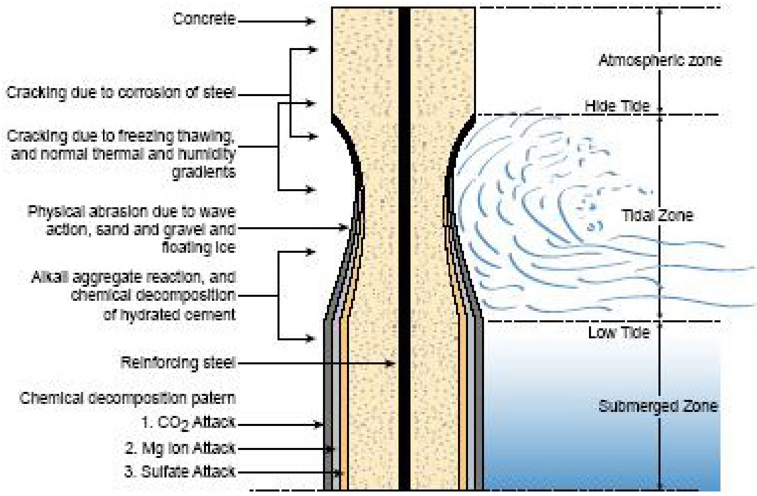


FIG. 39. Seawater attack of concrete mechanism [145].

FIG. 40. Seawater damage to concrete [146].

(g) Biological attack

Concrete structures can undergo biodeterioration when exposed to contact with lichens, mosses, algae, roots of plants and trees, marine molluscs, sponges, soil, water, sewage, as well as food, agricultural products and waste materials. Biodeterioration can occur in foundations, walls, dams, harbour and maritime structures, bridges, tanks, pipelines, cooling towers and silos, among others. Organisms colonize material surfaces, and their pores, capillaries and microcracks, causing concrete damage resulting in aesthetic, functional or structural problems. Increased roughness as surfaces age facilitates the colonization of microbes. Microorganisms affect the concrete mainly by contributing to erosion of the exposed concrete surface, reducing protective cover depth, increasing concrete porosity, and increasing transport of degrading materials into the concrete that can accelerate cracking, spalling and other damage and reduce the service life of the structure [147].

Microbes have extremely diverse modes of metabolism, are natural inhabitants of soil, and can survive extreme environments such as the inner wall of cooling towers [148]. The effects of microbial metabolism are initially primarily of a chemical nature. Later on, microbes may influence physical processes such as the progression of freeze/thaw damage or crack formation leading to water ingress and rebar corrosion. All types of microorganisms (bacteria, algae, lichens, yeasts and fungi) can be involved, where they can act separately or jointly in complex interactions.

Table 7 contains some selected examples of biological attack mechanisms, and Fig. 41 presents pictures of some biological attacks on concrete.

TABLE 7. SELECTED EXAMPLES OF BIOLOGICAL ATTACK

Mechanism	Description	Comment
Physical damage	Organic growth on concrete can retain water on surfaces leading to high moisture content and increased risk of deterioration due to freezing. Rock boring molluscs and sponges, which are common in reefs or areas where seabeds are composed of limestone, may invade underwater concrete structures and piles containing limestone aggregate. Marine borers reduce load carrying capacity and expose rebar to corrosive sea water. Boring sponges produce interconnected bore holes, which can cause crumbling of surface material.	
Chemical damage (e.g. organisms dissolving concrete through leaching of calcium and other alkaline binding materials [149])	Acids formed by the degradation of dead organic matter (humic acid) [88] or by microorganism metabolisms (sulphuric or nitric acids). Sulphur oxidizing bacteria such as thiobacilli present in wastewater and sewage environments produce sulphuric acid. Acid reacts with free lime [$Ca(OH)_2$] forming gypsum ($CaSO_4 \cdot 2H_2O$), producing a corroding layer surface that penetrates into the concrete. Newly formed gypsum crystals react with cement's calcium aluminate to produce ettringite that increases internal pressure leading to cracks [151]. Nitric acid on concrete calcareous components resulting in production of calcium nitrate (a soluble salt) that is either lost from the concrete, resulting in formation of corrosion pits, or remains to add salt to the pore water [152]. Gluconic, malic and oxalic acids produced by fermentative bacteria that are natural soil inhabitants causing concrete damage [153].	Sulphate reducing bacteria are primarily responsible for above ground concrete degradation, and nitrifying bacteria are responsible for degradation below ground [150].

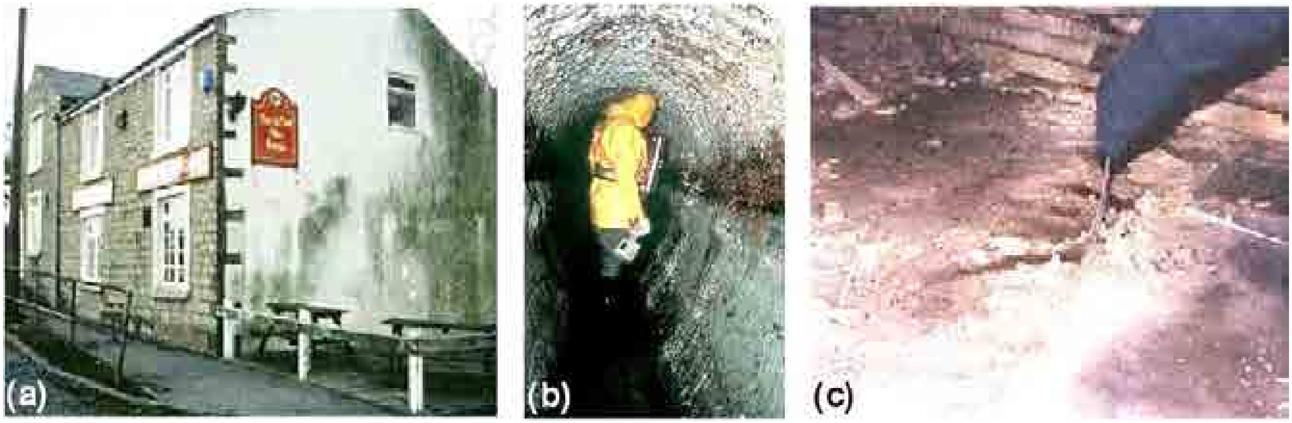


FIG. 41. Concrete biological attack: (a) algae growth on the outside wall of house [150]; (b) biogenic sulphuric acid attack in a sewer system [154]; (c) decaying concrete floor in flooded cellar [150].

4.5.3. Containment liners

4.5.3.1. Metallic liner and structural steel

Corrosion is the main degradation mechanism for steel components within NPPs. Typically, containment liner plates and any installed steel that is not in contact with concrete are coated to prevent corrosion. Components may have been designed with a specific corrosion allowance, however, little allowance will have been provided for the relatively thin (i.e. ~6.3 mm) liner plates.

Liner and structural steel component corrosion is similar to that for conventional reinforcing steel. Figure 42 presents a number of corrosion types that may be found on metals [155]. Corrosion may progress rapidly, depending on environmental aggressiveness. Predictive tables of corrosion data for structural steel in numerous environments are available [156]. For industrial environments, atmospheric general corrosion rates are in the range of 0.02–0.04 mm/a. This same reference reported pitting rates of 0.056 mm/a for low carbon steels placed in polluted sea water. Surface corrosion rates typically range from 0.001–0.03 mm/a [9].

For liner plates, localized corrosion leading to a loss of leaktightness is of most concern. Metallic liner corrosion can begin at either the inside or outside liner surface.

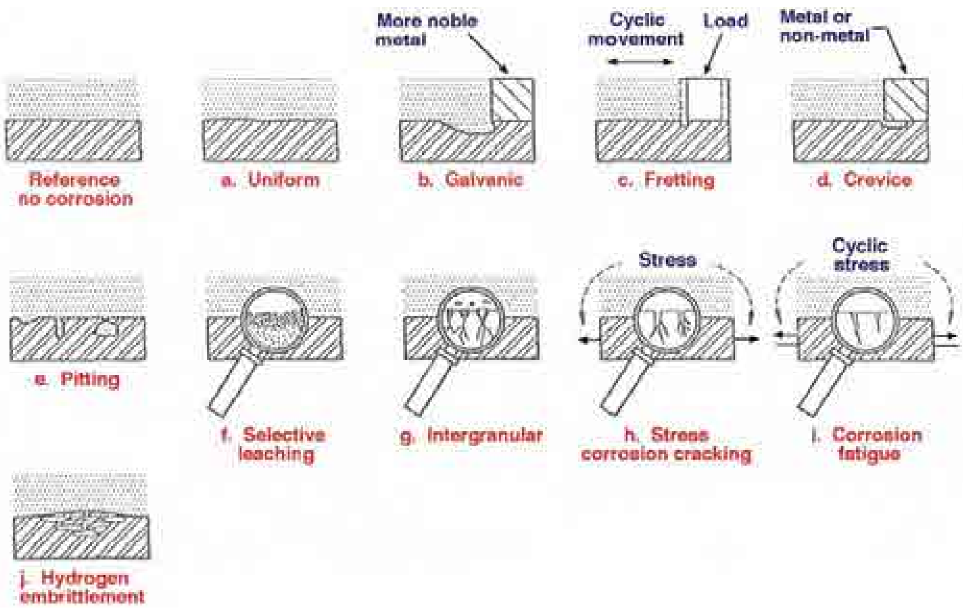


FIG. 42. Schematic representation of forms of corrosion that may be found in metals (adapted from Ref. [155]).

Inside liner corrosion is the most common and is typically found at a floor to wall joint where the liner becomes embedded in the concrete basemat. These have been due to coating failures or moisture barrier degradation [157]. Areas experiencing loss of coating integrity can accumulate moisture, or adjoining floor liner sealants can fail. Strong inspection programmes can normally detect this corrosion before it challenges liner integrity

Incidences of liner corrosion have been identified where corrosion began on the concrete side of the liner due to the presence of a foreign object (e.g. wood, brush handle, worker's glove or felt) [157]. Figure 43 presents an example of liner corrosion that originated at the concrete side of a liner due to an embedded foreign object. Detection was through a visual examination that identified corrosion products (e.g. staining or a paint blister) on the accessible inside liner surface. Visual examinations performed in accordance with codes were unable to identify the corrosion until it had penetrated the liner wall. The difficulty was that the entire large liner surface area was impractical to inspect with the available ultrasonic methods.

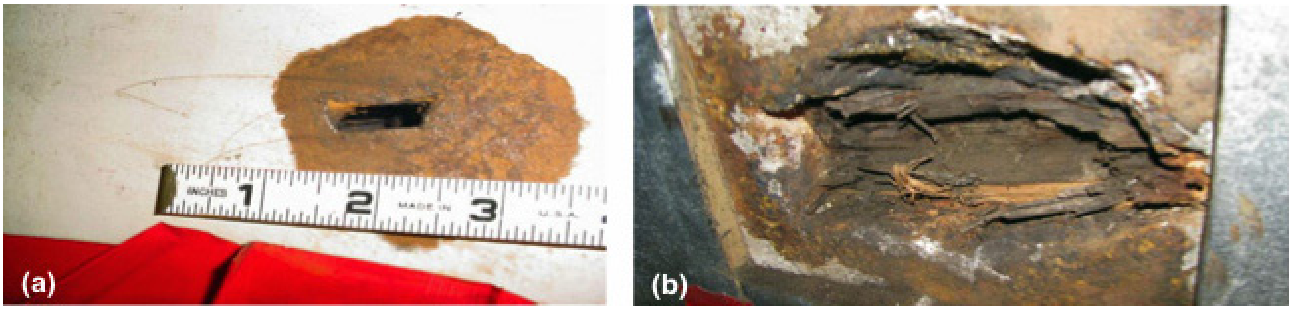


FIG. 43. Examples of corrosion originating from the concrete side of the liner: (a) paint blister; (b) wood embedded in concrete after a section of liner was removed [158].

Repeated loading from equipment like the polar crane or from flow induced vibrations may impact the function and performance of liner plates and structural steel. Repeated loads are generally addressed during design via adherence to national design codes. However, conditions outside of design predictions and local stress intensification points (e.g. material flaws) may result in fatigue. For liner plates, possible fatigue sites include base metal delaminations, weld defects, arc strikes, shape changes near penetrations, structural attachments and concrete floor interfaces. For structural steel (liner attachments and anchorages), the most susceptible fatigue locations include large containment penetration frames (i.e. hatches) and liner anchorages near vibrating loads (such as in structural attachments) [9].

4.5.3.2. Non-metallic liners, coatings and sealers

Non-metallic liners and coatings are subjected to a number of degradation stressors such as elevated temperatures, radiation, abrasion and high humidity. The primary form of non-metallic liner degradation has been the occurrence of cracks due to localized effects (e.g. stress concentrations) in substrata such as, movement of joints, or physical or chemical concrete changes. Delaminations may also occur due to pressure fluctuations.

The primary degradation mechanism for joint sealants is weathering, which results from environmental exposure to sunlight, changes in humidity and pressure fluctuations.

Non-metallic liners and joint sealants can also suffer physical damage during their lives.

4.5.4. Mild steel reinforcing systems

Concrete has limited capacity for plastic deformation and absorption of mechanical energy. Rebar is typically installed in locations where tensile stresses are anticipated to address this. Fortunately, rebar and concrete are mutually compatible. They have similar coefficients of thermal expansion, and the relatively high pH of the concrete pore water (pH~12.5–13.6) contributes to formation of an oxide film that passivates steel against corrosion. Disruption of the passive film can, however, occur due to carbonation or chloride intrusion. Durability concerns need to address performance of the embedded rebar as well as the interaction of concrete and steel [9].

Mild reinforcing steel can degrade due to corrosion, exposure to elevated temperatures, irradiation and fatigue. Of these, corrosion is of most concern with respect to durability of concrete structures. The following sections cover each ageing mechanism in more detail.

4.5.4.1. Corrosion

Corrosion of steel in concrete is an electrochemical process. General corrosion refers to a relatively uniform reduction of thickness over the surface of a corroding material. It is relatively easy to measure and monitor. Pitting corrosion is a localized form of corrosion where the bulk of the surface remains unattacked. Pitting is often found at locations where resistance against general corrosion provided by passive surface films or coatings has broken down.

Both water and oxygen must be present for corrosion to occur. There is no corrosion in dry concrete or in fully immersed concrete that does not contain entrained air. The electrochemical potentials that form corrosion cells may be generated in two ways:

(1) Composition cells formed when two dissimilar metals are embedded in concrete, such as rebar and aluminium conduit, or when significant variations exist in steel surface characteristics;
(2) Concentration cells formed due to differences in concentration of dissolved ions near steel, such as alkalis, chlorides and oxygen [120].

One of two metals (or different parts of the same metal) becomes anodic and the other cathodic to form a corrosion cell. Corrosion may also be induced by stray electrical currents or galvanic action with embedded steel of different composition. Figure 44 illustrates the electrochemical process of steel corrosion in moist and permeable concrete.

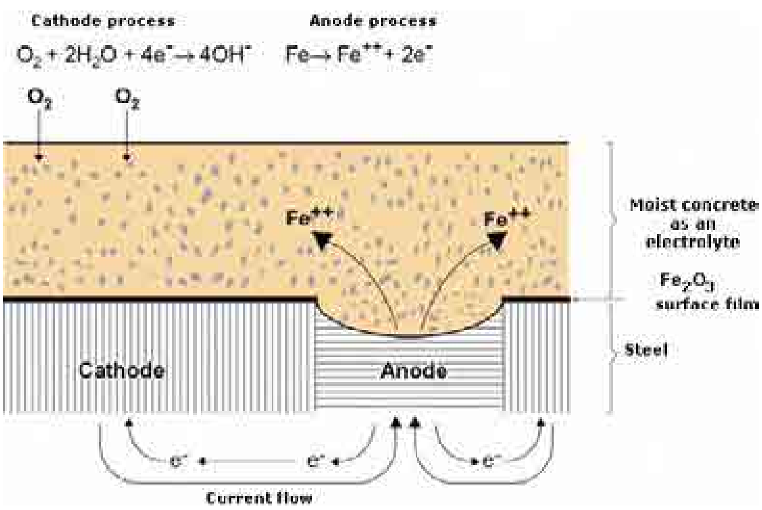


FIG. 44. Electrochemical reaction illustrating corrosion of steel in concrete (adapted from Ref. [159]).

Figure 45 presents examples of rebar corrosion in general civil engineering structures.

In good quality, well compacted concrete, reinforcing steel with adequate cover should not be susceptible to corrosion, because the highly alkaline conditions (pH>12) cause a passive iron oxide film (gamma Fe_2O_3) to form on the concrete surface (i.e. metallic iron will not be available for anodic activity). The passive film may be relatively thick to inhibit corrosion by providing a diffusion barrier to reaction products of the reacting species (Fe and O_2) or, as is more common, the layer can be very thin. The film does not actually stop corrosion, but reduces corrosion rates to insignificant levels [162]. Corrosion can occur if this passivating environment is altered by a reduction of concrete pH or by the introduction of chlorides that destabilize the passive layer.

FIG. 45. Corrosion of reinforced concrete. (a) Seawater structure; (b) bridge structure on highway 401 in Ontario, Canada (adapted from Refs [160] and [161], respectively).

Concrete pH can reduce as a result of leaching of alkaline substances by water or carbonation. Carbonation is a slow process, whereby calcium hydroxide (lime) is converted to calcium carbonate (calcite) due to ingress of carbon dioxide into concrete. When pH falls below about 11.5, a porous oxide layer (rust) can form on reinforcing steel due to corrosion [163]. More recent research indicates that the corrosion threshold is considered to be reached once pH is reduced to 9.5, and there is a steep decrease in corrosion potential indicating decomposition of the passive layer at a pH of about 8 [164]. Since carbon dioxide is a minor component of the atmosphere (~0.03% by volume), penetration of CO_2 from the environment is generally slow. Carbonation rates depend on concrete permeability, moisture content, CO_2 content, temperature and relative humidity of the environment (i.e. 50–75% r.h. with 60–65% being the maximum and extremes being capable of preventing carbonation). Carbonation rates at exposed surfaces are considered to be roughly proportional to the square root of time for concrete kept continuously dry at normal relative humidity [165].

Carbonation generally proceeds in concrete as a front, beyond which concrete is unaffected, and behind which pH is reduced. Carbonation causes concrete strength to increase, but this is generally insignificant because only surface zones become carbonated. Although carbonation reduces concrete permeability, it produces a greater propensity for shrinkage cracking that can counteract any positive durability effects of reduced permeability [166]. Expected times to corrode increase as concrete cover increases and water to cement ratios decrease. In NPPs, carbonation will most probably occur inside concrete surfaces that are exposed to relatively low humidity and elevated temperatures [167]. Depth of carbonation can be observed by treating freshly exposed concrete surfaces with phenolphthalein (a pH indicator) [168]. Figure 46 shows carbonated concrete identified using phenolphthalein. More precise methods for determining carbonation depth include petrography (microscope), X ray diffraction and differential thermal analysis techniques to analyse drilled powder samples that were obtained from various depths [166].

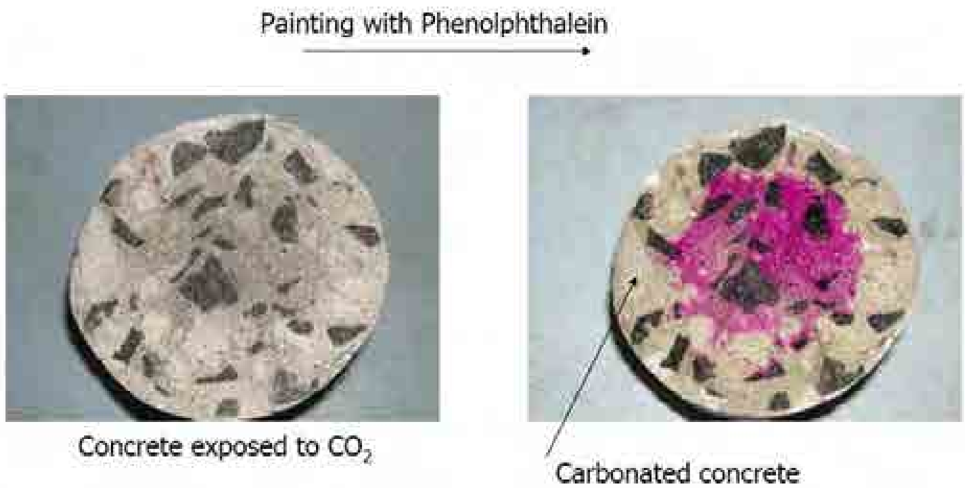


FIG. 46. Painting with phenolphthalein to identify carbonated concrete (painted concrete on right) [159].

Steel corrosion is most commonly caused by chloride ions that destroy the passive iron oxide film, even at high alkalinity (pH>11.5). The mechanism through which the film is destroyed is not fully understood; either chloride ions convert insoluble iron oxide to soluble iron chloride, or they become included in the oxide layer in a manner that makes it permeable to air [169]. Chloride penetration causes local variations in steel concentration, forming concentration cells. Chloride ions are attracted to steel anodic regions, and increased acidity around anodic sites can lead to local dissolution of cement paste. Cracks resulting from direct loading or other causes can allow rapid penetration of CO_2 or chloride ions into rebar, thereby causing local failure of the oxide film. This may lead to pitting corrosion and reduction of rebar cross-section. The volume of corrosion products may be so small that no external signs appear [9].

Diffusion of chlorides can also occur in uncracked sound concrete through the cement paste's capillary pore structure. Diffusion rates are strongly dependent on a number of factors, including water to cementitious material ratio, cement type, temperature and maturity of concrete. Some chlorides react chemically with cement components (e.g. calcium aluminates) and are effectively removed from the pore solution. The amount of total chlorides available in the pore solution to cause breakdown of the passive film is a function of a number of parameters (e.g. C_3A and C_4AF content, pH and source of chlorides (mix or environment)). The threshold value of chloride concentration below which significant corrosion does not occur is dependent on these parameters. Different organizations have proposed various values: British Standard 8110 and European Standard ENV list 0.4% Cl by mass cement (see Refs [170] and [171]), whereas the ACI lists 0.15% water soluble Cl by mass cement [172]. Investigators have reported minimum threshold values for chloride ion content, in order to initiate corrosion, to be between 0.026–0.033% (approximately 0.6–0.8 kg/m^3) of total chloride ion content by mass of concrete [162]. The threshold acid-soluble-chloride content to initiate steel corrosion reported by various investigators ranges from 0.15% to 1.0% [172].

Chlorides can come from external (e.g. sea water or de-icing salts) or internal (e.g. aggregate or mix water) sources. Concrete tends to hold more moisture when large amounts are present, which also increases the corrosion risk by lowering electrical resistivity. Once steel passivity is destroyed, electrical resistivity of concrete and the availability of oxygen determine corrosion rates.

When metallic iron is transformed to ferric oxide (rust) it increases in volume and can initiate cover concrete cracking. Because corrosion is fairly uniform, such cracking usually occurs prior to a particular structural cross-section becoming excessively weak, thus giving a visual warning of deterioration [173]. Occasionally, however, cover spalling occurs before any surface signs of deterioration are visible. Structural strength and serviceability are only reduced and jeopardized when rebar corrosion causes a significant loss of steel cross-section, or there occurs a loss of bond between steel and concrete [174].

4.5.4.2. Elevated temperature

Elevated temperatures can result from fires, internal heat sources (e.g. NPP reactors) and other sources. Large reinforced concrete structures have high thermal inertia; this results in relatively slow rates of temperature rise. Internal rebar temperatures, thus, remain sufficiently low to avoid significant softening. Such structures, therefore, generally perform well under elevated temperatures. However, under certain scenarios (e.g. rapid heat buildup), spalling of concrete can occur and expose rebar to high temperatures.

The most important rebar properties are yield stress and modulus of elasticity. Almost all information available on elevated temperature effects addresses residual strength of rebar after fire exposure [9]. German reinforcing data indicates that for temperatures up to ~200°C, yield strength is reduced by 10% or less, and at 500°C it falls to about 50% its room temperature value [175]. Hot rolled steels resist temperature effects better than cold drawn or twisted steel. With cold worked steel, work hardening effects that increase rebar strength under normal exposure conditions suffer regression if exposed to high temperatures (i.e. >400°C) [176]. At temperatures lower than 400°C, residual hardening due to ageing may be observed. Steel modulus of elasticity exhibits similar reductions to that of yield stress with increasing temperature. Other data confirms temperature effects above 200°C on mild steel reinforcement, and provides a threshold temperature of about 300°C for loss of bond properties with concrete [177]. Figure 47 presents data on stress-strain, Young's modulus and elongation as a function of temperature and yield and strength as a function of temperature for a 3500 kgf/cm^2 (343 MPa) specified minimum yield strength 51 mm diameter steel bar [178]. Additional information is available on the elevated temperature effects on stress-strain behaviour of 12 mm and 25 mm diameter quenched and tempered steel bars, and a comparison of results with recommendations is available in the European Committee for Standardization (CEN) standards for structural fire design [179, 180].

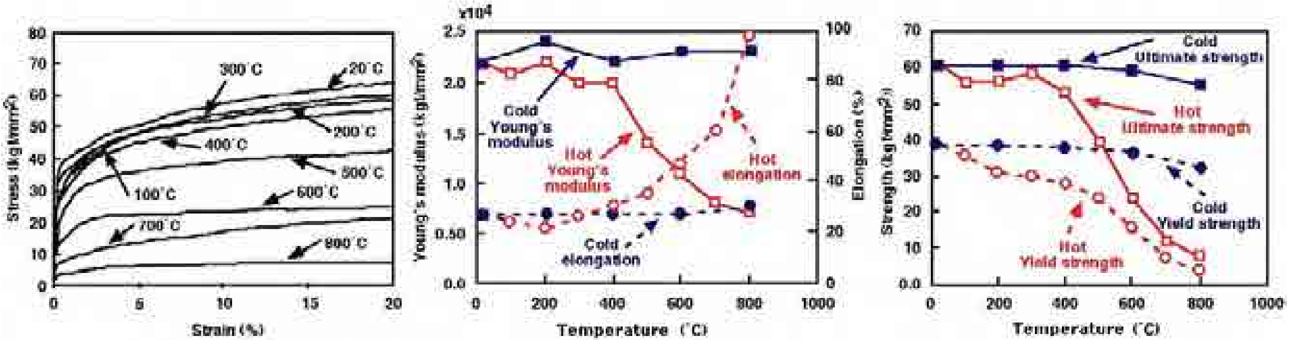


FIG. 47. Effect of temperature on properties of a 3500 kgf/cm^2 (343 MPa) minimum specified yield strength steel bar (adapted from Ref. [178]).

4.5.4.3. Irradiation

Neutron irradiation changes the mechanical properties of carbon steels. It increases yield strength and the ductile to brittle transition temperature. The changes result from the displacement of atoms from their normal sites by high energy neutrons, causing formation of interstitials and vacancies. A threshold level for neutron fluence of 1×10^{18} n/cm^2 has been cited for alteration of reinforcing steel mechanical properties [181]. Fluence levels of this magnitude are not likely to be experienced by concrete structures in NPPs, except possibly in the concrete primary biological shield wall over an extended operating period [182].

4.5.4.4. Fatigue

Fatigue of rebar is similar to the concrete fatigue that was discussed in Section 4.5.2.1(f). Applied repeated loading or vibration causes a loss of bond between the rebar and concrete. In extreme cases, rebar strength may be reduced or failures may occur at applied stress levels less than yield. However, there have been few documented cases of fatigue failures of reinforcing steel in concrete structures, and those occurred at relatively high stress/cycle combinations [117]. 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 [84].

4.5.5. Post-tensioning systems

Potential causes of post-tensioning system degradation include:

— Corrosion;
— Exposure to elevated temperatures;
— Irradiation;
— Fatigue;
— Stress relaxation/end effects.

Of these, corrosion and loss of prestressing force are the most pertinent consequences from an NPP durability perspective.

4.5.5.1. Corrosion

Most corrosion related to prestressing systems is caused by water seepage through zones of porous concrete or vulnerable areas, such as leaking seals, joints, anchorages or cracks, which then flows to the areas of concern [183]. The corrosion can be highly localized or uniform. Typical failures in general civil engineering structures have resulted from local attacks produced by one, or a combination of, the following: pitting, stress corrosion or hydrogen embrittlement [9].

Pitting results in local intense material loss at tendon surfaces, potentially reducing the cross-section to where it is incapable of supporting load. Figure 48 illustrates the effect of pitting on tensile strength and elongation of a cold deformed 5 mm diameter wire having a specified minimum tensile strength of 1800 MPa (German Specification St 1570/1770) [183].

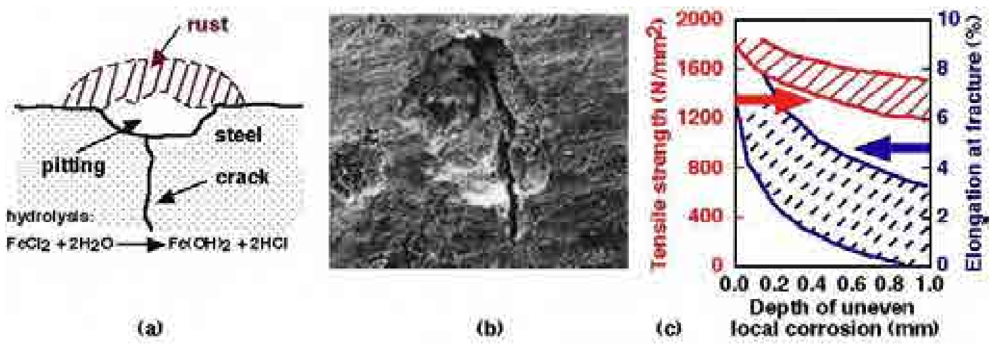


FIG. 48. Pitting corrosion of prestressing steel: (a) mechanism; (b) pitting induced stress corrosion cracking; (c) effect of pit depth on tensile strength (MPa = N/mm^2) and elongation [183].

Stress corrosion cracking results in fractures of a normally ductile metal or alloy under stress (tensile or residual) while in specific corrosive environments.

Hydrogen embrittlement, frequently associated with hydrogen sulphide exposure, occurs when hydrogen atoms enter the metal lattice and significantly reduce ductility. It may also occur as a result of improper application of cathodic protection to post-tensioning systems [184–186]. Post-tensioning system failure can also occur as a result of microbiologically induced corrosion. Due to the stressed (tensioned) state of post-tensioning systems, their tolerance for corrosion attack is much less than for rebar [85].

4.5.5.2. Elevated temperature

Elevated temperature can significantly impact all heat treated and drawn wires. Such wires may not regain their initial strength when cooled because subsequent heating destroys crystal transformations achieved by the initial heat treating process. Short term heating (e.g. 3–5 min) even to temperatures as high as 400°C, however, may not harm mechanical properties of prestressing wire [187]. A Belgian study [175] involving 30 types of prestressing steel indicated that thermal exposures up to ~200°C do not significantly reduce (<10%) tensile strength of prestressing wires or strands. References [188] and [189] support these results. The effect of elevated temperatures (from 21–649°C) on the stress-strain behaviour of one type of prestressing steel (ASTM A 421) is provided in Ref. [190]. As temperature levels experienced by prestressing tendons in NPP containment are typically below 200°C, the risk of thermal damage under normal operating conditions is low.

The loss of prestressing force increases at elevated temperatures due to increases in the relaxation of steel and volumetric changes of concrete (i.e. creep and shrinkage). As shown in Fig. 49, the change in slope of prestress force versus time is associated with a reactor startup, when temperatures increase from about 20°C to 30–45°C [191].

Elevated temperature exposures impact the relaxation of prestressing tendons. As exposure temperature and initial stress levels increase, prestressing wire relaxation losses can increase significantly [192–195].

Results obtained from prestressing wires obtained from four British manufacturers indicate that as exposure temperature was increased from 18–120°C, stress relaxation was approximately proportional to the temperature increase [196].

Figure 50 shows a 15.2 mm 7 wire strand stress loss from initial stress of 75% of specified minimum breaking strength [192]. Relaxation at 40°C is about twice as much as at 20°C. Based on testing performed by Briton Wires [195], the average relaxation after 40 years was estimated to be 2% at 20°C and 5% at 40°C.

Relaxation tests were performed at Forsmark NPP (Sweden) for 1000 hours, and to 70% of their ultimate strength at temperatures of 20°C and 50°C [194]. Results were 1.4% at 20°C and 2.7% at 50°C.

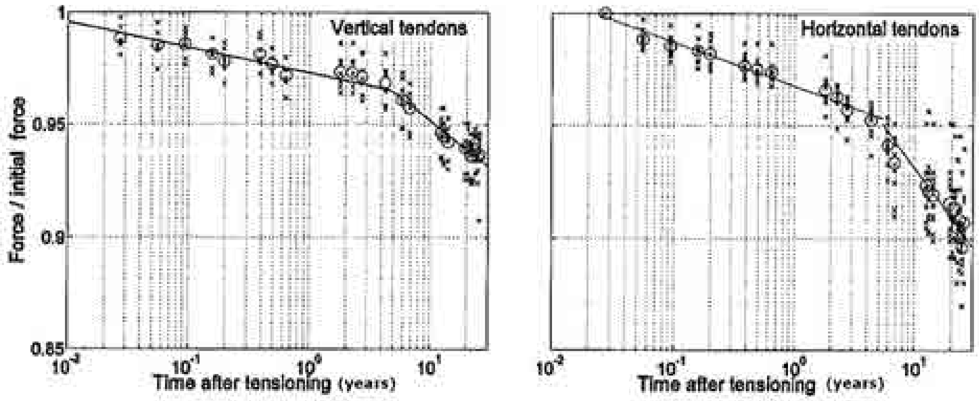


FIG. 49. Prestressing (initial) and residual force as a function of time as measured by fixed gauges at Forsmark 1(adapted from Ref. [191]).

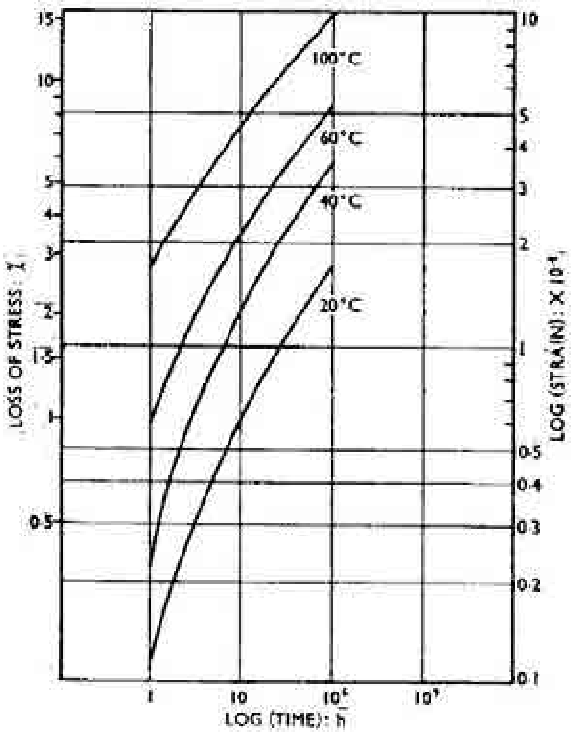


FIG. 50. Relaxation of stabilized (low relaxation) 7 wire strands at 75% of F_u at various temperatures [192].

Temperature has significant effect on creep [93, 191, 194, 195, 197]. This is not typically addressed in generic creep prediction models, and temperature is only of concern for prestressed nuclear CCSs. Creep rate increases with temperatures up to about 70°C. The creep rate at 70°C was shown to be about 3.5 times higher than the rate at 20°C, at least for the first 15 months under load [93]. Based on results from a number of studies [191], creep at temperatures of between 40 and 50°C is about twice the creep at a temperature of 20°C. This is consistent with the results in Fig. 51, showing short term creep. Investigations in Ref. [195] show that the creep strain is 2–3 times greater at 52°C than at 27°C.

Elevated temperature effects on drying shrinkage are less pronounced than the effect of reduced relative humidity. A 15% increase in shrinkage can be expected when temperature is increased from 23°C to 60°C at a constant relative humidity [197].

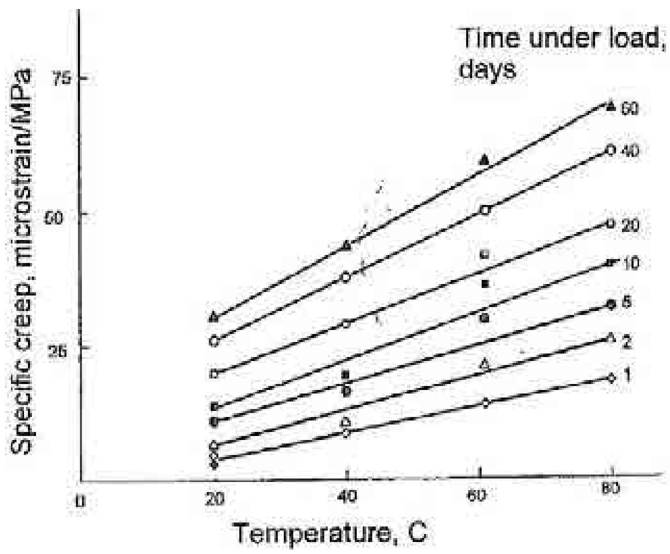


FIG. 51. Effects of temperature on creep [197].

4.5.5.3. Irradiation

Irradiation affects prestressing steel's mechanical properties similar to mild reinforcement (see Section 4.5.4.3). Defects can propagate or combine, and effectively both strengthen the steel and reduce its ductility. At higher temperatures, defects can recombine and cancel each other and, for a given neutron dose, reduce irradiation damage [181]. Results obtained from studies in which 2.5 mm diameter prestressing wires were stressed to 70% of their tensile strength and irradiated to a total fluence of 4×10^{16} n/cm^2 (flux of 2×10^{10} n/cm^2/s) showed that for exposures up to this level, the relaxation behaviour of irradiated and non-irradiated material was similar [181].

4.5.5.4. Fatigue

Repeated reversals of stress or variations in stress applied to prestressed concrete structural elements (beams in particular) can result in fatigue failure in any of the following modes [9]:

- — Failure of concrete due to flexural compression;
- — Failure of concrete due to diagonal tension or shear;
- — Failure of prestressing steel due to flexural, tensile stress variations;
- — Failure of pretensioned beams (grouted tendons) due to loss of bond stress;
- — Failure of end anchorages of post-tensioned structures [198].

Most fatigue failures that occurred while testing prestressed concrete beams resulted from tendon fatigue due to stress concentrations that occurred in the tendon at a location where a crack forms. In unbonded post-tensioned construction, end anchorages can be subject to variations in stress under changing external loads. Unbonded tendons are not generally used in members subjected to frequent variations in stress [9]. High cycle and low cycle dynamic tensile test requirements for prestressing tendon systems used in containments are available in Refs [87] and [199].

4.5.5.5. Loss of prestressing force

Maintaining adequate prestressing force in a post-tensioned concrete containment is important to overall NPP safety, especially during postulated accident conditions. The main contributors to losses of force levels applied by the prestressing tendons include:

- — Friction;
- — End anchorage deflection (take up and slip);

— Elastic shortening;
— Tendon relaxation;
— Concrete creep and shrinkage [187, 200–202].

Tendon relaxation and concrete creep and shrinkage are time dependent factors and, thus, are ageing related. Such factors are typically conservatively accounted for in design, however, if the material used (steel or concrete), or the exposure conditions, are outside the considered range, prestressing levels may drop below the design value and structure integrity might be compromised.

Stress relaxation is the loss of stress (force) in prestressing steel when strain (elongation) does not vary. It is related to tendon material properties, initial stress levels, exposure temperatures and time. Creep and shrinkage represent volume changes of concrete that occur over the life of a structure, and can significantly affect force levels in the tendons.

Data collected from more than 150 containment structures aged between 3–40 years indicate that, for the majority, loss of prestress has been less than predicted [203]. However, higher than estimated prestressing losses have been reported on a number of containment structures in France, the Russian Federation and the USA [73, 195, 203, 204]. Higher than expected prestressing losses were also experienced in other structures such as bridges [204]. Higher than expected losses of prestressing force in containment structures were because actual ambient temperatures exceeded design values [73].

Wire relaxation is a material property that depends on temperature. The higher the temperature, the higher the relaxation. In the USA, the main cause for lower than predicted prestressing forces has been identified as higher relaxation of tendon steel due to higher ambient temperature. The average temperature was measured to be about 32°C at the tendon locations, as opposed to 20°C that was considered in designs [204]. To a lesser extent, higher than estimated loss of prestressing was attributed to higher creep and concrete shrinkage strain.

In the Russian Federation, higher losses of the prestressing force were associated with higher tendon relaxation and creep strain of concrete [204]. In France, where grouted prestressing systems are used, low prestressing forces were attributed to creep and shrinkage of containment concrete.

Since containment is typically designed with large structural integrity margins, higher than expected loss of prestress does not necessarily jeopardize structural integrity. However, under accident conditions, margins are reduced and, therefore, there is a risk of cracking, which may result in leak rate increases.

The influence of elevated temperatures on prestress force losses, including tendon steel relaxation and concrete volumetric changes (i.e. creep and shrinkage), was discussed in Section 4.5.5.2. Besides temperature, other parameters such as relative humidity, size and geometry of the concrete member, concrete constituents and mix design influence the amounts and rates of creep and shrinkage, thus these parameters also influence prestressing force losses.

Information pertaining to concrete creep and shrinkage and their relationship to prestress loss in CCBs is available in Refs [93, 195, 197, 205].

Table 8 gives a complete review of stressors, degradation mechanisms and repair or mitigation methods.

4.6. SITES OF DEGRADATION

Potential degradation locations related to concrete, mild reinforcing steel, post-tensioning systems and liners are identified in Table 8. Further details surrounding ageing mechanisms for specific structures and components are provided below.

4.6.1. Containment penetrations

As containment deforms inelastically at high internal pressures, large loads may develop at penetrations. Inner and outer pipe supports near penetrations may fail if they are relatively weak. Alternatively, pipes may hinge or buckle, particularly for thin wall pipe with one or more elbows between penetration and support. This may result in a cracked or crimped pipe with reduced flow; however, at this accident stage it is likely no flow exists. If the pipe is heavy walled with a straight run to a strong support, sufficient load may be developed to fail the penetration. Failure may occur in the concrete sleeve anchor, the sleeve to closure plate weld or the closure plate to process pipe

TABLE 8. DEGRADATION FACTORS THAT CAN IMPACT CONCRETE STRUCTURE PERFORMANCE

Part (a) concrete material

Ageing stressors/environment	Degradation mechanisms	Ageing effect	Potentially critical parts/ locations	Condition monitoring and mitigation	Consequences of ageing
Percolation of fluid through concrete due to moisture gradient	Leaching and efflorescence	Increased porosity and permeability, lowers strength May indicate changes to cement paste, makes concrete more vulnerable to hostile environments	Structures exposed to water (e.g. subgrade structures, water intake, spent fuel bays) Near cracks, areas of high moisture percolation	Condition monitoring: • Visual inspection, periodic analysis of the fluid, testing of concrete samples Mitigation: • Minimizing percolation of fluid by diversion, if possible, using protective liners/coatings	Reduced load carrying capacity of the concrete over long term leaching Corrosion of steel reinforcement
Exposure to alkali and magnesium sulphates present in soils, sea water or groundwater	Sulphate attack	Expansion and irregular cracking	Subgrade structures and foundations	Condition monitoring: • Visual inspection, periodic analysis of water and soil chemistry, testing of concrete samples Mitigation: • Minimizing access of sulphates to the structure by diversion of the groundwater, if possible (e.g. improving drainage), waterproofing concrete • For new concrete: sulphate resistant cements or partial replacement of cements with supplementary cementing materials can be used to minimize potential occurrence	Progressive loss of strength and mass Reduction in load carrying capacity
Exposure to aggressive acids and bases	Conversion of hardened cement to soluble material that can be leached	Increased porosity and permeability	Local areas subject to chemical spills, adjacent to pipework carrying aggressive fluids	Condition monitoring: • Visual inspection, periodic analysis of water chemistry, testing of concrete samples Mitigation: • Minimizing access of acids and bases to the structure by improving protection of the concrete	Reduced load carrying capability Acid rain is unlikely to be an issue for containment structures

TABLE 8. DEGRADATION FACTORS THAT CAN IMPACT CONCRETE STRUCTURE PERFORMANCE (cont.)

Part (a) concrete material

Ageing stressors/environment	Degradation mechanisms	Ageing effect	Potentially critical parts/ locations	Condition monitoring and mitigation	Consequences of ageing
Combination of reactive aggregate, high moisture levels and alkalis	Alkali–aggregate reactions	Cracking, which may eventually lead to complete destruction of the concrete mass, gel exudation, aggregate pop-out	Areas where moisture levels are high and improper (reactive) materials utilized	Condition monitoring: • Visual inspection, periodic analysis of concrete humidity and testing of concrete samples Mitigation: • Minimizing concrete exposure to moisture by repairing the cracks and waterproofing concrete surface • For new concrete: eliminate potentially reactive materials; use low-alkali content cements or partial cement replacement	Reduced load carrying capacity
Cyclic loads/vibrations	Fatigue	Cracking, strength loss	Equipment/piping supports	Condition monitoring: • Visual inspection, monitoring of the loading, analysis Mitigation: • Needs to be taken into account during design stage • If necessary, implementing design changes to accommodate cyclic loadings and vibration	Typically localized damage, excessive deflection, brittle fracture
Exposure to flowing gas or liquid carrying particulates and abrasive components	Abrasion, erosion, cavitation	Continued loss of material and exposure of aggregate	Cooling water intake and discharge structures	Condition monitoring: • Visual inspection Mitigation: • Eliminate source and improve quality of concrete surface	Reduced load carrying capacity of concrete Increased resistance to flow in fluid systems

TABLE 8. DEGRADATION FACTORS THAT CAN IMPACT CONCRETE STRUCTURE PERFORMANCE (cont.)

Part (a) concrete material

Ageing stressors/environment	Degradation mechanisms	Ageing effect	Potentially critical parts/ locations	Condition monitoring and mitigation	Consequences of ageing
Exposure to thermal cycles at relatively low temperatures and high humidity	Freeze-thaw	Cracking, spalling	External surfaces where geometry supports moisture accumulation	Condition monitoring: • Visual inspection, testing of concrete samples Mitigation: • Minimizing concrete exposure to moisture by repairing the cracks and waterproofing the concrete surface • For new concrete: air entrainment used to minimize potential occurrence	Disintegration of concrete, reduction in load carrying capacity
Thermal exposure/thermal cycling	Moisture content changes and material incompatibility due to different thermal expansion values	Cracking, spalling, strength loss, reduced modulus of elasticity	Generally an issue for hot spot locations, near hot process and steam piping	Condition monitoring: • Visual inspection, periodic measurements of concrete surface temperatures Mitigation: • Controlling temperature of exposure, where possible, providing insulation to the concrete surface	Reduction of mechanical properties and load carrying capacity, can increase concrete creep that can increase prestressing force losses
Irradiation	Aggregate expansion, hydrolysis	Cracking, loss of mechanical properties	Structures proximate to reactor vessel	Condition monitoring: • Periodic measurements of level of radiation, testing of concrete samples Mitigation: • Controlling levels of radiation, where possible, providing insulation and shielding	Prolonged exposure may cause reduction of mechanical properties and load carrying capacity Typically, containment irradiation levels are below threshold levels to cause degradation

TABLE 8. DEGRADATION FACTORS THAT CAN IMPACT CONCRETE STRUCTURE PERFORMANCE (cont.)

Part (a) concrete material

Ageing stressors/environment	Degradation mechanisms	Ageing effect	Potentially critical parts/ locations	Condition monitoring and mitigation	Consequences of ageing
Consolidation or movement of soil on which the structure is founded	Differential settlement	Cracking	Connected structures or independent foundations	Condition monitoring: • Visual inspection, periodic settlement monitoring Mitigation: • Needs to be taken into account during design stage • If necessary, implementing design changes, such as strengthening	Excessive deflections, differential movement, misalignment of components and equipment
Exposure to water containing dissolved salts (e.g. sea water, road salt)	Salt crystallization	Cracking	External surfaces subject to salt spray, structures in contact with fluctuating water (e.g. water intake structures), areas where road salt is applied	Condition monitoring: • Visual inspection, testing of concrete samples Mitigation: • Minimizing access to the water through use of low permeability concretes, sealers and barriers	Progressive disintegration, reduction in load carrying capacity

TABLE 8. DEGRADATION FACTORS THAT CAN IMPACT CONCRETE STRUCTURE PERFORMANCE (cont.)

Part (b) containment liners

Ageing stressors/ environment	Degradation mechanisms	Ageing effect	Potentially critical parts/ locations	Condition monitoring and mitigation	Consequences of ageing
Electrochemical reaction with environment (metallic)	Composition or concentration cells leading to general or pitting corrosion	Loss of cross-section	Areas of moisture storage/ accumulation, exposure to chemical spills, or borated water near the interface where the liner is embedded in concrete	Condition monitoring: • Visual inspection, thickness measurements Mitigation: • Application of protective coatings and waterstops, minimizing humidity levels, where possible, repairs	Reduced leaktightness
Elevated temperature (metallic)	Microcrystalline changes	Reduction of strength, increased ductility	Near hot process and steam piping	Condition monitoring: • Visual inspection, periodic temperature measurements Mitigation: • Controlling temperature	Causes reduction in mechanical properties of steel, enhances corrosion Of significance only where temperatures exceed ~200°C
Irradiation (metallic and non-metallic)	Microstructural transformation (metallic), increased cross-linking (non-metallic)	Increased strength, reduced ductility	Structures proximate to reactor vessel	Condition monitoring: • Visual inspection, testing Mitigation: • Controlling radiation levels	Reduces ductility of steel; typically, containment irradiation levels are below threshold levels to cause degradation
Cyclic loading due to diurnal or operating effects (metallic and non-metallic)	Fatigue	Cracking	Inside surfaces of CCB	Condition monitoring: • Visual inspection, testing (for non-metallic) Mitigation: • Recoating (for non-metallic) • Repair where possible (for metallic)	Reduced leaktightness
Localized effects (non-metallic liners)	Impact loadings, stress concentrations, physical and chemical changes of concrete	Cracking	Inside surfaces of CCB Potential problem in high traffic areas	Condition monitoring: • Visual inspection, testing Mitigation: • Protect the liner during maintenance activities	Reduced leaktightness

TABLE 8. DEGRADATION FACTORS THAT CAN IMPACT CONCRETE STRUCTURE PERFORMANCE (cont.)

Part (c) mild steel reinforcing systems

Ageing stressors/ environment	Degradation mechanisms	Ageing effect	Potentially critical parts/ locations	Condition monitoring and mitigation	Consequences of ageing
Depassivation of steel due to carbonation or presence of chloride ions	Composition or concentration cells leading to pitting, general or micro-cell corrosion	Concrete cracking and spalling, loss of reinforcement cross-section	Outer layer of steel rebar in all structures where joints, cracks or local defects are present	Condition monitoring: • Visual inspection, testing of concrete samples, monitoring (e.g. half-cell potential, resistivity and polarization resistance) Mitigation: • Application of protective coatings; improving quality of concrete cover, cathodic protection	Prominent potential form of degradation, leads to reduction in load carrying capacity and serviceability
Elevated temperature	Microcrystalline changes	Reduction of yield strength and modulus of elasticity	Near hot process and steam piping	• Condition monitoring: Visual inspection, periodic measurements of concrete surface temperature Mitigation: • Controlling temperature of exposure, where possible, insulating concrete surface	Causes reduction in mechanical properties of steel, enhances corrosion Typically of significance only where temperatures exceed ~200°C
Irradiation	Microstructural transformation	Increased yield strength, reduced ductility	Structures proximate to reactor vessel	Condition monitoring: • Periodic measurements of level of radiation Mitigation: • Controlling levels of radiation, where possible, and providing insulation and shielding	Reduces steel ductility Typically, containment irradiation levels are below threshold levels that may cause degradation
Cyclic loading	Fatigue	Loss of bond to concrete	Equipment/piping supports	Mitigation: • Needs to be taken into account during design stage • If necessary, implementing design changes to accommodate cyclic loadings and vibration	Typically localized damage, brittle failures

TABLE 8. DEGRADATION FACTORS THAT CAN IMPACT CONCRETE STRUCTURE PERFORMANCE (cont.)

Part (d) post-tensioning systems

Ageing stressors/ environment	Degradation mechanisms	Ageing effect	Potentially critical parts/ locations	Condition monitoring and mitigation	Consequences of ageing
Localized pitting, general corrosion, stress corrosion or hydrogen embrittlement	Corrosion due to specific environmental exposures (e.g. electrochemical, hydrogen or microbiological)	Loss of cross-section and reduced ductility	Tendon and anchorage hardware of prestressed concrete containments	Mitigation: • Grease or grouts are used during construction to provide protection for corrosion • Good quality control during construction is essential. Grease can be re-introduced	Could cause breakage of post-tensioning system components resulting in reduction of prestressing force Lesser problem for bonded tendons as broken they re-anchor themselves in concrete
Elevated temperature	Microcrystalline changes	Reduction of strength, increased relaxation and creep	Near hot processes	Mitigation: • Controlling temperature of exposure	Thermal exposure not likely to reach levels that can produce ageing effects in prestressing material Larger than anticipated loss of prestressing forces
Irradiation	Microstructural transformation	Increased strength, reduced ductility	Structures proximate to reactor vessel	Mitigation • Controlling level of radiation	Reduction in mechanical properties of steel Typically, containment irradiation levels are below threshold levels to cause degradation
Cyclic loading due to diurnal or operating effects	Fatigue	Failure of prestressing under extreme conditions	Tendon and anchorage hardware of prestressed concrete containments	Mitigation: • Controlling loading	Brittle failure

TABLE 8. DEGRADATION FACTORS THAT CAN IMPACT CONCRETE STRUCTURE PERFORMANCE (cont.)

Part (d) post-tensioning systems

Ageing stressors/ environment	Degradation mechanisms	Ageing effect	Potentially critical parts/ locations	Condition monitoring and mitigation	Consequences of ageing
Long term loading	Stress relaxation, creep and shrinkage of concrete	Loss of prestressing force	Prestressed concrete containments	Mitigation: • Needs to be taken into account during design • Maintaining temperatures within design limits is necessary • Unbounded system cables can be retensioned, if needed • For bonded tendons, an appropriate monitoring system to measure structure strain can be implemented (e.g. embedded strain gauges and thermometers)	Larger than anticipated loss of prestressing forces

weld, or cause excessive shear and bending of the closure plate. Forged head penetrations are usually stronger than flat plate types, with failure of such penetrations usually limited to loss of the concrete anchor. While conditions leading to failure of a non-bellows-type pipe penetration are not a common occurrence, it has been identified as the controlling mode of failure for at least one containment [15].

4.6.2. Spent fuel pools

Age related degradation mechanisms that can produce leaks through stainless steel SFP liners include intergranular stress corrosion cracking and crevice corrosion. Leakage can also occur due to seam or plug weld defects, blockage of leakage collection systems by precipitation of reaction products or foreign material and liner damage. Leakage not captured by monitoring systems can migrate through construction joints and concrete cracks and result in environmental release of contaminated water. Figure 52 shows an example of a leak chase system and possible leakage path. When borated water bypasses such collection systems, there is an added concern that leakage over an extended period of time can result in concrete erosion due to the mildly acidic nature of borated water. This can also cause corrosion of any carbon steel it contacts (e.g. metallic pressure boundary or rebar). Exterior walls of spent fuel bays are often not accessible and cannot be visually monitored for degradation.

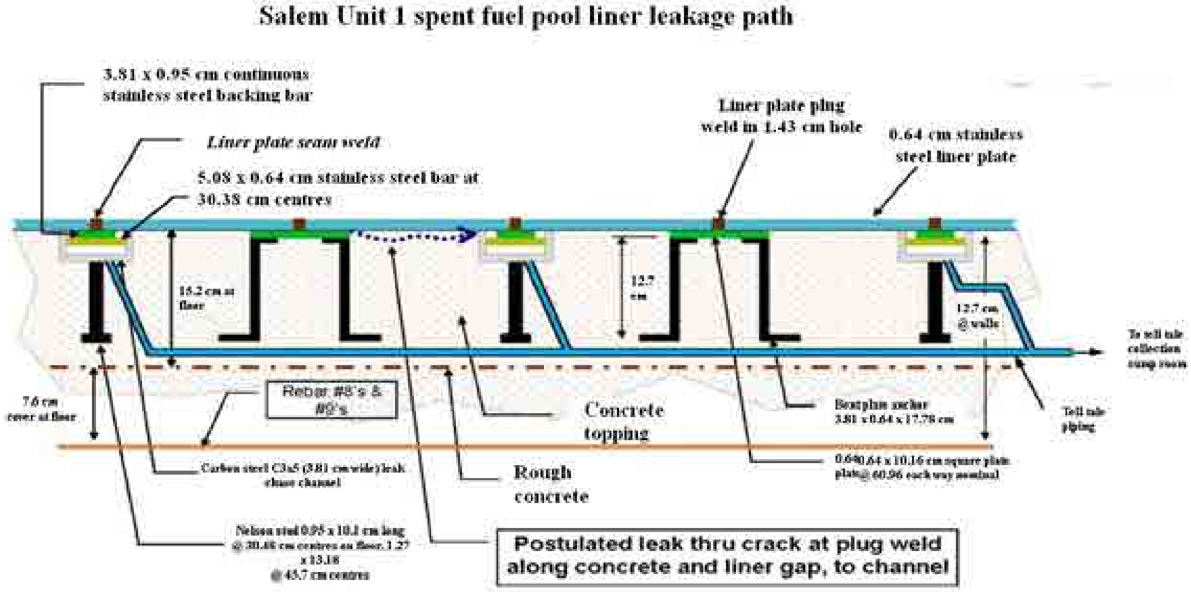


FIG. 52. Cross-section of leak chase system for a PWR and a postulated path for leakage occurrence (adapted from Ref. [17]).

SFPs with non-metallic liners can experience liner cracking, which causes concrete exposure to pool water. Degradation of non-metallic liners is covered in Section 4.5.3.2.

4.6.3. Cooling towers

Cooling towers are subject to degradation due to their relatively thin sections and the severe environments they experience (e.g. high humidity, strong winds, differential foundation settlement, freeze-thaw cycling and varying temperatures) [206, 207].

The primary degradation mechanisms for concrete cooling towers are freeze-thaw, ice formation, corrosion due to acidic water chemistry and corrosion from aggressive marine environments. Concrete carbonation and ASR can also contribute to general degradation but in general are not a cause of major damage in cooling towers.

Freezing and thawing of water in concrete causes extensive cracking and/or spalling resulting in loss of concrete integrity and an increase of permeability that allows water to reach and corrode rebar (see Section 4.5.2.1(b)). Ice formation in cooling towers occurs because these towers promote maximum contact between air and water. This is beneficial in the summer, but needs to be adequately monitored in winter. Although a certain degree of ice formation can be tolerated by concrete structures, ice formation can become unacceptable because of the extra weight load and physical impact of ice falling either on the plastic fill or on concrete surfaces. Varying degrees of rebar corrosion due to aggressive water occurs in many cooling towers, especially those using municipal water for cooling. In these cases, chloride laden water permeates through concrete to the rebar, corroding it and substantially decreasing the bearing capacity of reinforced concrete elements (see Section 4.5.2.2(d)). Cooling towers situated close to a marine environment can experience degradation because they use aggressive marine water for cooling. It is sometimes difficult to differentiate between aggressive water and an aggressive environment. However, cooling towers located in coastal areas often show severe degradation on the outer side of the shell (not in contact with the aggressive water) (see Section 4.5.2.2(f)).

Cooling tower degradation analysis needs to include the consideration of the following:

- Operating conditions (startup and shutdown cycles, thermal and drying gradients and fatigue cycles);
- Loading conditions (self-weight, wind loads, earthquake, temperatures, differential foundation settlements that can cause dynamic effects, creep and mechanical stresses);
- Early age behaviour (construction processes, such as slip forming leaving ridges on the concrete, construction defects and concrete drying).

4.6.4. Water intake structures

Water intake structures and structures exposed to lake or sea water undergo degradation primarily from aggressive marine water (see Section 4.5.2.2(f)).

4.6.5. Concrete pipe

Failure modes of concrete pressure pipes are as follows:

- Joint leaks;
- Poor quality mortar coating;
- Poor bedding;
- Excessive external loading;
- Corrosive environment;
- Construction damage (coating damaged and not repaired);
- Hydrogen sulphide (wastewater applications);
- Broken prestressing wires due to corrosion or poor material quality (PCCP only).

Prestressing wire breakage and its associated loss of prestress in PCCP pipe is initiated via corrosion or embrittlement of prestressing wires caused by any of the following:

- Aggressive external environments (corrosive/aggressive soils);
- Design deficiencies (improper coating, incorrect wire spacing, improper design/operation of cathodic protection);
- Manufacturing deficiencies (using high strength wire that is susceptible to embrittlement, using inferior mortar (highly porous or susceptible to ASR), installing closely wrapped wires causing delamination of mortar coating);
- Installation deficiencies (e.g. poor bedding, coatings damaged during construction);
- Poor pipe operation (surge events; improper hydraulic system design, operation and maintenance; excessive external loading (e.g. higher than design soil cover, live loads, dropped loads));
- Unaddressed joint leaks [208].

4.6.6. Masonry walls

Masonry block walls are subject to the same degradation mechanisms as reinforced concrete walls with the degradation mechanisms of interest being cracking (e.g. joint and through wall), durability of the masonry mortar and corrosion of metal components. Detailed information on the evaluation of the physical conditions of masonry units and masonry assemblages relative to environmental conditions is available in Ref. [209].

4.6.7. Structural anchorages

Anchors must transfer a wide variety of loads (e.g. tension, bending, shear and compression) to concrete. Several factors related to failure or degradation of anchorage systems include:

— Design detail errors, installation errors (improper embedment depth or insufficient lateral cover and improper torque);
— Material defects (low anchor or concrete strengths);
— Shear or shear-tension interaction, slip and preload relaxation [25].

Ageing mechanisms that could impair an anchorage's ability to meet performance requirements are primarily those that result in concrete deterioration (e.g. fatigue and vibration) because if a failure occurs, it will most likely initiate in the concrete. Corrosion can also impact anchorage performance.

4.6.8. Foundations, piles and underground structures

Primary age related degradation mechanisms that can impact concrete basemats include freeze-thaw damage, rebar corrosion, sulphate attack, leaching and AAR. Degradation of subfoundations made with porous concrete via cement erosion has also been reported [37].

For concrete or steel pile systems, excessive displacement after long term loading can cause settlement of supported structures. Differential settlement is an ageing concern. Age related degradation mechanisms that could potentially impact concrete piles are the same as noted for the basemat. Corrosion of structural steel piles, used in certain containment configurations for transferring loadings to greater depths below grade, is also a possible degradation mechanism. Similar to other containment steel, the concern for piles is from localized corrosion resulting in significant loss of cross-sectional area. One study [210] examined corrosion data from 43 piling installations of varying depths (up to 41.5 m) with times of exposure ranging from 7–50 years in a wide variety of conditions. The study concluded that the type and amount of corrosion observed in steel pilings driven into undisturbed soil, regardless of soil characteristics and properties, was insufficient to significantly affect the performance of pilings as load-bearing structures. However, pilings placed in oxygen enhanced fills, those exposed above grade, or those exposed to sea water, salt spray or underground aggressive water may be somewhat affected [211].

Outdoor electrical equipment related concrete foundations can suffer tilting, differential settlement or upheaving due to poor soil conditions, freeze-thaw impacts, ASR impacts and attack by chemicals such as road salt. Where steel is embedded, corrosion can be of concern due to impressed voltage differences or due to water, dirt or debris accumulation at the steel/concrete surface, or at the water ingress into the cementitious grout [20].

Primary age related degradation mechanisms for other underground concrete structures are essentially the same as for the basemat.

4.6.9. Waterstops, joint sealants and gaskets

Waterstops at concrete joints, joint sealants and gaskets between adjacent components are primarily constructed of metal or organic compounds and make a significant contribution to the leaktightness of containment buildings. During operation, organic components degrade by exposure to oxygen, ozone, microorganisms, ultraviolet radiation, elevated temperatures and stress. Each of these mechanisms acts to cause depolymerization, which manifests itself in increased hardness, increased brittleness, dimensional shrinkage, loss of adhesion and crack formation.

4.7. CONSEQUENCES OF AGEING DEGRADATION AND FAILURES

Typical consequences of ageing and failures related to concrete, mild reinforcing steel, post-tensioning system and liners are identified in Table 8. Additional consequences are dependent on structure function, which is not considered in that table. Major consequences can include leakage and/or loss of structural integrity. In the worst cases, these issues can have an impact on NPP licence extensions or continued plant operation. This may require action ranging from increased monitoring and trending up to and including mitigating measures such as extensive concrete repair, replacement or recoating. Some sample related OPEX is given in Table 8.

4.8. RESEARCH AND DEVELOPMENT RESULTS

Access to concrete related R&D is an important part of an NPP's AMP, since new developments may contradict assumptions made during a plant's design or operating stages.

Even though concrete has been extensively used as a construction material for 2000 years, its use in the nuclear industry is comparatively recent. There is a need to improve the understanding of processes affecting concrete ageing and deterioration in nuclear civil structures. In nuclear facilities, concrete may be subjected to stressors and lifetime expectations that are not typical for the structures that make up the majority of the available information on concrete ageing and deterioration.

The penetration of harmful substances, such as chlorides, is more complicated than can be described by ordinary diffusion laws. Approaches require careful consideration of the interactions between ions in solution and between spaces and solid surfaces inside concrete. The threshold values need further research regarding harmful substances that induce corrosion of rebar, or other durability limit states. The importance of understanding these phenomena increases when the operation or life of concrete structures exceeds the time periods covered by experimental research. The present state of knowledge is insufficient to define threshold values for the ageing management effects of radiation and high temperatures on concrete.

Sample research related to concrete at the time of this publication includes:

- Development of material and property databases;
- Using decommissioned plants to further compile material data and evaluate long term concrete performance in an NPP environment;
- Evaluation of long term effects of temperature and radiation;
- Developing damage models and acceptance criteria;
- Non-intrusive methods for inspection of heavily reinforced thick walled concrete structures and basemats;
- Inspection methods for metallic pressure boundary components including containment liner backsides;
- Utilization of reliability theory to address time dependence changes in structures to demonstrate operability and to estimate end of life;
- Applying probabilistic modelling of component performance to provide risk based criteria to evaluate ageing impacts on structural capacity;
- Determining impacts of refuelling cavity and spent fuel bay leakage on concrete and embedded steel.

4.9. OPERATING EXPERIENCE

4.9.1. Relationship to an ageing management programme

An effective OPEX review programme is essential to effective ageing management. OPEX needs to be actively sought from the following:

- The nuclear industry (internal to the company, utilities external to the company, plant owner groups, international collaboration programmes) [212, 213];
- International organizations, such as INPO, WANO and the IAEA, as well as organizations external to the nuclear industry (various associations engaged in concrete technology, construction and research).

Internal company OPEX, including materials and modelling developments, needs to be shared externally. Problems in similar plants may point to generic issues that need to be addressed.

4.9.2. Degradation experience related to concrete

In general, NPP CCB performance has been very good [80, 83, 214], however, many NPP concrete structures, including CCBs, have experienced degradation that has required remedial action. Ageing management practices and OPEX of several Member States are described in Appendix III. Appendix VI provides OPEX related to inspection and repair actions identified through a 1990s IAEA survey on concrete containment ageing [214].

Some initial problem areas that were identified were primarily related to material, construction or design errors. Examples of some specific problems that have been reported include:

— Low 28 day compressive strengths;
— Voids under prestressing tendon bearing plates resulting from improper concrete placement;
— Cracking of prestressing tendon anchorheads due to stress corrosion or embrittlement;
— Containment dome delaminations due to low quality aggregate materials and absence of radial rebar or unbalanced prestressing forces [215–220];
— Occurrence of excessive voids or honeycomb in the concrete;
— Contaminated concrete;
— Cold joints;
— Cadweld (rebar connector) deficiencies;
— Materials out of specification;
— Higher than code-allowable concrete temperatures;
— Exposure to freezing temperatures during concrete curing;
— Misplaced rebar;
— Prestressing system buttonhead deficiencies;
— Water-contaminated corrosion inhibitors.

Although not ageing related, delamination events have occurred during detensioning activities associated with cutting holes in containment to replace steam generators.

As structures age, incidences of degradation, primarily related to environmental effects, increase [215–220]. Examples of some problems include rebar corrosion in water intake structures, excessive containment building leakage, corrosion of prestressing tendon wires, leaching of tendon gallery concrete, low prestressing forces and leakage of corrosion inhibitors from tendon sheaths.

Other related problems include cracking and spalling of containment dome concrete due to freeze-thaw effects, rebar corrosion in cooling tower structures, tendon anchorhead failures due to stress corrosion cracking, cracking of shield building concrete due to freeze-thaw effects, corrosion of concrete containment liners and concrete cracking due to ASR. There have been instances of borated water leakage through seam and plug welds associated with SFP stainless steel liners that have permitted borated water to interact with concrete, potentially resulting in concrete erosion [221].

Appendix IV presents a summary of selected documented concrete problem areas in NPPs, as derived from the IAEA survey on concrete containment ageing [214], as well as certain significant OPEX identified publically since then.

4.9.3. IGALL ageing management plans

Proven ageing management plans for NPP systems, structures and components are being documented under the IAEA's IGALL project. For concrete and related civil structures, the following AMPs have been produced and can be used as the basis for the development of a plant specific AMP:

— AMP 301 In-service inspection for containment steel elements [222];
— AMP 302 In-service inspection for concrete containment [223];
— AMP 303 Safety class 1, 2 and 3 piping and metal containment components supports [224];

— AMP 304 Containment leak rate testing [225];
— AMP 305 Masonry walls [226];
— AMP 306 Structures monitoring [227];
— AMP 307 Water control structures [228];
— AMP 308 Protective coatings monitoring and maintenance programme [229];
— AMP 309 Non-metallic liners [230];
— AMP 310 Ground movement surveillance [231];
— AMP 311 Containment monitoring system [232];
— AMP 312 Concrete expansion detection and monitoring system [233];
— AMP 313 Containment prestressing system [234].

4.9.4. Lessons learned related to new concrete construction

Refer to Section 4.2.8.2.

4.9.5. Lessons learned related to concrete containment buildings

EPRI has summarized degradation research into CCBs [20]. The reference indicates that BWR concrete containments (except for Mark III containments) generally degrade through metallic liner corrosion, usually as a result of seal failure, which can normally be repaired before it causes concrete damage. BWR concrete containments (except Mark III) are not exposed to the outside environment. Degradation in PWR containments, however, is usually through environmental attack. During early plant life, concrete degradation can occurr immediately following plant construction [81]. Of these occurrences, about 30% are in containment buildings, with voids being the main problem. These voids are unintentionally left in concrete when the mortar could not fill spaces between coarse aggregates (honeycombs), or are caused by inadequate concrete placement and vibration. Both problems relate to poor design and construction, which are the the leading causes of age related degradation in concrete [235].

The most common occurrences of later degradation (up to 18 years of plant life) were concrete cracking and spalling [235, 236]. No degradation surveys were identified as having been carried out after approximately 18 years.

Assessing concrete structure degradation after a long period of operation is difficult because issues such as creep and embrittlement cannot be visually observed and reported visual degradation observations are usually symptoms of these phenomena. Many public and private institutions are researching ageing for concrete that is up to 80 years old. EPRI has prepared a materials degradation matrix of concrete structures in NPPs to guide and focus future R&D activities related to concrete ageing.[1]

Subjecting a concrete structure to stresses for which it was not designed can result in cracking. Replacement of large components, such as steam generators and pressurizers, in order to extend plant life, will often require an opening to be cut through the containment wall, and utilities need to predict the containment structure response. A notable example of a problem associated with such a replacement is the permanent shutdown of the Crystal River NPP following a number of delamination events [237]. Promoting awareness that concrete often ages without showing visual symptoms will encourage utilities to be more careful when subjecting structures to unexpected stresses.

Concrete degradation information summarized by EPRI is shown in Table 9, which highlights individual problems, their effects on structure life management and suggests ideas on how to solve these problems. Long term operation of existing NPPs will require more frequent inspection and maintenance.

[1] ELECTRIC POWER RESEARCH INSTITUTE, Program on Technology Innovation: Nuclear Concrete Structures Aging Reference Manual, Concrete Aging — Issues for Long-Term Operation of Nuclear Power Plants, Report 1023035, EPRI, Palo Alto, CA (2011).

TABLE 9. SUMMARY OF DEGRADATION OCCURRENCES IN CONCRETE CONTAINMENT STRUCTURES (*adapted from ref.* [20])

Problem	Description of problem	Potential impact in life management	What to do	Source (for EPRI)
Voids and honeycombs	Lower density of hardened concrete due to insufficient vibration Very thick and heavily reinforced sections	Facilitates water ingress, decreases structural integrity and strength Act as weak spot in cases of unexpected stresses	If structural integrity compromised, concrete section needs to be replaced	[238]
Efflorescence in concrete domes	Efflorescence on inner face of shield buildings — can only be observed in plants with an annulus	Corrosion of rebar with decrease in strength; Corrosion of metal liner	Follow best practice recommendation on sealing procedures	Site visits
Cracking and spalling in domes	Due to freeze-thaw in domes — seen on outer side of dome	Facilitate water ingress to liner and rebar	Benchmark how serious and generic the issue is	[9]
Cracking in tendon gallery	Groundwater seepage in underground portions of safety related structures	Rebar corrosion with a decrease in strength	Follow best practice recommendation on sealing procedures	Site visits [9, 238]
Anchors	Cracks in concrete around anchors	Decrease in prestressing force, anchor integrity compromised	Normally corrected during tendon inspection	[81]
Embrittlement, creep and fatigue	Embrittlement and creep due to ageing, forming microcracks More prevalent in post-tensioned containments	In case of unexpected stresses or water ingress, microcracks can coalesce more easily forming large cracks	Promote awareness that concrete ages Problem must be considered when new stresses are imposed to determine current structure state	[2]

4.9.6. Lessons learned related to spent fuel pools

Typical concrete steel liner interface and drainage channels are sketched in Fig. 53.

The concrete structure of SFPs is not in contact with pool water; however, stainless steel liner leaks through, for example, seam welds, plug welds or coating degradation, can result in acidic water coming in contact with concrete.

There are three main problems regarding SFP concrete degradation:

(1) Easier path for leakage: Construction joints and seismic gaps are similar to large pre-existing cracks in concrete and can offer easier leakage paths.
(2) Lack of support for liner: Acidic water, especially when it contains boric acid, dissolves cement paste, which erodes the concrete and, by reducing liner support, causes significantly larger leaks.
(3) Corrosion of the reinforcement: Acidic water can reach and corrode rebar, with a consequent decrease in structure strength [20].

SFP concrete structures in BWRs and PWRs have significantly different inspection and maintenance practices because of accessibility to the structures. Spent fuel concrete structures for BWRs can usually be accessed and leaks can be visually detected and contained. Concrete structures for PWR and PHWR SFPs were not designed to be inspected or repaired. Few pools have access to all four side walls so opportunities for visual inspections are limited.

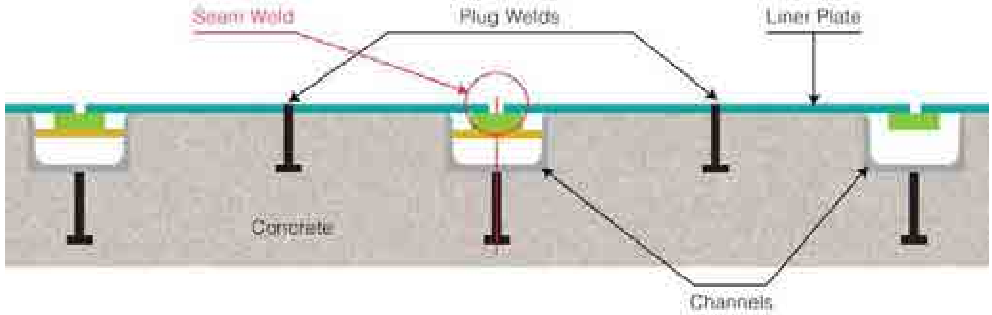


FIG. 53. Typical concrete liner interface with plug welds (fusion bonds can also be used).

Several cases of groundwater contamination have originated through construction joints and seismic gaps of SFPs. For that reason, it is recommended that utilities map construction joints under and surrounding SFPs so as to make inspections, monitoring and eventual leakage collection easier. It is recommended that site engineers have a thorough knowledge of this map.

Utilities generally manage and contain leaks by repairing liners rather than repairing the concrete structures. BWR and PWR or PHWR SFP leak repairs are normally performed by underwater welding, specialized epoxies or other fillers. Where liner repair is not possible, collecting leakage is an alternative. This is significantly easier in BWRs owing to accessibility from all sides of the structure. PWRs or PHWRs leak repairs are more challenging. The process includes digging, locating the leak and installing a collection system. Chronic leaks often necessitate a groundwater monitoring programme. EPRI has produced a guide on advanced electromagnetic inspection methods for fuel pool and transfer canal liners.[2]

4.9.7. Lessons learned related to concrete cooling towers

A number of events have precipitated cooling tower inspection or maintenance activities by utilities. An EPRI/NMAC report [21] describes that, although no natural draft towers have collapsed at any NPP, such towers have suffered collapse throughout the world. Notable collapses are as follows:

- In 1965, three out of a group of eight reinforced concrete natural draft cooling towers were blown down in strong winds at Ferrybridge Power Station in the UK.
- In 1973, a single 137 m tall natural draft cooling tower collapsed under moderate winds at Adeer Nylon Works power plant just off the southwestern coast of Scotland.
- In 1979, one natural draft tower at Bouchain in France collapsed under minimal winds. This tower was known to have had serious dimensional errors from the beginning. It is now suspected that the collapse could have been caused by progressive deterioration.
- In 1984, one natural draft tower under construction at the Fiddlers Ferry Power Station collapsed under high wind gusts in the UK leading to design changes in the tower.

Reference [239] states that, in addition to cooling towers that actually collapsed, there is on record a number that had deteriorated to the extent that they had to be demolished because they posed a very high risk of collapse. Examples include the French towers at Pont sur Sambre and Ansereuilles in France.

An INPO study published in 2008 [240] indicated that many nuclear stations have experienced or identified structural failures or deficiencies in their cooling towers. These structural problems affected unit operation and, in a few cases, could have led to personnel injury, although none occurred. The study describes an incident in 2007 when personnel at one station toured a mechanical draft cooling tower cell that collapsed the following day. The study goes on to say that although trends reflected some improvement between 2001 and 2006, the number of events, and resulting lost power generation, increased significantly in 2007. The study's conclusions pointed to an overreliance

[2] ELECTRIC POWER RESEARCH INSTITUTE, Advanced Electromagnetic Inspection Methods for Fuel Pool and Transfer Canal Liners, Report 1025214, EPRI, Palo Alto, CA (2012).

on visual inspections and supplemental personnel to detect problems, lack of station oversight, lack of station review of cooling tower OPEX and certain design deficiencies.

4.9.8. Lessons learned related to water intake structures

Many NPPs are situated near the coast and use sea water as a coolant that passes through a series of reinforced concrete channels at both the inlet and outlet sides. These channels are a unique type of marine structure and should ideally be given special consideration when assessing corrosion risk. It has been found that corrosion risk to these structures is high wherever turbulence occurs or where flow levels are high. This increased risk is the result of higher rates of oxygen diffusion to the rebar. A cooling channel cannot be considered to be completely submerged since structure outside surfaces (i.e. culvert or pipe) may be surrounded by dry ground or may be exposed to the environment. Reinforcement near outer and inner concrete surfaces will normally be electrically in contact, with the former acting as a cathode. Corrosion may occur where there is an imbalance in electrochemical potential, at locations where rebar intersects or at construction features that interrupt concrete cover uniformity. Construction joints between structural elements, such as between floors and walls or walls and roof slabs, are areas at particular risk. Experience in Scandinavia has shown that steel rebar corrosion risk in homogeneous concrete is high if cover thickness is less than 30 mm and chloride levels exceed 0.2% total chloride ion content expressed as a percentage of concrete weight.

A visual inspection of marine structures can be effectively carried out provided concrete surfaces are thoroughly cleaned, preferably by high pressure water. Corrosion under water will not manifest itself in a manner normally associated with atmospheric corrosion (i.e. cracking and spalling of cover concrete). Corrosion products for underwater corrosion tend to be less expansive, partly due to the leaching effects of flowing water. Corrosion tends to occur as a point attack resulting in severe local steel pitting. This may be evidenced by rust staining and hard crusts of corrosion products on the concrete surface. By relating patterns of corrosion points to rebar lay out and structural features, such as position of spacers and construction joints, it is possible to form a fairly accurate picture of the extent of significant corrosion damage.

An EPRI study [20] identified that in the USA, the main degradation sources in cooling intake structures were aggressive water conditions (high chloride content or low pH water that initiates rebar corrosion and subsequent cracking and loss of strength) and biofouling (i.e. the presence of shell mussels and other organisms attaching themselves to concrete structures). Because biofouling organisms can seriously affect mechanical equipment, this problem requires constant attention. Most degradation in water cooling systems was identified in coastal plants. Differential settlement was also seen as a cause of cracking and concrete deterioration in plants with underwater intake pipes.

4.10. INSPECTION/MONITORING/MAINTENANCE HISTORY

A data collection and records keeping system is an important part of an AMP. Sections 4.10–4.13 of NS-G-2.12 [1] provide details on typical contents (e.g. failures, malfunctions, maintenance actions and decisions, and operating conditions). Inspection, monitoring and maintenance history are key attributes to be recorded, as they allow for trending of degradation over time, assessment of past degradation remediation and prediction of future performance.

Some plants have continuous measurement methods/sensors installed, either as part of initial installations or retrofits. More information on available instrumentation is covered in Section 7.3.2. These instruments can provide real time information on structure performance to allow timely detection and action to address degradation. During construction, these systems can provide information regarding initial prestressing losses in order to verify design assumptions and to provide a baseline for future measurements.

4.11. MITIGATION METHODS

It is important to detect ageing related deterioration and to mitigate negative effects before loss of integrity, interference with operations or structural failure occurs.

As described previously in Table 1, effective AMPs need to include mitigating methods for the effects of ageing mechanisms. Methods to mitigate these effects include:

— Maintenance methods and practices, and condition, monitoring, including refurbishment and periodic replacement of parts and consumables;
— Operating conditions and practices that minimize ageing rates;
— Possible changes in design and materials of the component.

For concrete structures, mitigation strategies are typically aimed at minimizing the negative effects of the ageing degradation mechanism, as well as monitoring and controlling, within acceptable limits, parameters that structures or components are exposed to, such as temperature, radiation level and water and soil chemistry.

As water is a necessary component for many chemical and physical ageing mechanisms (i.e. the alkali–aggregate reaction, leaching, freeze-thaw and corrosion), waterproofing or diverting water away from the structure or component are considered to be effective mitigation strategies. Cathodic protection can also be applied in some circumstances to mitigate corrosion of steel rebar, concrete pipe prestressing wire or other components embedded in concrete.

Ageing mechanisms such as fatigue, settlement and loss of prestressing force are typically conservatively accounted for in designs. If such mechanisms are detected, design changes and modifications may be required to maintain margins.

Refer to Table 8 for mitigation methods for identified ageing degradation mechanisms, and Section 8 for repair strategies.

4.12. CURRENT STATUS, CONDITION INDICATORS

Knowledge of the current status of concrete structures is essential for assurance of continued safe operation of NPPs, and is typically gained via assessments against documented condition indicators. Condition indicators can be established for detecting, monitoring and trending structure ageing degradation. Some structure and safety class dependent condition indicators related to concrete structures are:

— Integrated leak test leakage rates;
— Spent fuel pool leakage rates;
— Ingress of harmful substances (e.g. chloride content of concrete);
— Depth of concrete carbonation;
— Prestressing force;
— Structural movement/deflections;
— Cracking (extent and magnitude), spalling, delamination and corrosion products.

Some further sources of condition indicators and their assessment include ACI 349.3R [241], an assessment checklist published by EPRI [242] and a CSA guide for evaluating concrete structures impacted by alkali–aggregate reactions [243].

Limited access to parts of structures can often make direct measurements difficult. Approaches typically taken consist of monitoring the environment and the accessible adjacent structures with similar environmental conditions. For example, foundations are frequently evaluated by monitoring groundwater and soil for harmful substances (such as chlorides, sulphates and low pH, among others) to indicate degradation potential.

5. DEVELOPMENT AND OPTIMIZATION OF ACTIVITIES FOR AGEING MANAGEMENT OF CONCRETE STRUCTURES

Generic attributes of an effective AMP are defined in Section 2. Components of an effective AMP typically include items such as:

— Structure description and functional requirements;
— Design standards;
— Baseline condition (results of condition assessment);
— Examination methods and frequency;
— Acceptance criteria;
— Mitigation methods;
— Personnel qualifications;
— Documentation requirements.

Regulatory codes and standards are described in Section 5.1. Section 5.2 describes gathering plant data, performing condition assessments, developing, coordinating and optimizing an AMP.

5.1. REGULATORY REQUIREMENTS, CODES, STANDARDS AND SAFETY CRITERIA

Codes, standards and rules generically refer to laws, decrees, nuclear regulations and nuclear and conventional (industrial) codes and standards that are required to be applied to an NPP. The structure of these rules is plant or country specific and AMPs need to include requirements imposed by national regulators. Reinforced concrete structures at NPPs are designed, constructed, operated and inspected in accordance with national consensus codes and standards (e.g. Refs [101, 102, 244–249, 199, 250–256]). Many codes and standards are listed in Section 18 of IAEA Technical Training Series No. 17 [257]. These rules were developed over time by experienced people and are based on knowledge acquired in testing laboratories and supplemented by field experience. Differences in codes and standards occur from country to country due to different approaches to ensuring plant safety. Although all codes and standards tend to be conservative, they are typically generic in nature and provide minimum requirements. Additional requirements for specific structures, components or environments are typically defined in technical specifications.

Table 10 provides a listing of several national codes, standards and documents for design, construction, inspection and repair of concrete structures and describes how they relate to ageing management. In general, the approach has been to address ageing management indirectly.

5.1.1. Requirements for applicable codes, standards and safety criteria

Codes and standards applicable to ageing management are typically the same as those used in design. Obsolescence management programmes need to provide guidance on and monitor changes in standards and regulations over time (see Ref. [1], para. 5.5).

For containment structure design, widely accepted codes and standards need to be used (see Ref. [29], paras 3.25 and 3.26). The selected codes and standards:

— Need to be applicable to the particular design concept;
— Need to form an integrated and comprehensive set of standards and criteria;
— Should ideally not use data and knowledge that is unavailable in the host State, unless such data can be analysed and shown to be relevant to the specific design and the use of such data represents a safety enhancement for the design.

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES

Origin	Document	Commentary relative to ageing management considerations
Belgium	ASME Section III, Rules for Construction of Nuclear Facility Components [258]	Requires stresses in reinforcing steel to be limited
	ASME Section XI [250] Subsection IWE, Requirements for Class MC and Metallic Liners of Class CC Components of Light-Water Cooled Plants	Requires visual inspection of liner
	ASME Section XI [250] Subsection IWL, Requirements for Class CC Concrete Components of Light-Water Cooled Plants	Requires visual inspection of concrete and examination of post-tensioning systems
	USNRC Regulatory Guide 1.90, Inservice Inspection of Prestressed Concrete Containments with Grouted Tendons [259]	Defines the instrumentation for monitoring prestress level
Canada	REGDOC-2.6.3, Fitness for Service – Aging Management [260]	Replaced RD-334, Aging Management for Nuclear Power Plants [261]. Sets regulatory requirements and provides guidelines for managing ageing of SSCs of an NPP. Requires integrated ageing management plans and SSC condition assessments.
	CSA-N287.1, General Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, 1993 (R2009) [244]	Ageing addressed through considerations for quality assurance and verification to obtain durable concrete
	CSA-N287.2, Material Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, 2008 [245]	Ageing addressed through provision of requirements for materials used in containment structures for obtaining durable concrete structure
	CSA-N287.3, Design Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, 1993 (R2009) [102]	Ageing addressed through provision of design considerations
	CSA-N287.4, Construction, Fabrication and Installation Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, 2009 [246]	Ageing addressed through provision of requirements for construction fabrication and installation for obtaining durable concrete structure
	CSA-N287.5, Testing and Examination Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, 2011 [247]	Ageing addressed through specification of tests and examination activities during construction to ensure concrete quality
	CSA-N287.6, Pre-operational Proof and Leakage Rate Testing Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, 2011 [248]	Pre-operational proof and leakage rate test is intended to confirm design assumptions, performance of structure as constructed and to identify leak paths (if any), thus reducing potential for ageing degradation

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
Canada (cont.)	CSA-N287.7, In-Service Examination and Testing Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, 2008 [249]	Ageing addressed through in-service inspection/monitoring/testing to ensure concrete leaktightness and integrity of concrete, prestressing system and liner
	CSA N290.3, Requirements for the containment system of nuclear power plants (in preparation) [262]	Being written as a general standard for containment systems of NPPs, and will establish nuclear safety design, procurement, installation and testing requirements to control and minimize radioactive releases. The publication is being developed in response to the lessons learned from the Fukushima accident to specifically address: post accident design pressures, venting, H_2 gas concentration control requirements and multi-unit stations/events.
	CSA N291, Requirements for Safety Related Structures for CANDU Nuclear Power Plants, CSA, 2008 (R2013) [263]	Ageing addressed through in-service inspection/monitoring/testing to ensure integrity of safety related structures and components
	CSA-A.23.1/A23.2, Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete, CSA, 2009 [95]	Ageing addressed through provision of requirements for concrete, its constituents and construction considerations as well as through specification of tests/acceptance criteria to ensure concrete durability
	CSA-A.23.3, Design of Concrete Structures, CSA, 2004 (R2010) [264]	Ageing addressed through provision of design considerations
	CSA-A864-00 (R2005), Guide to the Evaluation and Management of Concrete Structures Affected by Alkali-Aggregate Reaction [243]	Provides guidance on addressing specific degradation mechanisms

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
European Union	EN 1990, Basis of Structural Design	The informative annexes of EN 1990 describe methods to use to consider the effect of design life on partial safety factors
	EN 1991 1, Actions on Structures	In material specific design codes, ageing and design life are mainly covered under the title durability
	EN 1992 2, Design of Concrete Structures [265]	Standards related to protection, repair and strengthening, address ageing through guidelines for design and execution of maintenance
	EN 1993 3, Design of Steel Structures	Execution and production standards reference ageing indirectly through quality requirements
	EN 1994 4, Design of Composite Steel and Concrete Structures	
	EN 1995 5, Design of Timber Structures	
	EN 1996 6, Design of Masonry Structures	
	EN 1997 7, Geotechnical Design	
	EN 1998 8, Design of Structures for Earthquake Resistance	
	EN 1999 9, Design of Aluminium Structures	
	EN 1504, Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity. Parts 1 to 10. [266–275]	
	EN ISO 12696, Cathodic Protection of Steel in Concrete (see ISO entry in this table)	
	EN 206-1, Concrete – Part 1: Specification, Performance, Production and Conformity [276]	
	EN 13670, Execution of Concrete Structures	

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
Finland	YVL 4.1, Concrete Structures for Nuclear Facilities, 1992	Ageing management of nuclear facilities is a special topic in regulatory guides (YVL) published by STUK
	YVL 1.8, Repairs, Modifications and Preventive Maintenance at Nuclear Facilities, 1986 [277]	
	YVL A.8, Ageing Management of a Nuclear Facility [278]	
	YVL B.6, Containment of a Nuclear Power Plant [279]	
	YVL E.6, Buildings and Structures of a Nuclear Facility draft 2012	
	SFS-EN 1992 and its National Annexes, Design of Concrete Structures	
	SFS-EN 1990 and its National Annex, Basis of Structural Design	
	SFS-EN 1991 and its National Annexes, Actions on Structures	
	SFS 7022, Concrete, Application of Standard SFS-EN-206-1 in Finland	
	SFS 5975, Execution of Concrete Structures, Application of Standard SFS-EN-13620 in Finland [280]	
	SFS-EN 1504, Products and Systems for the Protection and Repair of Concrete Structures. Definitions, Requirements, Quality Control and Evaluation of Conformity. Parts 1 to 10.	
	SFS-EN ISO 12696, Cathodic protection of steel in concrete (see ISO entry in this table)	
	ASME Boiler and Pressure Vessel Code, Section III, Division 2, Code for Concrete Containments [281]	
France	RCC-G Design and Construction Rules for Civil Works of PWR Nuclear Islands [256] (AFCEN)	Specifies durable materials
	Règles de conception et de construction du génie civil des îlots nucléaires REP (technical code for PWR reactor civil works)	Describes monitoring devices needed to monitor containment buildings during leak rate tests and throughout plant lifetime
	All French reactors except EPR design	
	ETC-C EPR Technical Code for Civil Works (part 1 is dedicated for design and is mainly based on European ENs) [255]	Specifies durable materials
		Describes monitoring devices needed to monitor containment buildings during leak rate tests and throughout plant lifetime
		Takes into account environmental conditions and defines exposure classes and limits to crack opening

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
France (cont.)	ENs (and standard NF EN 206-1)	Takes into account environmental conditions and defines exposure classes
	France Fascicule 65 du Cahier des Clauses Techniques Générales des marchés publics de travaux (Technical rules for public civil works)	Specifies choice of materials that avoid or limit the consequences of a process (steel corrosion, delayed ettringite, AAR) or situations (freeze-thaw)
	Recommendations for the prevention of damage by the alkali–aggregate reaction, Laboratoire central des ponts et chaussees (LCPC)	Specifies the choice of materials that avoid or limit the consequences of an AAR
	Recommendations for preventing disorders due to delayed ettringite formation, LCPC	Specifies the choice of materials that avoid or limit consequences of delayed ettringite
	Annales de l'ITBTP N° 517 d'octobre 1993: Surveillance des ouvrages atteints d'Alcali-Réaction (monitoring of buildings submitted to alkali aggregate reaction)	
	Association française de genie civil: Collection of documents and professional rules dealing with ageing management to be taken into account at the design phase (performance of materials), during operation phases (maintenance and repair): — Corrosion; — Delayed ettringite formation; — Alkali–aggregate reaction.	
Hungary	Government Decree No. 118/2011. (VII. 11.) on nuclear safety requirements of nuclear facilities and related regulatory activities, and nuclear safety authority procedures for nuclear facilities	Requires comprehensive review of ageing management of passive and long life system components in a licence request, inspector to review ageing management activities, ageing to be included in periodic safety assessments
	N.B.SZ Guides, 1.26, Surveillance of Ageing Management	Describes nuclear authority needs connected to ageing management documents and describes authority activity, describes content of AMPs
India	Design and Construction of Nuclear Power Plant Containment Structures (in preparation), Atomic Energy Regulatory Board	Long term strain measurements using embedded gauges, determination of tendon forces and changes in force, in-service surveillance to identify any ageing related degradation

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
India (cont.)	Rules for Construction of Nuclear Power Plant Components, Section III, Division 2, Rules for Construction of Nuclear Facility Components, ASME, 1992 [281]	See commentary under entry for the USA.
	CAN/CSA-N287 Series: CSA N287.1 [244], CSA-N287.2 [245], CSA N287.3 [102], CSA N287.4 [246], CSA N287.5 [247], CSA N287.6 [248], CSA N287.7 [249]	See commentary under entry for Canada.
	French Code RCC-G, Design and Construction Rules for Civil Works of PWR Nuclear Islands, Electricité de France, 1988 Edition [256]	Instrumentation for long term structural monitoring, requirements for structural integrity and leakage rate tests
International Standards Organization (ISO)	ISO 16311 series on maintenance and repair of concrete structures: Part 1: General principles [282] Part 2: Assessment of existing concrete structures [283] Part 3: Design of repairs and prevention [284] Part 4: Execution of repairs and prevention [285]	Covers maintenance and repair of all kinds of concrete structures, un-reinforced and reinforced concrete, prestressed concrete and steel concrete composite structures or their structural members
	ISO 12696 Cathodic protection of steel in concrete [286]	Not ageing specific, but covers performance requirements for cathodic protection of steel in concrete for both new and existing structures (buildings and civil engineering structures, including normal reinforcement and prestressed embedded reinforcement)
Japan	JSME S NE1–2011, Codes for Nuclear Power Generation Facilities — Rules on Concrete Containment Vessels for Nuclear Power Plants [252]	Provides material and design for concrete containment vessels
	JSE Guidelines for Concrete No. 15, Specifications for Concrete Structures, 2007 “Design”, Japan Society of Civil Engineers [287]	Provides material and design for general concrete structures
Russian Federation	NP-017, General Requirements to Service Life Extension of NPP Power Units	Establishes general criteria and requirements for evaluation of the possibility of NPP power unit service life extension and for actions for safety assurance during the prolonged life period
	STO 1.1.1.01.006.0327-2008, NPP Service Life Extension	Determines procedure, sequence and performance time of life extension work Determines non-recoverable and irreplaceable power unit elements, which need lifetime extension

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
Russian Federation (cont.)	STO 1.1.1.01.007.0281-2010, Lifetime Management of Elements of NPP Power Units	Establishes procedure of lifetime management of elements of NPP power units
	RD EO 0447-03, Methods for Estimation of Condition and Estimation of Residual Lifetime of NPP Reinforced Concrete Constructions, Important for Safety	General methods for service life extension of reinforced concrete constructions, important for safety
	RD EO 0538-2004, Methods for Justification of Service Life of Containment of NPP with ВВЭР-1000	Establishes procedure for work performance of estimation of the technical conditions and service life of WWER-1000 NPP containment
	RD EO 0462-03, Methods for Justification of Service Life of Building Constructions, Buildings and Structures of NPP with RBMK	Establishes procedure for work performance and estimation of technical conditions and service life of buildings and structures of high-power channel-type reactors (RBMKs)
	RD EO 1.1.2.99.0007-2011, Standard Operating Procedure for Operation of NPP Factory Buildings and Structures	Instruction covers factory buildings and structures located on NPP sites, including operation and maintenance
	RD EO 1.1.2.99.0624-2011, Monitoring of NPP Building Constructions	Establishes general rules for monitoring performance and estimation of the technical conditions and of service life of building construction
	RD EO 0129-98, Requirement for Repair and Maintenance of Containment Prestressing System of NPP with WWER-1000 and with Reactor System V-320	Establishes general rules for the organization of maintenance, and repair of containment prestressing system for WWER-1000 NPPs. Also establishes general rules of work planning of equipment serviceability checks, putting into operation and requirements, which are used for technical maintenance.
	RD EO 0130-98, Requirement for Repair and Maintenance of Containment Prestressing System of NPP with WWER-1000 and with Reactor System 302, 338, 187	Establishes general rules of maintenance and repair organization and repair of containment prestressing system for WWER-1000 NPPs. Also it establishes general rules of planning of serviceability checks, repairs, putting into operation, and requirements for equipment, which are used for technical maintenance.
Switzerland	Concrete – Part 1: Specification, Performances, production and conformity – National Elements NE for SN EN 206-1:2000 [288] EN replaces former national standard SIA162, Concrete Structures [289]	Designer to consider expected environmental conditions, particular protective measures and likely maintenance to ensure a durable structure. Exposure classifications based on defined environmental conditions.

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
United Kingdom	EN 2 replaces all national codes dealing with the design of structural concrete (such as BS 8110, BS 8007, BS 5400 in the United Kingdom). All the parts of ENs relevant to the design of concrete have been published, and BS 8110 is no longer being supported by the relevant British Standards Institute committee.	Designer to consider expected environmental conditions, particular protective measures and likely maintenance to ensure a durable structure. Exposure classifications based on defined environmental conditions.
	British Standard 4975, Prestressed Concrete Pressure Vessels for Nuclear Engineering, British Standards Institution [101]	Guidance to ensure vessel is operated in manner such that anticipated loadings are not exceeded; covers serviceability aspects, monitoring, inspection and reporting
	The Tolerability of Risk from Nuclear Power Stations, The Health and Safety Executive [290]	Provides guidelines on the tolerable levels of individual and social risk to workers and the public from nuclear power stations
	Safety Assessment Principles for Nuclear Plants, The Health and Safety Executive [291]	Provides the framework against which technical judgments on the Licensees' written demonstration of safety (the safety case) is made, includes principles that deal specifically with the need for adequate through-life inspection and maintenance procedures
	Licence Condition Handbook, The Health and Safety Executive [292]	Requires licensee to make and implement adequate arrangements for the regular and systematic testing, examination, inspection and maintenance of all plant structures that may affect safety (licence condition 28)
	Pressurized Systems and Transportable Gas Container Regulations, The Health and Safety Executive, 1989 [293]	Requires a written scheme of examination to be drawn up and endorsed by a structurally qualified, competent person
	The Structural Use of Reinforced Concrete in Buildings, code of practice 114 [294], and The Structural Use of Prestressed Concrete in Buildings, code of practice 115 [295], British Standards Institution	Design codes in force at time of design and construction of early vessels; concrete mix specifications required by designers typically exceeded code requirements to take account of durability aspects
	Rules for Construction of Nuclear Power Plant Components, Section III, Division 2, Code for Concrete Reactor Vessels and Containments, ASME [281]	Design and construction rules for PWR containment with revisions to definitions and specification of an ultimate load requirement to make consistent with BS4975
	Rules for the In-service Inspection of Nuclear Power Plant Components, Section XI, Code for Concrete Reactor Vessels and Containments, ASME [250]	1995 code provided basis for inspection procedures adopted for Sizewell B PWR prestressed concrete containment vessel

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
United States of America	Code of Federal Regulations, Title 10, Energy	
	Part 50, Domestic Licensing of Production and Utilization Facilities [296]	
	§ 50.65, Requirements for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants [297]	Intended to provide basis to ensure the implementation of an effective programme to manage and to mitigate the effects of ageing
	Appendix A to Part 50, General Design Criteria for Nuclear Power Plants [298]	Ageing not addressed directly, but requires containment to be designed to permit periodic inspection of important areas and an appropriate surveillance programme
	Appendix J to Part 50, Primary Reactor Containment Leakage Testing for Water Cooled Power Reactors [299]	Requires general visual inspection and leak-rate tests at specified intervals, results can be used for trending
	Part 54 – Requirements for Renewal of Operating Licenses for Nuclear Power Plants [34]	Provides methodology for determination of need for an ageing management review of long lived structures
	ACI/American Society of Civil Engineers (ASCE)	
	ACI 318, Building Code Requirements for Reinforced Concrete [96]	Ageing addressed through concrete cover and water to cement ratios to protect rebar from corrosion
	ACI 201.1R, Guide for Making a Condition Survey of Concrete in Service [300]	Provides terminology to perform and report on visual condition of concrete in service (e.g. checklist for conduct of inspections and visual examples of types of imperfections and distress)
	ACI 349, Code for Requirements for Nuclear Safety Related Concrete Structures [251]	Ageing addressed through requirements for concrete cover and water to cement ratios to protect steel rebar from corrosion
	ACI 349.3R, Evaluation of Existing Nuclear Safety Related Concrete Structures [241]	Provides recommended guidelines for use in evaluation of existing safety related concrete structures
	ASCE/SEI 11/99, Guideline for Structural Condition Assessment of Existing Buildings [209]	Provides guidelines for assessing the structural conditions of existing buildings constructed of combinations of material including concrete, masonry, metals and wood

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
United States of America (cont.)	ASME	
	Section III, Rules for Construction of Nuclear Power Plant Components, Division 2, Code for Concrete Reactor Vessels and Containments [258]	Ageing approach similar to that of ACI 318
	Section XI, Rules for Inservice Inspection of Nuclear Power Plant Components [250]	Requires visual and post-tensioning system inspections at prescribed intervals, general performance of post-tensioning system with time
	NRC Regulatory Guides, NUREG Reports, Inspection Manual Chapters (IMC), Information Notices (IN) and Inspection Procedures (IP)	
	RG 1.107, Qualification for Cement Grouting for Prestressing Tendons in Containment Structures [301]	Ageing not addressed directly. Describes method that NRC considers acceptable for use of Portland cement grout as a corrosion inhibitor for prestressing tendons in prestressed CCSs. Also provides quality standards for using Portland cement grout to protect prestressing steel from corrosion.
	RG 1.18, Structural Acceptance Test for Concrete Primary Reactor Containments [302]	Ageing not directly addressed, provides acceptable method for structural acceptance test to demonstrate capability of concrete primary reactor containment to withstand postulated pressure loads (does not cover leakage through containment)
	RG 1.35, Inservice Inspection of Ungrouted Tendons in Prestressed Concrete Containments [303]	Provides a procedure acceptable to the NRC for inspection of ungrouted tendons
	RG 1.35.1, Determining Prestressing Forces for Inspection of Concrete Containments [72]	Provides a means for determining prestressing force levels to be used for in-service inspections
	RG 1.188, Standard Format and Content for Applications to Renew Nuclear Power Plant Operating Licenses [35]	Provides the format and content for applications to renew an NPP operating licence
	RG 1.90, Inservice Inspection of Prestressed Concrete Containments with Grouted Tendons [259]	Describes an approach that NRC staff considers acceptable for use in developing an appropriate surveillance programme for prestressed CCSs with grouted tendons
	RG 1.136, Design Limits, Loading Combinations, Materials, Construction and Testing of Concrete Containment [304]	Ageing not directly addressed; provides information on NRC position on acceptability of use of particular ASME Code cases

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
United States of America (cont.)	RG 1.127, Inspection of Water-Control Structures Associated with Nuclear Power Plants [305]	Describes basis acceptable for developing appropriate in-service inspection and surveillance programmes for dams, slopes, canals and other water control structures associated with emergency cooling water systems or flood protection of NPPs
	NUREG-1800, Standard Review Plan for License Renewal Applications for Nuclear Power Plants [71]	Provides guidance to NRC staff reviewers to ensure quality and uniformity in performing safety reviews for NPP operating licence renewals in accordance with 10 CFR Part 54
	NUREG-1801, Generic Aging Lessons Learned (GALL) Report [306]	Contains NRC staff generic evaluation of existing plant AMPs and the technical basis for determining where existing programmes are adequate without modification and where they need to be augmented for extended period of operation
	IMC 2515, Light-Water Reactor Inspection Program – Operation Phase [307]	Provides guidance to NRC staff for review and inspection activities to ensure safe operation of light water reactors, including post-approval site inspections associated with licence renewal
	IMC 2516, Policy and Guidance for License Renewal Inspection Programs [308]	Provides guidance to NRC staff regarding review and inspection activities associated with licence renewal
	IN 99-10, Degradation of Prestressing Tendon Systems in Prestressed Concrete Containments [73]	Issued to alert addressees with respect to degradation of prestressing system components of prestressed concrete containment in several areas, such as: (a) Prestressing tendon wire breakage; (b) The effects of high temperature on the prestressing forces in tendons; (c) Trend analysis of prestressing forces.
	IP-71002, License Renewal Inspections [309]	Provides procedures for inspecting and verifying the documentation, implementation and effectiveness of the programmes and activities associated with an applicant's licence renewal programme
	IP-71003, Post-Approval Site Inspection for License Renewal [310]	Provides procedures for inspecting and verifying completion of licence renewal commitments and licence conditions added as part of the renewed licence, and ensuring that selected AMPs are implemented in accordance with renewal regulations

TABLE 10. SELECTED STATE PRIMARY REGULATIONS, CODES AND STANDARDS RELATED TO DESIGN, CONSTRUCTION AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS AND OTHER STRUCTURES (cont.)

Origin	Document	Commentary relative to ageing management considerations
United States of America (cont.)	Industry reports	
	NEI 95-10, Industry Guidelines for Implementing the Requirements of 10 CFR Part 54, The License Renewal Rule [311]	Guidance document providing an acceptable approach for implementing requirements of 10 CFR Part 54 and how to perform a licence renewal assessment for a plant and what information to submit in a licence renewal application
	NUMARC 90-01, Pressurized Water Reactor Containment Structures License Renewal Industry Report [82]	Provides the technical basis for licence renewal of US pressurized water reactor containment structures
	NUMARC 90-10, Boiling Water Reactor Containments License Renewal Industry Report [312]	Provides the technical basis for licence renewal of US boiling water reactor containment structures
	NUMARC 90-06, Class I Structures License Renewal Industry Report [22]	Provides technical basis for licence renewal of NPP Class 1 structures in the United States of America
	American Water Works Association	
	AWAA Manual M9, Concrete Pressure Pipe [23]	Ageing not specifically addressed; covers recommended practices for design, installation and testing of concrete pressure pipe
	NACE International	
	NACE International standard SP0100 (former RP0100) Cathodic protection to control external corrosion of concrete pressure pipelines and mortar coated steel pipelines for water or wastewater service [30]	Addresses ageing via guidelines for corrosion control of PCCP through application of cathodic protection

Codes and standards have also been developed by various national and international organizations, covering areas such as:

— Materials;
— Manufacturing (e.g. welding);
— Civil structures;
— Pressure vessels and pipes;
— Instrumentation and control;
— Environmental and seismic qualification;
— Pre-service and in-service inspection and testing;
— Quality assurance;
— Fire protection.

Safety criteria, including availability, operability or reliability requirements as documented in plant safety analyses or other documents, are other structure requirements to be met, and may not be defined in codes and standards critical to NPP safety.

5.1.2. Hierarchy of applicable codes and standards

Codes and standards are applied as part of a hierarchy in individual countries, with the top level being national implementing legislation. An example of a hierarchy from a European source [253] using five levels is shown in Fig. 54, and is described further below.

5.1.2.1. Level I legislation

Laws and regulations are mandatory. This group includes codes and regulations applied to the plant by competent authorities, that is, the applicable legislation in each country. Level I rules also include any specific requirements of a country's nuclear regulator.

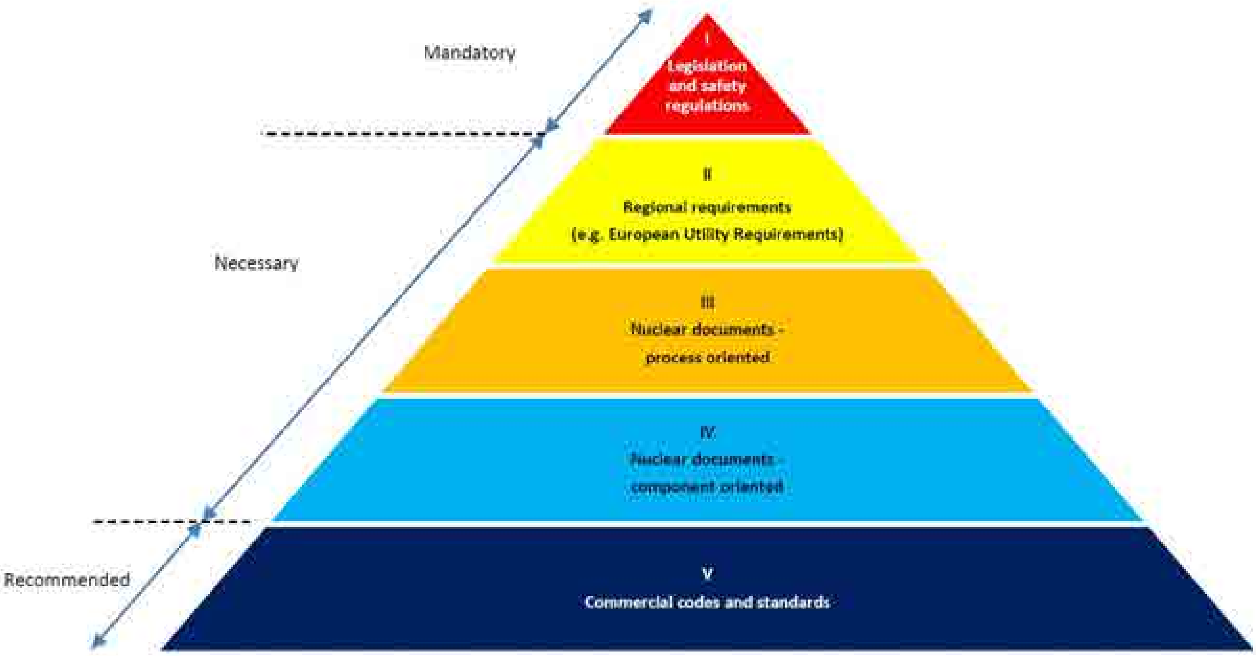


FIG. 54. Levels of rules (adapted from Ref. [253]).

5.1.2.2. Level II European utility requirements

European utility requirements contain up to date requirements for new light water reactor power plants. They are located high in the requirements applicability structure hierarchy as they indicate the methodology for selecting nuclear documents, codes and standards. This does not mean they supersede Level III nuclear documents. European utility requirements are applicable only in some countries, and their application is not compulsory.

5.1.2.3. Level III nuclear documents, process oriented

This level includes nuclear documents (including codes and standards) developed in various countries specifically for nuclear facilities. It applies to the most important safety aspects of the facilities and establishes adequate requirements for SSCs. Examples of such rules in various jurisdictions are BS 5882, BTP 9.5.1, CSN, DIN (NKe), EN-ISO-9000 9001 9002 9003, ETC-S, KTA 1401, KTA 1500, KTA 2200, 3205, KTA 2201, Letter SIN 3110/84 (RCC-P), RG 1.26, RG 1.29, RG 1.60, RG 1.143, RSK, 10 CFR 50 Appendix A and B, and CSA.

5.1.2.4. Level IV nuclear documents, component oriented

SSCs important to safety are designed according to standards approved in the nuclear industry. As part of the design process, the scope and applicability of standards to be applied need to be determined. Examples of the most relevant rules are ACI 349, AFNOR, ANSI, ASME CC, ASME III Div. 2, ASME VIII, BS 5500, BS 8110, DIN, EN-ISO-29000, ETC-C, ETC-M, FEM, IFB, ISO, CSN, KTA 1500, KTA 2200, KTA 2201, RCC-M/C, RG 1.29, RG 1.143, RSEM, RFS V.2.h, SNE-GS3.01 and CSA.

5.1.2.5. Level V conventional codes and standards

This level includes codes and standards usually applied to any SSCs in conventional facilities and in non-safety class areas of nuclear facilities. National standards in European Union countries are generally adaptations of European standards (standards for structural design called EN).

Design of safety related structures can be based on the EN, so it may be one of the regulation systems of the Level IV category. The European Pressurized Reactor/Evolutionary Power Reactor (EPR) Technical Code for Civil Works (ETC-C [255]), contains rules for design, construction and testing of EPR civil engineering structures [254]. It includes principles and requirements for safety, serviceability and durability conditions for concrete and steelwork on the basis of EN design principles, together with specific provisions for safety classified buildings.

5.1.3. Primary rules for concrete structure design, construction and inspection

Table 10, contains a list of primary rules for concrete structure design, construction, inspection and repair as required by various national jurisdictions.

5.1.4. Eurocode standards

Countries in the European Union are adopting the principles of structural design found in the EN standards, which are published by the CEN. ENs are reference design codes and contain references to some nationally determined parameters. To implement ENs, European Union Member States publish national standards containing specific values of nationally determined parameters. This means that national standards that are based on ENs are not exactly the same. This process, however, leads to more uniform standards of structural design. As special structures such as NPPs are beyond the ENs' scope, their application to nuclear facilities may require national licensing authorities to issue separate national requirements and guidelines.

ENs include standards for concrete structure construction and standards dealing with concrete and reinforcing steel materials, tendons, prestressing kits and precast concrete products. Figure 55 presents an overview of the system of EN standards for concrete work. In European Committee for Standardization member countries, EN standards have the status of a national standard, but on a national level, special requirements are often given as national annexes.

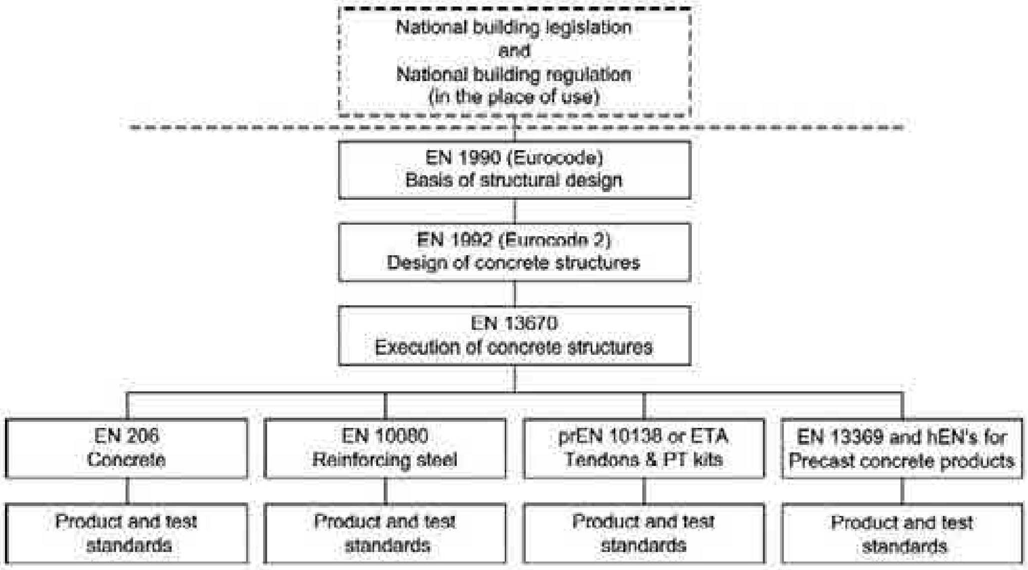


FIG. 55. System of European standards for design, execution and material for concrete works [280]. The dashed line indicates national standards that invoke the EuroNorm and similar standards; prEN is proposed EuroNorm; ETA is European Technial Approval; PT is post tensioning and hEN is harmonized EuroNorm.

European standards for protection and repair of concrete structures are described in the EN 1504 suite of standards, which consist of ten parts (see Refs [266–275]). EN 1504 Part 9, released in 2008, describes general principles for concrete protection and repair [274].

In EN 2 [265], ageing is mainly treated as durability. Chemical and physical exposure classes are among the environmental conditions defined in EN 206-1 [276]. Concrete quality and composition need to be suitable for the exposure class defined. Measurable ageing design values are primarily concrete cover and allowable crack width. Concrete cover is given as a function of exposure class and structural class. In EN 2, six structural classes are defined, based on service (design) life (50 or 100 years), structure type, concrete strength, concrete production quality control and exposure class. The exposure class also influences the width of allowable cracks. The recommended values for allowable cracks range from decompressed up to 0.4 mm. Recommended values for concrete cover range from 10–65 mm [265].

5.1.5. International Organization for Standardization standards

ISO standards on concrete protection and repair methods are available in four parts under ISO 16311 (Refs [282–285]).

An ISO standard (ISO 19338) on performance and assessment requirements for structural concrete design [313] is available, but is not nuclear specific. Other non-nuclear specific ISO standards are available on such topics as:

- — Concrete testing (ISO 1920 series);
- — Production (ISO 12439 for mix water requirements, ISO 14824 for prestressing tendon grout requirements, ISO 16204 for durability verification, ISO 22965 Parts 1 and 2 for specifying required concrete and its constituent materials, ISO 22966 for execution (construction, pouring and installation process) of concrete structures);
- — Simplified design (ISO 15673, 28841 and 28842);
- — Non-traditional reinforcing materials (ISO 10406 and ISO 14484).

5.2. PROGRAMMES AND ACTIVITIES

An AMP includes documentation of relevant programmes and activities (in particular, inspection and monitoring requirements and appropriate acceptance criteria) and the description of mechanisms used for programme coordination and continuous improvement. The documentation of programme activities, coordination and improvement methods are covered in this section, while acceptance criteria are discussed later in Section 7.4.1.

Various NPP and external programmes and activities contribute to concrete structure ageing management. These include NPP operating procedures and practices affecting structure service conditions and the environment to which structures are exposed, as well as inspection, monitoring, testing, condition assessment and maintenance activities. OPEX feedback and relevant research programmes also are important.

Safety authorities of Member States increasingly require licensees to define AMPs for selected SSCs by documenting relevant established programmes and activities, their respective roles in managing SSC ageing and their coordinating mechanisms. A typical process for this (where an AMP does not previously exist) involves gathering plant documentation, performing a baseline condition assessment (CA), and developing and documenting an AMP based on the condition assessment results. It is important that these steps in setting up and implementing an AMP, as well as interpreting its results, are performed under the direction of a person with appropriate engineering knowledge and experience.

5.2.1. Gather plant documentation

Details of concrete structure design, construction, commissioning and operational history including performed inspections and repairs are required for effective comparison against design basis and external experience. Experience from ageing assessments shows that the data listed in Table 11 is of value. Sources include plant records and architect/engineer or technical service organization/consultant files. Data may be grouped into four types: baseline; construction and commissioning; operational history and inspection and surveillance.

(a) 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 degredation and gives insight into impacts any degradation might have on functional or performance requirements. Design documentation may also include details of provisions made for ensuring long term concrete structure integrity (e.g. dealing with creep effects) or of design limits (e.g. minimum prestressing 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.
(b) Construction and commissioning data enable the review of concrete material quality and workmanship. Experience with both nuclear and non-nuclear engineering structures has shown that the most frequent cause of failure is poor quality construction or design error including improper material selection, with symptoms often evident at the earliest stages of structure life. It is important to identify locations where there were non-conformances with design or construction, and where repairs or modifications were necessary.
(c) Operational history data provide information on concrete loads and actual environmental and operating conditions and need to be compared to the original design basis to check for non-compliance. Operational data are particularly valuable for detailed assessments of potential future impacts of degradation mechanisms.
(d) Inspection and surveillance data provide historical information on actual structure and environmental conditions, and are a baseline against which ongoing performance can be evaluated. This is valuable in trending degradation progress, and in determining whether design assumptions for environmental conditions (e.g. maximum room temperatures and water chemistry, among others) 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 remedial measure effectiveness.

Components of civil structures (e.g. walls, floors, doors and foundations, among others) are increasingly being entered into NPP databases using specific equipment codes or tags. This allows for easier recording and retrieval of design, maintenance, modification and repair information specific to the applicable structures.

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

Ensuring that data are placed in a special file or database (i.e. making it readily accessible) will save considerable time when evaluating structure condition. A systematic and consistent approach will allow for easy comparison among similar plants, and will assist in identifying data gaps. Practical guidance on the implementation of an effective system for data collection and record keeping for the purpose of ageing management is given in Ref. [314]. An example of such a system is a service life management system developed in Finland for NPP concrete structures [315]. This system aims at producing information related to the condition of safety classified concrete structures. It needs to predict the need and timing of structure maintenance and repair based on the different ageing mechanisms that are present.

5.2.2. Perform condition assessment

A key element of ageing management is the systematic and rigorous assessment of structures. Ageing assessments generally involve a review of plant data to assess the effect of age related degradation on structures, to establish their current condition and to provide predictions of future performance. Gathering of plant data is described in Section 5.2.1.

Condition assessment is a commonly recognized form of ageing assessment and is typically applied to structures or groups of structural components with similar characteristics. Condition assessment involves a general review of design, manufacturing, construction, commissioning, operations, inspections and maintenance at a component level.

Another form of ageing assessment, life assessment, is typically applied to critical and complex structures that are designed to not be replaced as part of a normal maintenance programme and that are subject to long term degradation mechanisms. Life assessment involves a detailed review of design, manufacturing, construction, commissioning, operations, inspections and maintenance at a subcomponent level.

Typically, condition assessments:

— Establish structure baseline conditions (to which results of the AMP's periodic inspections, testing and monitoring will be compared);
— Identify ARDMs that may cause degradation;
— Identify critical structural components and areas most susceptible to degradation;
— Provide health prognosis.

Baseline condition assessment allows for optimum use of existing structure information, prioritization of ageing management activities, and assists in the systematic collation of structure records. This helps to focus more detailed subsequent activities. The AMP can then be optimized to concentrate efforts on critical areas, and to use inspection and monitoring techniques that are appropriate for the suspected degradation. Condition assessment results enable schedule optimization for periodic inspections by prioritizing structures and components in terms of safety significance, environmental exposure and anticipated degradation tolerance. Refer to Section 5.4 for further details on AMP optimization.

As stated in NS-G-2.12 [1], an ageing management review is an integral part of a condition assessment. Thus, in addition to current condition and future health predictions, condition assessment is used to identify changes necessary to address issues related to ageing effects, and may include economic improvement opportunities. As a result of condition assessments, recommendations are provided to ensure structure integrity and the capability to satisfy design requirements until the end of lifetime. This includes any planned life extensions and requirements during decommissioning. The ageing management review process is illustrated in fig. 4 of NS-G-2.12 [1].

Condition assessment needs to be performed to establish the baseline condition of the structure prior to setting up an AMP (described in Section 5.2.3). Some utilities perform or update condition assessments periodically, in conjunction with periodic safety reviews as described in NS-G-2.10 [316]. The guide states that defining and updating knowledge of the actual condition of SSCs is necessary to satisfy periodic safety review objectives. Condition assessments are typically performed in preparation for life extensions. In Canada, for example, a condition assessment is required to be completed as part of the ageing review for long term operation beyond the assumed service life of the plant. As defined in the Canadian regulatory document REGDOC-2.6. [260], condition assessment also forms part of the integrated AMP framework.

A typical approach to performing a condition assessment is as follows:

- Gather and review design, manufacturing, construction, commissioning, operation, inspection and maintenance history, and any other relevant documentation.
- Define design basis and identify any structure changes during design, construction and operation.
- Establish the structure's physical and functional boundaries and associated components.
- Undertake an ageing assessment of structures or components remaining after screening. The screening process is outlined in fig. 3 of the Safety Guide NS-G-2.12 [1]. This includes assessing ARDMs, evaluating ageing and establishing structure health prognosis.
- Provide recommendations for each structure in terms of repair and monitoring, as well as maintenance activities to ensure plant safety and production goals over the plant's service life.

A recommended approach is to first list all structure components (e.g. ring girders, walls and dome), their function, materials of construction, environmental exposure and applied loads. A generic list of components, such as presented in Ref. [317] for light water reactors, may simplify this activity. Of particular importance is the identification of those containment components and other concrete structures that play key roles in maintaining safety or other important functions. For each of these components, identify and document:

- Operating limits or performance criteria defined in the design.
- Potentially significant degradation mechanisms. Significance is primarily measured in terms of potential impact on the structure's ability to perform its safety functions. Also relevant is the ability to detect degradation, and the potential costs of remedial action (refer to Section 4).
- Any known degradation, based on evidence from commissioning tests and available surveillance data.

Additional details related to component identification are provided in Refs [318] and [319].

A condition assessment's depth and quality is highly dependent upon the qualification and experience of the personnel performing it, and on the quality of the gathered information. In addition to a review of documents and drawings, interviews with site personnel, suppliers and constructors, among others, may help to identify information that was not formally documented. Walkdowns and photographic records are essential as they help to identify locations of distress and possible deterioration of a structure. Specifically, in the absence of historical data on plant condition, baseline inspections are necessary to gain plant familiarity and to assess current conditions. This includes a preliminary visual inspection of all accessible areas and involves the following:

- General condition survey of key structural elements, with an emphasis on detection of likely ageing mechanism symptoms.
- Knowledge of practical or safety related limitations to access those that will impose restrictions on inspection strategies. Plant operation cycles may lead to changing access conditions.
- Identification of elements or materials to be selected for detailed study.
- Discussion with operators on facility history, maintenance history and other related information.

Based on a review of available information, design and operational exposure conditions of a structure are also compared. Engineering judgement is typically used to document all potential ageing mechanisms that might influence performance of the structure being assessed. The evaluation process shown in Fig. 56 can be used to assess and prioritize ARDMs. A matrix similar to Table 12 is typically created to record evaluation results.

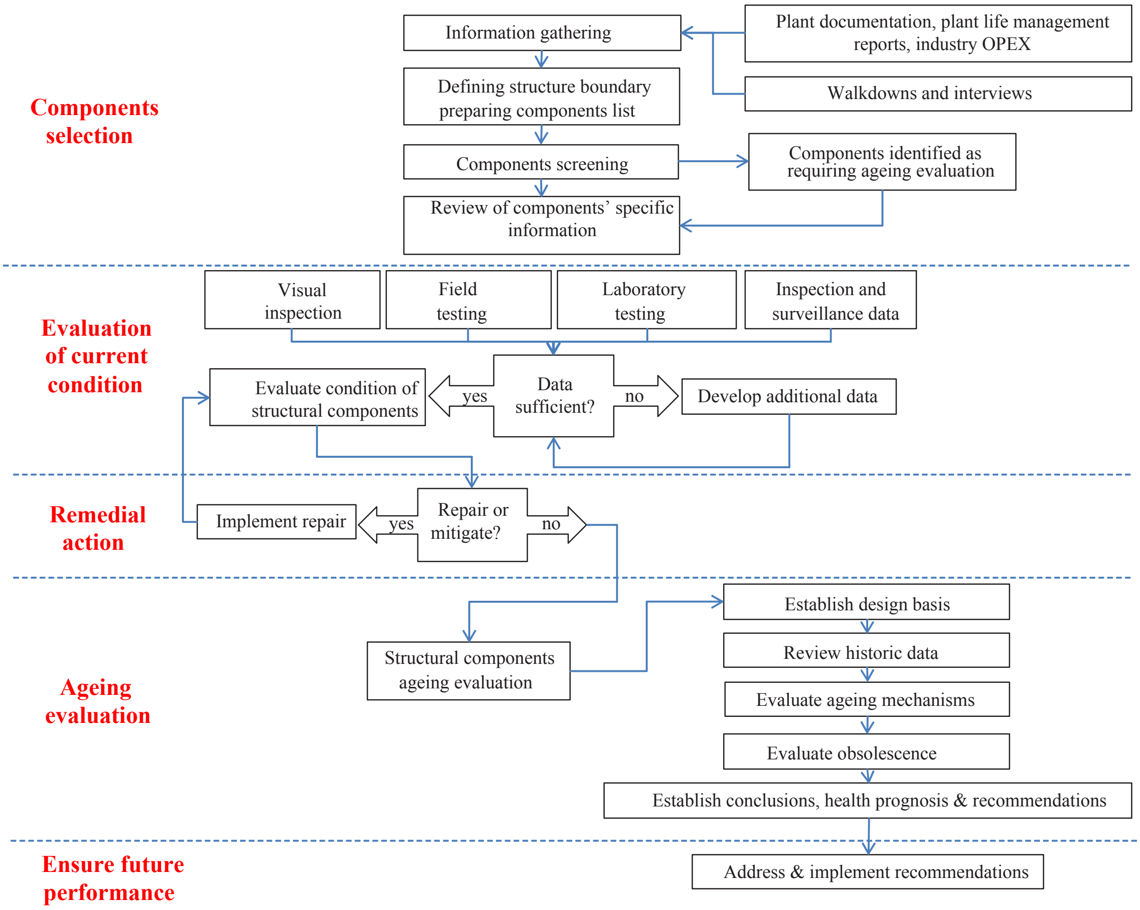


FIG. 56. Condition assessment process (adapted from Ref. [320]).

TABLE 12. EXAMPLE OF AN AGEING RELATED DEGRADATION MECHANISM MATRIX

ARDM	Concrete	Reinforcing steel	Steel components	Sealing compound
Chemical attack	Low	—	—	—
Fatigue/vibration	—	—	Low	—
Thermal exposure/cycling	Medium	—	—	Medium
Irradiation	Low	Low	Low	Medium
Corrosion	—	Medium	Medium	—

Structure susceptibility to each ARDM is evaluated as high, medium or low based on the following criteria (see also Fig. 57):

— High: The degradation mechanism is occurring or has occurred, either in this structure or in a similar structure under similar conditions. Steps have not 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.
— Low: The degradation mechanism is possible for the structure, but is easily managed with current programmes or has no impact on achieving target life.

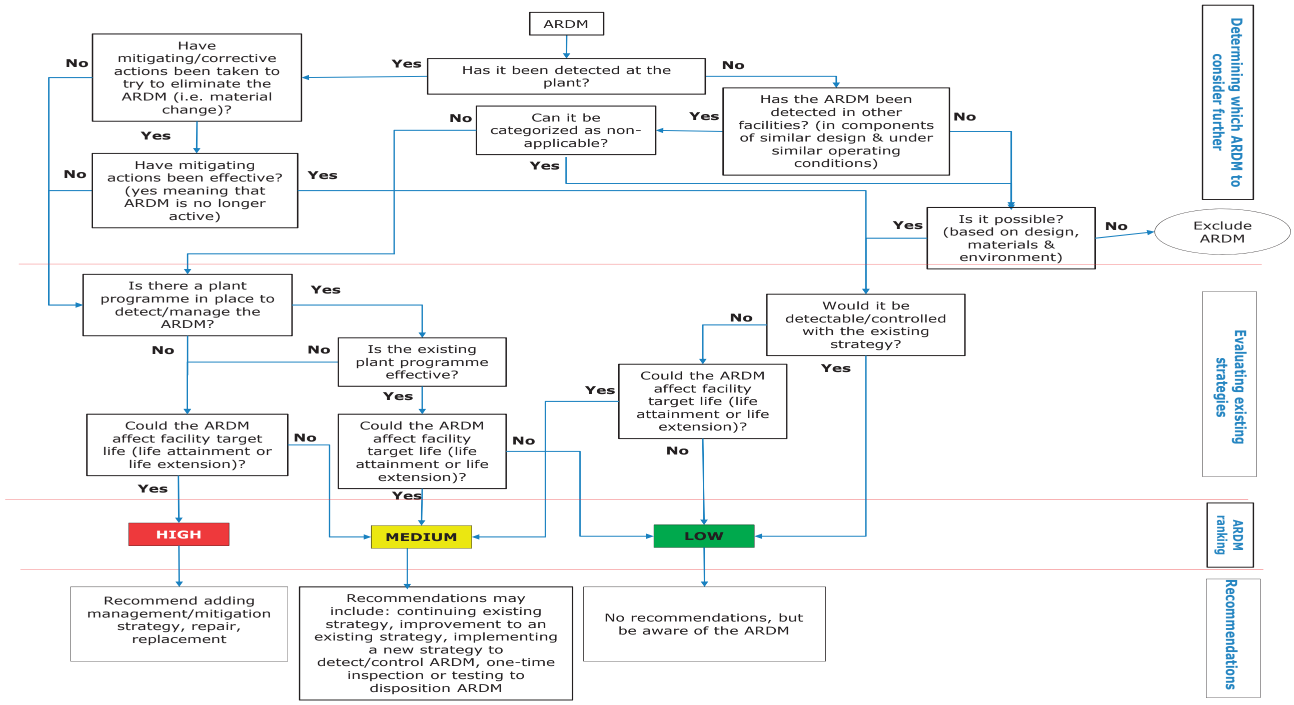


FIG. 57. Assessment of ARDMs (adapted from Ref. [320]).

ARDMs not included in the table, or those having no marks against them in the table, are considered unlikely for the structure given the environmental conditions or materials, or a combination thereof, and are not considered further.

Conclusions and health predictions can be provided based on a thorough review of available information, with particular attention to ageing related degradation. Where sufficient information to provide a health prognosis is not available, activities to support condition assessment results are typically recommended. Additionally, recommendations related to existing plant programmes are typically provided to ensure continuing structure health.

5.2.3. Develop ageing management programme

Following a baseline condition assessment, an AMP can be developed. Examination methods, frequencies and locations need to be defined. Appendix III contains some sample approaches by different countries, and Table 1 in Section 2 defines the generic elements of an effective programme. The programme needs to be integrated with the overall management system for the NPP, including corrective action elements [7, 8].

The examination methods used are broadly comparable and predominantly based on a combination of visual inspections, leakage rate tests and checks on tendon loads and corrosion (post-tensioned structures). Where instrumentation exists, results from automated monitoring have proven effective in demonstrating continued compliance with design. Where appropriate, the results from integrated leakage rate tests demonstrate the overall leaktightness of a containment building. Some utilities use detailed interim inspections of selected areas, or components within containment, to confirm their satisfactory condition and reduce periodic examination frequencies to, for example, ten year intervals.

As degradation often appears through visible indications on exposed concrete surfaces, the general condition of concrete is often assessed using visual inspection by knowledgeable and experienced personnel. Visual features identified during inspection may be related to workmanship, structural serviceability and material deterioration. 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.

For ease of plant access and reduced radiation and temperature levels, inspections related to the interior portions of CCBs are generally performed during outages. Interior portions of containment structures common to multiunit stations (e.g. the vacuum building for CANDU plants) require multiunit outages. Other structures, such as water intakes, also have access restrictions. Inspection intervals or frequencies are initially determined based on requirements contained in codes and standards (see Refs [249] and [321]). Early inspections provide confidence in design and construction quality. Through-life inspections provide valuable data for trending structural behaviour, giving confidence in ageing performance.

Selected locations may include known defects (which are monitored to check activity), or critical locations identified during the baseline condition assessment (see Section 5.2.2). Examples of areas that may be considered for more detailed inspection include:

— Tendon anchorage components;
— Intersection of dome and walls;
— Intersection of basemat and walls;
— Polar crane attachments to containment wall;
— Below grade wall surfaces.

More detailed examinations may also involve routine use of non-destructive techniques to provide quantitative data for trending behaviour of critical areas (see Section 7.2). For example, regular monitoring of half-cell potentials may be carried out if there is a threat of rebar corrosion.

Appropriate examination frequencies will be a function of accessibility and the likely rate of degradation and its potential consequence. Reference [214] indicates that a wide range of frequencies exist for conducting in-service examinations and leakage rate tests (e.g. leakage rate tests were conducted at intervals ranging from weekly to once every ten years). Frequencies also may be varied through plant life, perhaps increased if examination is being carried out as part of an evaluation programme for an observed defect, or reduced following evidence of stable (predictable) behaviour. Appropriate frequencies need to be assessed using engineering judgment, with the overall aim of ensuring detection of degradation before minimum performance requirements are reached.

For inaccessible concrete areas, greater reliance is placed on condition characterization through indirect monitoring (e.g. differential settlement of attached structures, or quantifying the severity of environmental stressors). This is often supported by a more detailed assessment of design, construction and operational history to demonstrate that significant degradation is unlikely.

In addition to concrete surfaces, tests are carried out on other containment system components. Examples include tests of prestressing tendon integrity and performance and leaktightness of seals and liners. In addition, inspection of active systems, such as cathodic protection or external dewatering systems, need to be included as part of a containment's AMP.

Acceptance criteria and considerations for repair, as addressed in Sections 7 and 8, need to also be included in the AMP.

Evaluation is largely based on engineering judgement. The personnel involved in the AMP need to be knowledgeable and experienced in evaluating structures (material degradation and structural integrity) and be familiar with design and functional requirements of nuclear concrete structures. Personnel qualification requirements need to be defined in the AMP.

Keeping clear records of all examinations, tests and mitigations will ensure a better understanding of ageing trends, which provides vital information to assist in controlling and mitigating their effects.

After each periodic examination, test, or repair, the following information needs to be documented:

— Scope of the activity;
— Methods and materials used;
— Personnel involved;
— Examination results and trends;
— Details of repair including quality assurance.

If needed, drawings need to be updated to include modifications to maintain current as-built drawings. The information needs to be retrievable during the life of the plant and during the decommissioning period.

5.3. COORDINATION MECHANISMS

Different organizational units at an NPP have important roles to play within a successful AMP. Experts from operations, engineering, equipment qualification, design and R&D need to participate in the ageing management team. Other NPP programmes can also play key roles, for example, a fire barrier inspection programme can also look at concrete in the same areas. Programme overlaps and gaps need to be understood and addressed. The publication NS-G-2.12 [1] recommends documentation of policies and objectives of the AMP, designation of an ageing management coordinator and documentation of team responsibilities and interfaces.

5.4. PROGRAMME EFFECTIVENESS

Section 5.2.3 discussed using interim inspections focussing on selected parts of containment. Generic inspection programmes [321] do not currently give guidance on prioritizing inspections for critical locations; however, a prioritized inspection programme will lead to cost and time savings.

Optimized surveillance relies on understanding ageing mechanisms, which permits listing the structures that are most susceptible to ageing, the mechanisms involved and the potential degradation significance. This may involve using a formal quantitative ranking system [317, 319], or a simple evaluation of containment materials, exposure conditions and documented performance.

Activities may be simplified using generic data, such as that provided in Section 4, to screen degradation processes and containment locations, and highlight those that might be most relevant. This preliminary assessment may point to the need for more focused reviews of specific degradation mechanisms for individual structures or components, and to examination of possible synergies between mechanisms.

Probabilistic methods have also been developed [322] for optimizing inspection frequencies based on the significance of ageing to overall plant risk. These methods account for degradation mechanisms, inspection and remedial measures. They have the advantage of feeding directly into an NPP's overall risk assessment and of quantitatively assessing the possibility that civil engineering structure degradation may impact performance of mechanical and electrical systems. This concept has been proven for structural components (e.g. shear walls and beams), but has not yet been applied to complete containment systems.

It is important to note that an AMP is not a static programme. As illustrated in Fig. 1, it needs to be periodically reviewed and updated to reflect plant experience. Depending on plant performance, this may lead to either increased or reduced inspection intervals. In addition, continued improvements in safety thinking, and developments in current knowledge and technology relating to both concrete durability and assessment techniques, need to be incorporated into the programme.

AMPs can be improved and optimized with the assistance of self-assessments and peer reviews, which need to be a regular part of any concrete structure AMP. Peer reviewers can be from within a utility, from similar plants or from the civil engineering industry at large.

6. PLANT OPERATION

6.1. OPERATING ACCORDING TO PROCEDURES AND TECHNICAL SPECIFICATIONS

Plant operation influences degradation rates of NPP systems, structures and components. Exposure of structures to operating conditions (e.g. temperature, pressure, humidity, radiation and aggressive chemicals) outside design limits can lead to accelerated ageing and premature degradation. Uncontrolled modifications involving drilling into or anchoring items to concrete surfaces can introduce degradation sites.

Since operating practices influence structure conditions, NPP operations staff have an important role within the AMP to minimize age related degradation of concrete structures by maintaining operating conditions within design limits. As an example for concrete pipelines (PCCP pipe) the control of maximum pressures (i.e. proper pipeline operating procedures) has been shown to positively contribute to pipe life.

6.2. CHEMISTRY AND CHEMISTRY CONTROL

Plant chemistry can impact concrete structures that contain water or can come into contact with water leaking through associated systems. For example, SFPs (see Section 3.3.1) contain demineralized or borated water with defined chemistry requirements. Out of specification SFP chemistry can accelerate structure degradation.

Where leaks are present, collection and analysis of the leaking water (as compared to water within the structure) can provide information regarding potential structure degradation (i.e. impact of leak on concrete and embedded steel).

6.3. ENVIRONMENTAL CONTROL

Environmental control of the internal NPP environment is part of a good AMP. High room temperatures, hot spots and irradiation can damage concrete and other equipment. NPPs typically have a defined equipment qualification programme that maintains information regarding room temperatures, humidity and equipment requirements, and provides for ongoing room condition monitoring to ensure the equipment qualification programme assumptions remain valid. Room environmental conditions obtained via this equipment qualification monitoring programme can be used to validate concrete structure design assumptions related to ambient conditions. For example, as concrete structures are designed with specific temperature limits, it can be verified that these limits are not exceeded during operation. Section 4 of Ref. [323] presents an example of qualification monitoring for electrical cabling.

Groundwater and soil chemistry monitoring can both provide an indication of, or changes in, harmful substances (degradation initiators), and in some cases can indicate structural leakage (e.g. tritium or other radioactivity in groundwater, among others).

6.4. OPERATING HISTORY, INCLUDING TRANSIENT RECORDS

Record-keeping of relevant operational data (e.g. environmental conditions, test conditions and results) is also essential for an AMP. In particular, it is prudent to attempt to control and monitor the operating environment of inaccessible parts of the structure (e.g. basemat and embedded portions of containment liner) where detection and repair of degradation would be difficult and costly.

Major structure transients and events (e.g. impacts, seismic events, detensionings, fires, chemistry excursions, severe weather events, modifications, among others) need to be recorded and evaluated at the time of occurrence and kept as part of structure records.

7. INSPECTION, MONITORING AND ASSESSMENT OF STRUCTURES

7.1. BACKGROUND

Almost from the time of construction, reinforced concrete structures start to deteriorate due to environmental exposure (e.g. temperature, moisture and cyclic loadings) [324]. Deterioration rates are dependent on the component's structural design, materials selection, construction quality, curing and environment aggressiveness.

Component service life ends when it can no longer meet functional and performance requirements. As noted by the deterioration of many roadways and bridges in the USA [325], this often occurs prior to achieving the desired service life. Experience has shown that concrete structures require maintenance and repair actions to combat degradation [214].

In-service inspection techniques are available that can indicate the occurrence and extent of age or environmental stressor related deterioration. Results obtained can be used to develop and implement remedial actions prior to the structure reaching unacceptable performance levels.

Staff tasked with performing inspections need to be trained and qualified as appropriate for tasks they are to perform [326]. Examples of requirements for inspection personnel are in ACI 349.3R and ASME Section XI Subsection IWL. Most inspection techniques require interpretation to define acceptable versus unacceptable results, which requires specialized training and experience.

Basic elements of assessment include:

— Preplanning and accumulation of background data (e.g. age, previous condition surveys, design documents, as built drawings and materials data sheets);
— Visual examination;
— In situ and laboratory testing;
— Evaluation of collated survey data and determination of cause(s) of deterioration.

From this information, a remedial measure strategy is developed based on damage consequences (e.g. effect of degradation on structural safety), time requirements for implementation (e.g. shutdown requirements, immediate or future safety concern), economic aspects (e.g. partial or complete repair) and residual service life requirements (e.g. desired residual service life will influence action taken) [327]. Basic remedial measure options include:

(a) Active intervention;
(b) More frequent inspections;
(c) Repairs to restore deteriorated or damaged parts of structure to a satisfactory condition;
(d) Action to prevent deterioration from getting worse (if safety margins are presently acceptable);
(e) Demolition and rebuild (where feasible) of all or part of a structure.

Quite often options (c) and (d) are considered jointly.

7.2. INSPECTION AND SURVEILLANCE

In-service examinations (that can include inspections, testing and monitoring) and surveillance are essential elements of an effective AMP. Concrete inspection and surveillance activities are designed to detect and characterize significant component degradation before fitness for service is compromised.

Surveillance activities are typically those performed by operations and engineering staff as part of plant walkdowns and review of inspection, maintenance and testing records. Inspections are those activities that are typically scheduled routinely to evaluate, in detail, the condition of a particular structure based on defined acceptance criteria.

Examination programme effectiveness and relevance can be significantly enhanced by plotting and tracing data collection and processing results to display structure degradation. Improved routine and in-depth examination techniques, such as non-contact NDT, embedded wireless sensors and NDT techniques with higher penetration, can support inspection of inaccessible structures.

As the concrete surface area of an NPP is large, with not all of it being readily accessible, risk informed techniques can be developed to choose particular areas or components that are representative of conditions over a wider area, or of areas or components most likely to deteriorate. Analytical models (see Section 4.3.1) and condition assessments (see Section 5.2.2) can assist with the selection of highly stressed areas.

Together with an understanding of concrete ageing degradation, examination and surveillance results provide a basis for decisions regarding the type and timing of maintenance actions to correct detected ageing effects. The rigor and extent of these increases as concrete structures develop problems. Normally, visual inspection of accessible surfaces of structures is conducted. Visual inspections are supplemented by non-destructive and destructive tests in areas exhibiting distress. For containment structures, periodic leak rate testing is also performed. Results can also impact decisions regarding changes in operating conditions and practices to control significant ageing mechanisms.

It is important to know the accuracy, sensitivity, reliability and adequacy of the non-destructive methods used to identify and evaluate the particular type of suspected degradation. The performance of examination method(s) must be evaluated in order to rely on their results, particularly in cases where they are used as part of fitness for service assessments.

There are a vast variety of test methods available for use in performing an examination of reinforced concrete structures and their materials of construction. This section focuses on methods most commonly used and on those that represent good practice for detection of degradation of concrete structures. Tables 13 and 14 provide a listing of methods that can be used to assess properties or characteristics of concrete, metallic materials and protective media. Additional information to that provided in these tables is in Refs [209, 328–333]. Often, the most effective approach to detect ageing is to use a combination of methods.

TABLE 13. METHODS TO ASSESS CONCRETE PROPERTIES OR CHARACTERISTICS

Evaluation method / Concrete property or characteristic	Air permeability (S)	Audio methods (N)	Break-off methods (S)	Carbonation depth (D)	Chloride testing (S)	Core testing (D)	Infrared thermography (N)	Instrumentation (N)	Magnetic methods (N)	Modal analysis (N)	Petrographic methods (D)	Probe penetration (S)	Pull-out testing (S)	Radar (N)	Radiation/nuclear (N)	Rebound hammer (N)	Stress wave transmission (N)	Tomography (N)	Ultrasonic pulse velocity (N)	Visual inspection (N)
Alkali–carbonate reaction											X									
Air content	X										X									
Acidity				X	X															
ASR											X									
Bleeding channels											X									X
Cement content											X									
Chemical composition											X									X
Chloride content					X	X														
Compressive strength			X			X						X	X			X			X	

TABLE 13. METHODS TO ASSESS CONCRETE PROPERTIES OR CHARACTERISTICS (cont.)

Concrete property or characteristic \ Evaluation method	Air permeability (S)	Audio methods (N)	Break-off methods (S)	Carbonation depth (D)	Chloride testing (S)	Core testing (D)	Infrared thermography (N)	Instrumentation (N)	Magnetic methods (N)	Modal analysis (N)	Petrographic methods (D)	Probe penetration (S)	Pull-out testing (S)	Radar (N)	Radiation/nuclear (N)	Rebound hammer (N)	Stress wave transmission (N)	Tomography (N)	Ultrasonic pulse velocity (N)	Visual inspection (N)
Concrete cover						X			X					X						
Aggregate content											X									
Mixing water content											X									
Corrosive environment	X			X	X															X
Cracking		X				X		X			X				X		X	X	X	X
Creep						X		X												
Delamination		X				X	X				X				X		X	X	X	X
Density						X									X					
Elongation						X		X												
Embedded parts														X	X			X		
Frost damage											X									
Honeycomb						X					X				X			X	X	X
Modulus of elasticity						X													X	
Modulus of rupture						X														
Moisture content						X					X									
Structural performance		X						X		X										X
Permeability	X										X									
Pull-out strength													X							
Aggregate quality											X									X
Freeze-thaw resistance											X									
Soundness						X									X			X		
Splitting-tensile strength						X														
Sulphate resistance											X									
Tensile strength			X			X														

TABLE 13. METHODS TO ASSESS CONCRETE PROPERTIES OR CHARACTERISTICS (cont.)

Concrete property or characteristic \ Evaluation method	Air permeability (S)	Audio methods (N)	Break-off methods (S)	Carbonation depth (D)	Chloride testing (S)	Core testing (D)	Infrared thermography (N)	Instrumentation (N)	Magnetic methods (N)	Modal analysis (N)	Petrographic methods (D)	Probe penetration (S)	Pull-out testing (S)	Radar (N)	Radiation/nuclear (N)	Rebound hammer (N)	Stress wave transmission (N)	Tomography (N)	Ultrasonic pulse velocity (N)	Visual inspection (N)
Concrete uniformity											X					X				X
Voids						X								X	X		X	X	X	X
Water–cement ratio											X									

Note: (N): Non-destructive method, (S): Semi-destructive method and (D): Destructive method.

TABLE 14. METHODS TO ASSESS METALLIC COMPONENTS AND PROTECTIVE MEDIA PROPERTIES OR CHARACTERISTICS

Property or characteristic \ Evaluation method	Coating measurement	Cross-cut test	Four-electrode method	Grease tests	Half-cell potential	Lift-off test	Liquid penetrant	Local leak test	Magnetic particle test	Rate of corrosion probes	Tendon mechanical tests	Ultrasonic tests	Visual inspection
Aggressive ions (P)				X									
Coating bond performance (P)		X											X
Broken wires (T)													X
Coating thickness (P)	X												
Weld cracks (L)							X		X				X
Coating distress (P)													X
Elongation (T)											X		
Free water quantity (P)				X									
Leakage (L)								X					X
pH value (P)				X									
Prestressing force loss (T)						X							
Reinforcement corrosion (R)			X		X					X			X
Structural degradation (L, T)													X

TABLE 14. METHODS TO ASSESS METALLIC COMPONENTS AND PROTECTIVE MEDIA PROPERTIES OR CHARACTERISTICS (cont.)

Property or characteristic \ Evaluation method	Coating measurement	Cross-cut test	Four-electrode method	Grease tests	Half-cell potential	Lift-off test	Liquid penetrant	Local leak test	Magnetic particle test	Rate of corrosion probes	Tendon mechanical tests	Ultrasonic tests	Visual inspection
Surface cracks (L, P)							X		X				X
Ultimate strength (T)											X		
Wall thickness (L)										X		X	
Yield strength (T)											X		

Note: (L): Liner, (P): Protective media, (R): Mild steel reinforcement and (T): Tendon.

7.2.1. Concrete

Primary manifestations of NPP concrete distress include cracking, voids, delaminations and strength losses. Corrosion of embedded steel reinforcement, which also is a primary form of degradation of concrete structures, is addressed in Section 7.2.2. Methods used to detect discontinuities in concrete structures fall into two general categories: direct and indirect. Direct methods involve a visual structure inspection, removal, testing and analysis of material, or a combination of these. Indirect methods measure some parameter from which a degradation estimate can be made. Often, the evaluation of concrete structures and materials requires a combination of test methods since no single testing technique is available that will detect all potential degradation factors.

Testing methods are also grouped into categories of non-destructive and destructive testing. Assessments of inaccessible concrete components can be done by removing material to expose the component of interest and applying the testing methods described in the following, indirectly through environmental evaluations (i.e. quantification of aggressiveness of the ambient environment), or via assessment of adjacent accessible structures exposed to similar environments. If the ambient environment is determined to be aggressive, additional testing and evaluation is required that may involve removal of material to expose components for direct inspection and testing.

7.2.1.1. Non-destructive testing

NDT utilizes non-invasive techniques to determine the integrity of a material, component or structure, or to quantitatively measure some object characteristic. The objectives of NDT are to:

— Determine material properties;
— Detect, characterize, locate and size discontinuities/defects;
— Determine manufacturing or fabrication quality;
— Check for deterioration after a period of service [334].

For purposes of definition, a flaw is a detectable lack of continuity or a detectable imperfection in a physical or dimensional attribute of the component or structure. A discontinuity can occur as a result of material selection, manufacturing processes, construction practices, handling, geometric configuration, operation, service loads or environmental conditions.

NDT methods can be used to:

— Indicate concrete strength;
— Indicate density and quality;
— Locate and characterize voids or cracks;
— Locate rebar and embedments;
— Indicate concrete cover depth;
— Indicate corrosion of reinforcing materials.

Tables 15, 16 and 17 present NDT methods for determining structural properties and assessing concrete condition, determining material properties of hardened concrete in existing structures and for repair evaluation.

TABLE 15. NON-DESTRUCTIVE TEST METHODS TO DETERMINE STRUCTURAL PROPERTIES AND ASSESS CONDITIONS OF CONCRETE [335]

Property	Methods		Comment
	Primary	Secondary	
Reinforcement location	Covermeter; ground-penetrating radar (GPR) (ASTM D4748)	X ray and gamma ray radiography	Steel location and distribution; concrete cover
Concrete component thickness	Intrusive probing impact echo (ASTM C1383); GPR (ASTM D4748)		Verify thickness of concrete; provide more certainty in structural capacity calculations; impact echo requires knowledge of wave speed and GPR of dielectric constant in place of specific calibration to known thickness on-site
Steel area reduction	Intrusive probing; ultrasonic thickness gauge (requires direct contact with steel)	Radiography	Observe and measure rust and area reduction in steel; observe corrosion of embedded post-tensioning components; verify location and extent of deterioration; provide more certainty in structural capacity calculations
Local or global strength and behaviour	Load test, deflection or strain measurements	Acceleration, strain and displacement measurements	Ascertain acceptability without repair or strengthening; determine accurate load rating
Corrosion potentials	Half-cell potential (ASTM C876)		Identification of the likelihood of corrosion
Corrosion rate	Linear polarization (SHRP-S-324 and SHRP-S-330)		Corrosion rate of embedded steel; rate influenced by environmental conditions
Location of delaminations, voids and other hidden defects	Impact echo; infrared thermography (ASTM D4788); impulse response; radiography; GPR (ASTM D6087); sounding (ASTM D4580)	Pulse echo; ultrasonic pulse velocity; intrusive drilling and borescope	Assessment of reduced structural properties; extent and location of internal damage and defects; sounding limited to shallow delaminations

TABLE 16. NON-DESTRUCTIVE TEST METHODS FOR DETERMINING MATERIAL PROPERTIES OF HARDENED CONCRETE IN EXISTING STRUCTURES (*adapted from Ref.* [335])

Property	Possible methods		Comment
	Primary	Secondary	
Compressive strength (ACI 228.1R)	Cores for compression testing (ASTM C42/C42M and C39/C39M); pull-out testing (post-installed) (ASTM C900)	Penetration resistance (ASTM C803/C803M)	Strength of in-place concrete; comparison of strength in different locations
Relative compressive strength	Rebound number (ASTM C805/C805M); ultrasonic pulse velocity (ASTM C597)	In-place pull-off test (ACI 503.1R: BS 1881-207) with appropriate calibration	Rebound number influenced by near surface properties; ultrasonic pulse velocity gives average result through thickness
Tensile strength	Splitting tensile strength of core (ASTM C496/C496M)	In-place pull-off test (ACI 503.1R; ACI 228.1R; BS 1881-207)	Assess tensile strength of concrete
Density	Relative density (specific gravity) of samples (ASTM C642)	Nuclear gauge (ASTM C1040/C1040M)	
Moisture content	In-place moisture probes (ASTM F2170) Preinstalled fibre optic relative humidity sensors	Nuclear gauge (ASTM C1040/C1040M); moisture content by drying (ASTM C642)	
Static modulus of elasticity	Compression test of cores (ASTM C469/C469M)		
Dynamic modulus of elasticity	Resonant frequency testing of sawed specimens (ASTM C215)	Ultrasonic pulse velocity (ASTM C597); impact echo; spectral analysis of surface waves	Requires knowledge of density and Poisson's ratio (except ASTM C215); dynamic elastic modulus is typically greater than static elastic modulus
Shrinkage/expansion	Length change of drilled or sawed specimens (ASTM C341/C341M)		Measure of incremental potential length change
Resistance to chloride penetration	90 day ponding test (AASHTO T259); ASTM C1543) permit ion migration test	Electrical indication of concrete's ability to resist chloride ion penetration (ASTM C1202)	Establishes relative susceptibility of concrete to chloride ion intrusion; assesses effectiveness of chemical sealers, membranes and overlays
Air content; cement content and aggregate properties (scaling, alkali–silica reactivity, freezing and thawing susceptibility)	Petrographic examination of concrete samples removed from structure (ASTM C856 and ASTM C457/C457M); cement content (ASTM C1084)	Petrographic examination of aggregates (ASTM C294, ASTM C295/C295M)	Assist in determination of cause(s) of distress; degree of damage; quality of concrete when originally cast and current
Alkali–silica reactivity	Cornell/SHRP rapid test (SHRP-C-315); petrography		Establish in field if observed deterioration is due to ASR

TABLE 16. NON-DESTRUCTIVE TEST METHODS FOR DETERMINING MATERIAL PROPERTIES OF HARDENED CONCRETE IN EXISTING STRUCTURES (*adapted from Ref.* [335]) (cont.)

Property	Possible methods		Comment
	Primary	Secondary	
Carbonation, pH	Phenolphthalein (qualitative indication); pH meter; fibre optic pH sensors embedded in concrete	Other pH indicators (e.g. litmus paper)	Assess corrosion protection value of concrete with depth and susceptibility of steel reinforcement to corrosion; depth of carbonation
Fire damage	Petrography; rebound number (ASTM C805/ C805M)	Spectral analysis of surface waves; ultrasonic pulse velocity; impact echo; impulse response	Rebound number permits demarcation of damaged surface
Freezing and thawing damage (in-place)	Petrography	Spectral analysis of surface waves; impulse response; ultrasonic pulse velocity	
Chloride ion content	Acid-soluble (ASTM C1152/C1152M) and water soluble (ASTM C1218/ C1218M)	Specific ion probe (SHRP-S-328)	Chloride ingress increases susceptibility of steel reinforcement to corrosion
Air permeability	SHRP surface airflow method (SHRP-S-329); Torrent (1992) and autoclam air-permeability tests		Measure in-place permeability index of the near surface concrete: within 0.6 in (15 mm)
Electrical resistance of concrete	AC resistance using four-probe resistance meter (FSTM 5-578)	SHRP surface resistance test (SHRP-S-327)	AC resistance useful for evaluating effectiveness of admixtures and cementitious additions; SHRP method useful for evaluating effectiveness of sealers
Water absorption (sorptivity)	ISAT, Figg, covercrete, or autoclam sorptivity tests		In-place moisture content considered for interpreting the data

TABLE 17. NON-DESTRUCTIVE TEST METHODS FOR EVALUATING REPAIRS [335]

Property/Condition	Method		Comment
	Primary	Secondary	
Bond strength	Pull-off test (ASTM C1583/C1583M); CAN/CSA A23.2; BS 1881-207		
Bond quality (absence of voids at interface)	Pull-off test (as above)	Impact echo; impulse response	
Injection of cracks or voids	Ultrasonic pulse velocity	Impact echo	Proper geometry required for ultrasonic pulse velocity reliability

(a) Visual inspection

Visual inspection is generally the first step in an inspection programme. It is performed in accordance with applicable codes, standards, specifications and procedures (e.g. ASME Section XI Subsection IWL, ACI 349.3 [241], CSA N287.7 [249]). Age related visible effects on the structure's surface (see Table 18) can be detected by visual inspection of concrete structures [53].

The five basic elements of a visual inspection include:

(1) The object;
(2) The inspector;
(3) The optical instrument;
(4) Illumination;
(5) Recording [336].

EPRI has developed a checklist, for NPP concrete structures and supports, that lists observable indicators, possible stressor or degradation mechanisms producing the indicator, and potential consequences of inaction [242].

TABLE 18. CAUSES AND SYMPTOMS OF CONCRETE DEGRADATION THAT CAN BE OBSERVED DURING VISUAL INSPECTION (*reproduced from Ref.* [53] *with permission*)

Cause	Symptoms
Construction faults	Bug holes, cold joints, exposed reinforcing, honeycombing, irregular surface
Cracking	Checkering, crazing, D-cracking, diagonal, hairline, longitudinal, map pattern, random, vertical, horizontal
Erosion	Abrasion, cavitation, joint-sealant failure
Spalling	Pop outs, spall
Distortion, deflection or movement	Buckling, curling, warping, faulting, settling, tilting
Disintegration	Blistering, chalking, delamination, dusting, peeling, scaling, weathering
Seepage	Corrosion, discolouration, staining, exudation, efflorescence, incrustation

Visual inspections also include periodic mapping and measurements to provide a history of crack appearance and development that can assist in identifying their cause and establishing whether a crack is active or dormant. Visual inspection tools include:

— Field books;
— Clipboards;
— Markers;
— Flashlights;
— Cameras;
— Measuring tapes;
— Callipers;
— Optical magnification devices;
— Mirrors;
— Feeler gauges;
— Crack comparators;

— Straight edges;
— Levels;
— Pocket knives;
— Wire and paint brushes;
— Screwdrivers;
— Pliers;
— Chipping hammers;
— Binoculars;
— Sounding lines.

Optical aids, such as fibrescopes and borescopes, allow inspection of inaccessible regions. Optical aid selection depends on factors such as object geometry and access, expected defect size and resolution requirements. Video cameras can be used to record current conditions for future reference.

Such a system has been developed for monitoring cooling towers in Belgium [337]. Monitoring is based on results obtained from topographic surveys, inventory of deterioration types and analysis of structure materials. Markers are used as reference points on the outer surface of the cooling tower shell and a long focal length telescope coupled to a video recording system is used to record results. Information such as type, number and classification of defects is stored in a database used to trace defect evolution and to generate reports.

A video microscope with image acquisition, image display and image analysis capabilities has been developed for monitoring cracks in structures [338]. Figure 58 presents an example building survey system.

FIG. 58. Building survey system (courtesy of and copyright retained by BAM Federal Institute for Materials Research and Testing [339]).

Scuba divers have traditionally been used to inspect underwater conventional concrete structures, and this is no different at NPPs. Some underwater structures, notably SFPs, are associated with radiation hazards that can make use of scuba divers problematic, though not impossible in some cases. To minimize personnel hazards, underwater vehicles (tethered or remotely operated vehicles) that use video or still cameras and other sensors, are an increasingly available option.

Visual inspections can be done in clear water; however, many types of structures are located in water that is not clear enough to perform a visual examination.

Photos of a typical underwater inspection remotely operated vehicle and pictures obtained are in Figs 59 and 60.

Where scuba divers can be used, many types of test devices (detailed in other sections) in use above water have been adapted for use below water, for example, rebound hammers and both direct and indirect ultrasonic pulse velocity systems. These tools provide good readings on the general condition of underwater concrete [340].

FIG. 59. Typical remotely operated vehicle for nuclear applications (courtesy of Atlantas Marine).

FIG. 60. Remotely operated vehicle photograph of debris in spent fuel pool during decommissioning work (courtesy of Atlantas Marine).

Limitations of visual methods are that they cannot reveal internal degradation of concrete structures when there are no visible surface symptoms (e.g. subsurface cracking, voids and delaminations and extent of cracking). Broad knowledge in structural engineering, concrete materials and construction methods is needed in order to extract the most information from the inspection. Useful guides are available to help recognize and classify different types of damage as well as probable causes (see Refs [241, 300, 341–343]).

(b) Stress wave propagation methods

Acoustic methods are based on elastic wave propagation in solids. Propagation of sound takes place via compression (P) waves, shear (S) waves in solids and surface waves or Rayleigh (R) waves along surfaces. Stress waves occur when pressure or deformation is suddenly applied to the surface of a material and are propagated in a manner analogous to how sound travels through air. Inhomogeneities in concrete cause scattering of sound waves that can be recorded and analysed. Several test methods based on stress wave propagation can be used for NDT of concrete structures.

Ultrasonic pulse velocity equipment for examination of concrete materials is essentially the same as that used for metallic materials, except a 30–200 kHz transducer is used instead of a 0.1–25 MHz transducer because of the greater attenuation characteristics of concrete. The basic components of ultrasonic pulse velocity include a means for producing and introducing repetitive pulses into the material that is being examined, and a means of accurately measuring the time required for these pulses to travel through the material to a receiver. Figure 61 presents an example of a portable ultrasonic test system. A material's condition is assessed through the determination of pulse velocity and stress wave amplitude at the receiver [344]. When travel time of stress waves between generator and receiver versus location is displayed, there will be a deviation in the curve at the position of the subsurface defect.

FIG. 61. Example of portable ultrasonic pulse velocity test equipment (courtesy of NDT James Instruments [345]).

The method for detecting internal structural changes is limited by concrete segregation, inhomogeneity and quality of acoustical contact. Aggregate, pores, defects and rebar have acoustical properties much different from those of the cement matrix and give rise to scattering and mode conversion of propagating ultrasonic waves. By using ultrasonic pulse velocity, it is possible to determine the concrete dynamic modulus of elasticity, Poisson's ratio, thickness and estimate in situ compressive strength. The method also can be used to detect concrete internal structure changes, cracking or voids and changes due to freezing and thawing or other aggressive environments. To measure for strength and related properties, the test must be calibrated to the specific concrete, as results are influenced by:

— Aggregate size, type and gradation;
— Cement type;
— Water to cementitious materials ratio;
— Admixtures;
— Degree of compaction;
— Curing conditions and age of the concrete;
— Acoustical contact;
— Concrete temperature;
— Moisture content;
— Size and shape of specimen;
— Presence of rebar.

The ultrasonic pulse velocity test is most useful when carrying out comparative surveys of concrete quality in or between similar concrete structures.

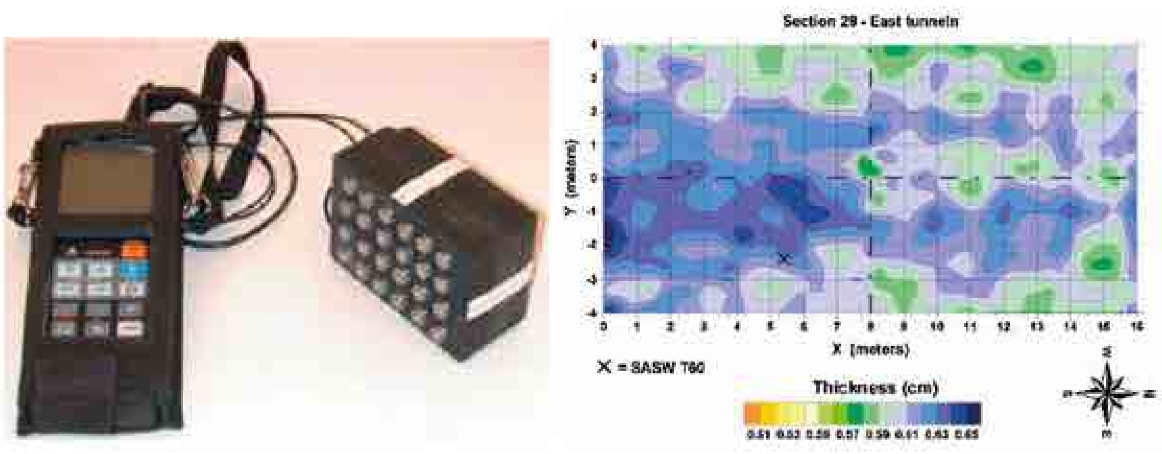
Ultrasonic pulse echo is based on the propagation and reflection of mechanical waves that are generated at concrete surfaces by an array of transducers that resonate horizontally. It involves the use of transmitting and receiving transducers that are normally placed close to each other on the testing surface. The transmitter also may be designed as a receiver. The pulsed signal input may be produced by a piezoelectric transducer or surface impacts may be used. The pulse propagates into the test object and is reflected by flaws or interfaces. The echo signals are analysed and their transmission times may be converted to velocities if the wave speed is known. In this way, it is possible to measure the depth of reflectors (e.g. cracks or large voids). Because of concrete heterogeneity, it may be difficult to distinguish actual defects. This method can only use relatively low frequency inputs because of the significant scattering effect that large aggregates have on signals. Modern pulse echo equipment has achieved some success in this respect by using arrays of transducers.

Ultrasonic arrays offer two key advantages over standard monolithic transducers:

(a) An array can make a range of different inspections from a single location;
(b) Most types of arrays can produce images at each test location, which permits rapid visualization of the internal structure of a component [346].

Phased arrays consist of collections of similar transducers that are fed by a set of pulsars that can fire an impulse or a predefined signal with an appropriate delay and amplitude to reach a desired beam direction [347]. Electronic scans are performed by multiplexing along an array. Signals transmitted and received between combinations of transducers make it possible (with the response averaged) to more clearly define relevant reflectors. The pulse echo method has the potential to locate and identify discrete defects or objects if sufficient focusing is achieved by the transducers. The synthetic aperture focusing technique utilizes information from many single measurements to suppress noise and improve defect location capability and measurement accuracy [348, 349].

Figure 62 shows an ultrasonic pulse echo unit with spring loaded, dry contact transducers and the results of concrete thickness measurements of a tunnel wall [350]. Figure 63 illustrates the identification of honeycombed concrete located beneath rebar using the synthetic aperture focussing technique (data was reconstructed from an ultrasonic array) [351]. The method can be used for thicknesses up to 1000–1200 mm, if aggregate size is 16 mm or less [352].



(a) (b)

FIG. 62. (a) Ultrasonic pulse echo unit with spring loaded array of dry contact transducers; (b) a C-scan, created from an ultrasonic pulse echo unit with array, showing thickness of a concrete tunnel [350].

Coda wave interferometry is a technique that allows one to observe differences in the coda portion of a diffuse field [353]. When an ultrasonic wave is emitted into a concrete specimen, the heterogeneous nature of concrete causes the wave to become highly scattered and a diffuse field is created. A diffuse field consists of two parts: the first arrival and the diffuse portions, which include the late coda contribution. Diffuse waves undergo multiple scattering, which causes them to arrive much later than the first arrival. However, diffuse waves are much more sensitive to small changes in concrete medium and they carry more information than the first arrival. Coda wave interferometry compares the two different time series of coda waves, such as stressed state and unstressed state and determines the degree of correlation. Comparing the difference between the two states allows one to monitor damage progression in the concrete specimen. There are two types of coda wave interferometry: the doublet technique and the stretching technique. The more advanced stretching technique is more commonly

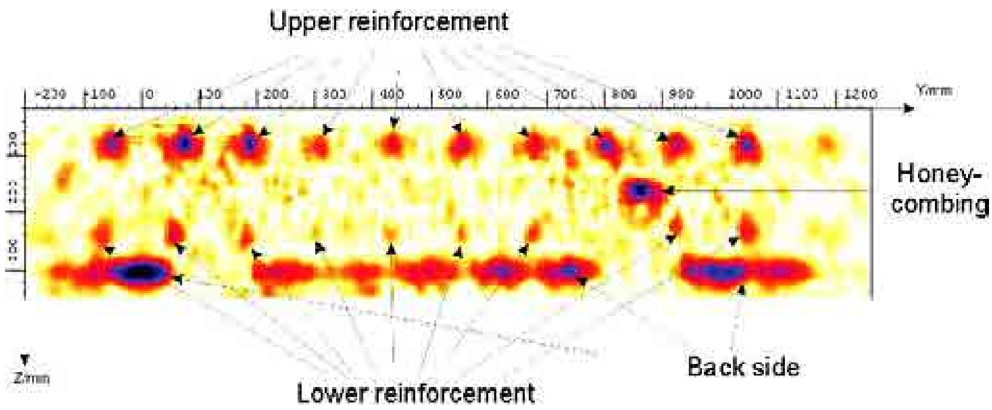


FIG. 63. Pulse echo results showing concrete honeycomb below steel reinforcement [351].

used. This technique is performed by scaling the time axis so as to stretch a waveform. The time axis is stretched or compressed until it has much in common with a reference time signal. Since time and velocity are proportional, relative velocity can be calculated from the scaling factor [353, 354]. Coda wave interferometry appears to be quite useful for detecting changes and monitoring the progression of damage in concrete. However, it does not seem to be able to locate defects and it also requires a high signal to noise ratio to be effective [353, 354].

(c) Tomographic methods

Acoustic tomography is an advanced NDT method based on the same approach as used for X ray computerized tomography and is used to examine concrete structures for cracks, voids and other internal defects [355–357]. Data collection consists of propagating mechanical waves through the surveyed plane section from different source locations to different receiver positions. During propagation between source and receiver, mechanical waves interact with the material. Local variations in elastic material properties result in corresponding variations in characteristics of received signals. Information from stress wave transmissions is used to reconstruct a velocity map of a slice through the interior of a body. Results are analysed with specialty computer software. Defects usually observed in concrete (e.g. microcracks, honeycomb and voids) generally have a greater effect on concrete modulus than on density and Poisson's ratio [358]. Figure 64 illustrates the application of acoustic travel time tomography to detect cracks in a dam structure.

X ray computerized tomography may be used on removed core samples (see Section 7.2.1.2(a)).

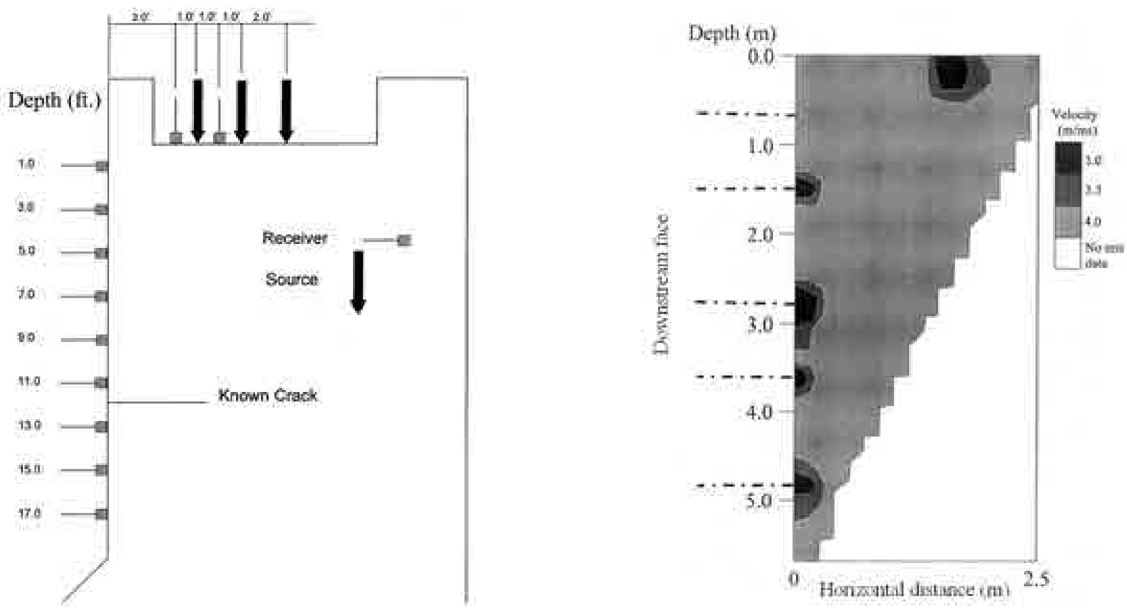


FIG. 64. Illustration of acoustic travel time tomography (dam cross-section and tomograph of dam; the five dashed lines indicate crack locations) [356].

(d) Impact methods

An impact echo test system is composed of three components, including:

— An impact source;
— A receiving transducer;
— A digital processing oscilloscope or waveform analyser that is used to capture the transient output of the transducer; store digitized waveforms and perform signal analysis [359].

A transient stress pulse is introduced into the concrete by mechanical impact of the surface and propagates along spherical wave fronts as P and S waves. The sound pulse or compression wave is reflected from the backside of the concrete, from internal reflectors (e.g. cracks), or from other objects that may cause changes in acoustic impedance and material density along the propagation path. Figures 65 and 66 present the impact echo principle. Information is obtained related to the complete, or a significant, volume of the concrete (i.e. the signal cannot be focused as with the ultrasonic pulse echo method).

Reflections or echoes are indicated by frequency peaks in resultant spectral plots that are used to locate discontinuities. Method applications include:

— Determining thickness and detecting flaws in plate-like structural members;
— Detecting flaws in beams, columns and hollow cylindrical structural members;
— Assessing the quality of bond in overlays;
— Measuring crack depth;
— Detecting the degree of grouting in post-tensioning ducts.

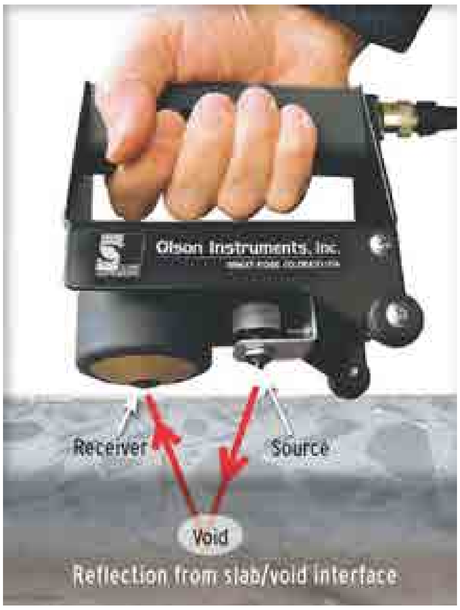


FIG. 65. Impact echo impactor equipment [360].

Spectral analysis of surface waves is used in testing concrete and in geophysical surveys. It requires access to one surface. A mechanical impact on the concrete surface is used to generate surface waves of different wavelengths. These are picked up by transducers placed at fixed distances from the impact source and the velocity of each wavelength component is evaluated. Transducers are placed in line with the impact source and with spacing determined by the depth to be measured. In the case of a massive concrete element, this may require access to a large surface area. The technique uses dispersion of surface waves to produce a surface wave velocity cross-section of the subsurface. The velocity of the Rayleigh wave is related to shear modulus (stiffness) and material density.

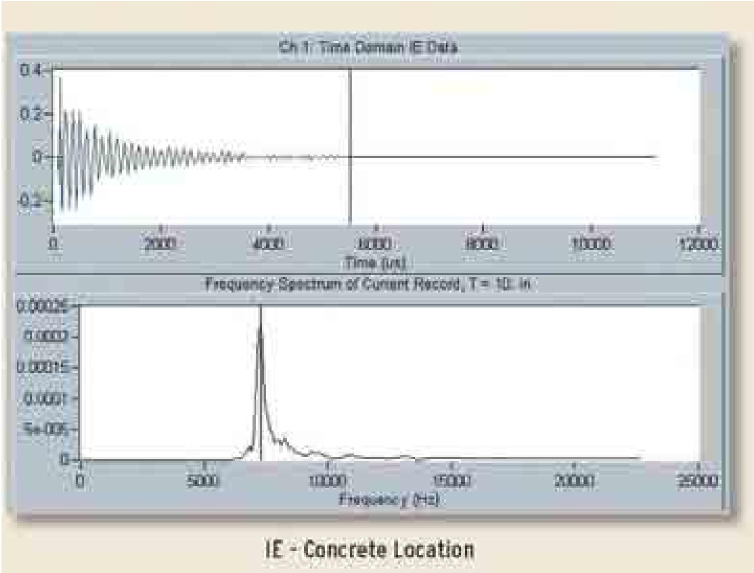


FIG. 66. Impact echo results; a single sharp peak indicates sound concrete [360].

Shear wave velocity profiles are determined from experimental dispersion curves (surface wave velocity versus wavelength) obtained from spectral analysis of surface wave measurements. Once shear wave velocity profiles are determined, shear and Young's moduli of the materials can be calculated. This method is well suited for:

— Testing large surfaces;
— Testing layered systems;
— Testing the condition of survey of liners of concrete tunnels;
— Mapping of subsurface cavities;
— Determining the depths of foundations or the condition of underlying material.

Figures 67 and 68 provide a schematic of the test set-up and use of wavelength to investigate different depths of a layered system.

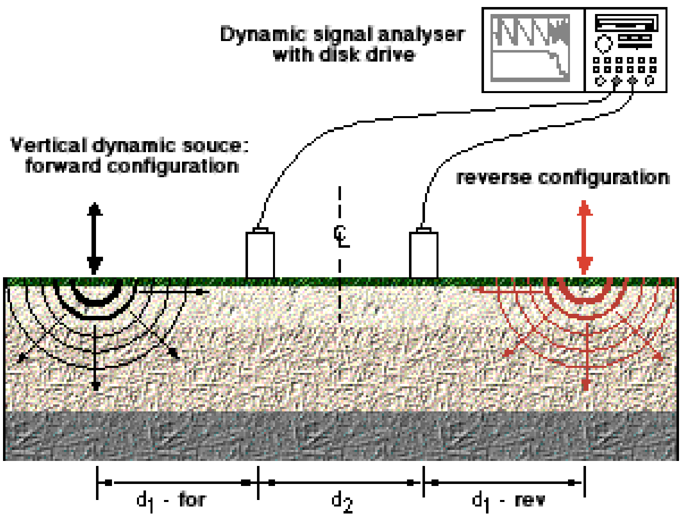


FIG. 67. Spectral analysis of surface waves test set-up (adapted from Ref. [361]).

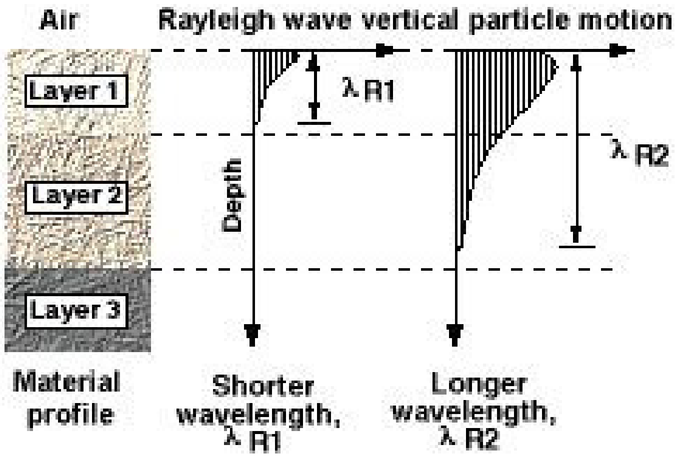


FIG. 68. Spectral analysis of surface waves, example application (use of different wavelengths to examine different depths of a layered system) [361].

Impulse response is a descendent of the forced vibration method for evaluating integrity of concrete cast-in-place bored piles [362]. Global structural health monitoring techniques such as this are based on finding shifts in resonant frequency or changes in structural mode shapes. Impulse response uses a low strain impact to send a stress wave through the tested element. The impactor is usually a sledgehammer (approximately 1 kg), with a load cell built into the hammerhead and linked to a computer. Compressive stress at the point of concrete impact is related to elastic properties of the hammer tip. Typical peak stress levels range from 5 MPa (hard rubber tip) to more than 50 MPa (aluminium tip). The impulse response test generates a compressive wave that is approximately 100 times that of the impact echo test, resulting in a plate responding in a bending mode over a much lower frequency range. Response to input stress is measured by a velocity transducer (geophone) that is connected to the computer. A fast Fourier transform algorithm is used to process the signal, in which the resulting velocity spectrum is divided by the force spectrum to obtain a transfer function (i.e. mobility). A graph of mobility plotted against frequency contains information on condition and integrity of the structure under test based on measured parameters, such as dynamic stiffness, mobility and damping and peak/mean mobility. Dynamic stiffness is used to determine concrete quality, element thickness and element support conditions. Mobility and damping can indicate surface unbonding and presence of honeycombs or cracking. The peak to mean mobility ratio is an indicator of the presence and degree of either unbonding within the element or voiding/loss of support beneath a slab on grade. Detection of voids or poorly compacted areas behind or below plate like structures is one application of this method [363]. Figure 69 shows an impulse response test set-up and sample results.

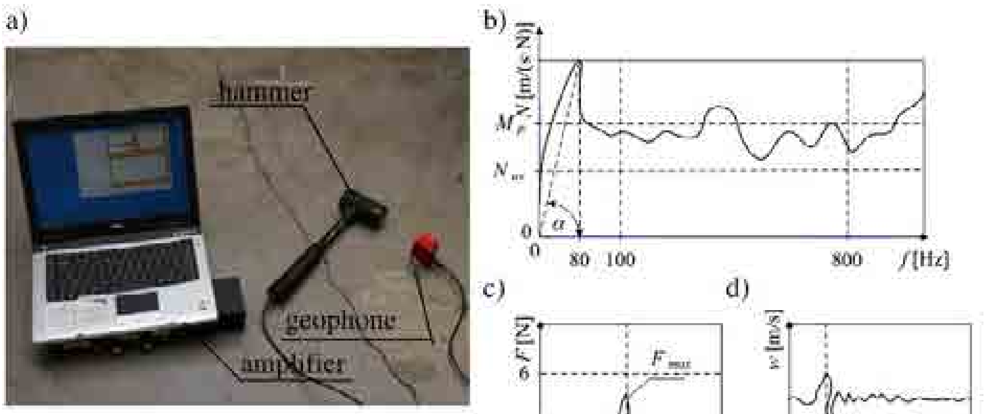


FIG. 69. Impulse response test method: (a) measuring system, (b) typical mobility (N) versus frequency curve; (c) typical trace of elastic force (F) generated by hammer; (d) typical trace of elastic wave velocity (w) recorded by geophone [364].

(e) Modal analysis

Modal analysis is the field of measuring and analysing the dynamic response of structures when excited by external or internal stimulus [365, 366]. The dynamic response of a structure is used to indicate its condition (i.e. defects or deterioration can be detected through comparisons of measured dynamic behaviour to expected behaviour). Also, in structural engineering, modal analysis uses overall structure mass and stiffness to find the various periods at which it will naturally resonate. Periods of vibration are important to note in earthquake engineering, as it is imperative that a building's natural frequency does not match the frequency of expected earthquakes in the region where the building is constructed. If a structure's natural frequency matches an earthquake's frequency, the structure may continue to resonate and be structurally damaged.

Modern modal testing systems are composed of transducers (typically accelerometers and load cells) or use non-contact approaches via a laser vibrometer, an analogue to digital front-end converter (to digitize analogue instrumentation signals) and a host personal computer to view and analyse data. Initially this was done with a single input, multiple output approach (i.e. one excitation point with responses measured at many other points). A hammer survey, using a fixed accelerometer and a roving hammer as excitation, gives a multiple input, single output analysis, which is mathematically identical to single input, multiple output, owing to the principle of reciprocity. Recently, multi-input, multiple output has become more practical, where partial coherence analysis identifies which part of the response comes from which excitation source. Typical excitation signals can be classed as impulse, broadband, swept sine and chirp. Signal analysis typically relies on Fourier transforms. The resulting transfer function will show one or more resonances, whose characteristic mass, frequency and damping can be estimated from measurements. Results can be used to correlate with finite element, normal mode solutions.

Mathematical modelling (finite element) is used to compare field measured and theoretically obtained vibration modes. The theoretical approach uses models to describe:

— Structure physical characteristics (e.g. mass, stiffness and damping properties);
— Structure behaviour as a set of vibration modes;
— Vibration response under given excitation conditions.

The response simulation can be used for NPP concrete structures to evaluate structural capacity to provide safe-shutdown in earthquake situations.

The application of modal analysis can be illustrated through comparison of the damaged state of a bridge to the state of the bridge after repair [367]. Figure 70 presents a view of damage to the main girder of the bridge resulting from a vehicle impact. Also presented in the figure is the electrodynamic exciter used to vibrate the bridge. Bridge response was measured in the vertical direction on its upper face using a chosen network consisting of 280 accelerometers. Eleven natural frequencies, modal shapes and damping frequencies were evaluated for the damaged bridge and twelve were evaluated for the repaired bridge. A comparison of frequency response functions for the damaged and repaired states measured at the middle of the damaged part of the main girder (point 112) are presented in Fig. 71. This figure illustrates how differences in mode shapes can be used for structural condition surveys (e.g. damage detection).

FIG. 70. Application of modal analysis to a damaged bridge girder: (a) view of damaged main girder; (b) electrodynamic exciter placement on a bridge [367].

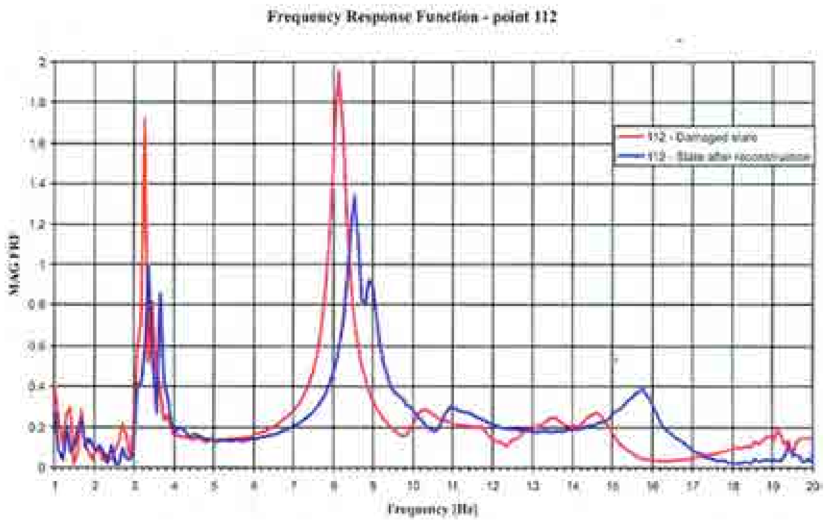


FIG. 71. Comparison of frequency response functions measured at point 112 (middle of damaged part of main girder) for bridge girder before and after repair [367].

The limitations of modal analysis are that it has to be executed by experienced staff and a baseline response is required for comparison.

(f) Nuclear methods

Nuclear methods for concrete NDT can be divided into three groups: radiographic, radiometric and neutron source. All three are based on the interaction between high energy electromagnetic radiation and the material inspected. Radiography is the method most often used to examine the quality of construction or materials in concrete (e.g. location of rebar and voids). The system consists of a radiation source (X ray or gamma ray) emitting a beam through the test article and a photographic film placed on the opposite side. Since a high density medium absorbs a greater amount of emitted energy, material density determines the energy being absorbed by the film. A two dimensional projection of the area being inspected is displayed on the film. Commonly used sources of radiation for industrial radiography are isotopes such as iridium (concrete thickness <300 mm) and cobalt (concrete thickness <600 mm). Linac systems are used to produce directed high energy X ray beams of far greater energy and radiation intensity than from isotopes of iridium or cobalt [368].

Radiography is one of the most capable NDT techniques when considering the amount of detail produced and the relative ease in understanding the data produced. The entire volume of the concrete and reinforcing, however, is projected onto a flat surface image and the geometry of rebar and internal voids displayed is somewhat distorted [353]. Techniques are available to calculate rebar depth and diameter [369]. Gamma radiometry systems consist of a source that emits gamma rays through the specimen and a radiation detector and counter. Direct transmission or backscattering modes can be used to make measurements. Neutron methods consist of an emission source and a gamma ray collection and counting system. The method can be used to measure structure moisture content. X ray and gamma ray computerized tomography involving reconstructing a cross-sectional image of an object from its measured intensities has also been investigated [370].

The limitations of the most commonly used nuclear method, radiography, are that:

— Radiation protection has to be observed while applying the method;
— Personnel need to be licensed or certified;
— The concrete structure needs to be accessible from both sides;
— Long exposure times are generally needed;
— Concrete sections are generally limited to 1 m or less in thickness.

Figure 72 presents a Megascan imaging capture system for inspection of post-tensioned bridges and other structures, and some results obtained for voided and fully grouted tendon ducts [371].

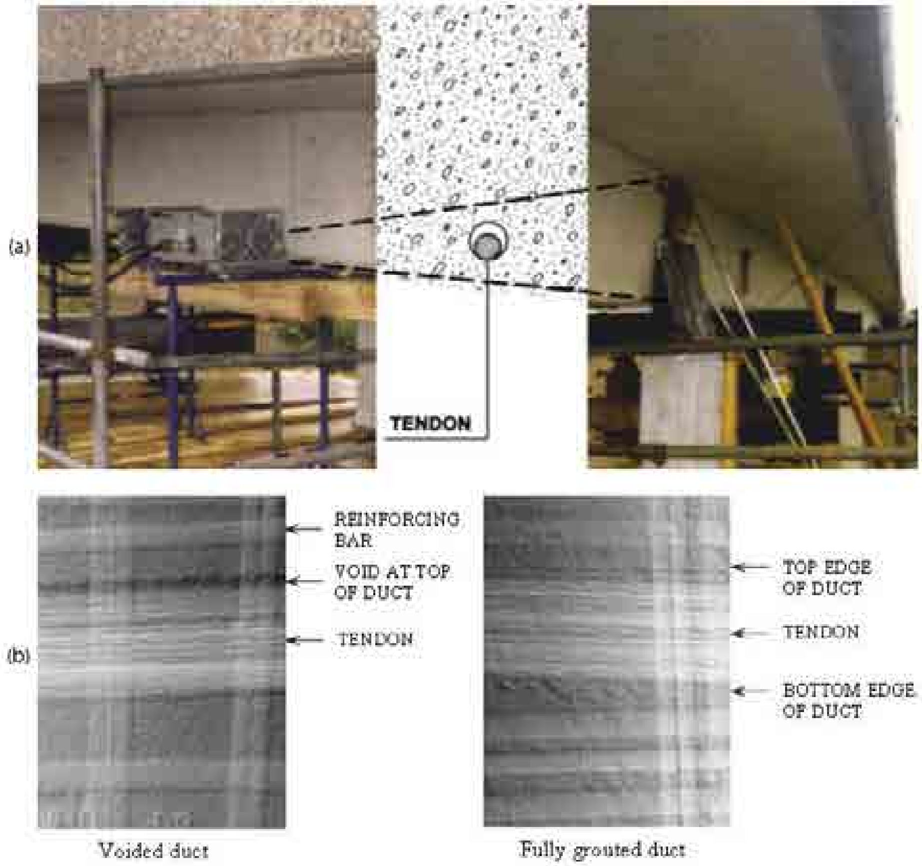


FIG. 72. The application of the Megascan imaging capture system: (a) positioning of Megascan system; (b) images showing voided and fully grouted tendon ducts [371].

(g) Electromagnetic methods

Commercial instruments (e.g. covermeters) monitor the interaction of rebar with another process, such as a low frequency electromagnetic field, to determine the location and concrete cover of rebar. Covermeters are of two types: magnetic reluctance and eddy current. Magnetic reluctance covermeters are based on monitoring changes in the magnetic flux flowing through the magnetic circuit that consists of the path through the yoke, concrete and reinforcing bar. Eddy current covermeters depend on rebar electrical conductivity and will detect magnetic as well as non-magnetic metallic objects (the signal from magnetic materials is stronger). Both methods are useful in locating rebar, measuring thickness of concrete cover and determining size and spacing of embedded rebar. The limitations of this method are that:

— The accuracy of estimated cover depth is affected by rebar size and spacing;
— Second layers of rebar cannot be identified;
— The ability to discern individual bars is affected by meter design, cover depth and bar spacing;
— Meters based on magnetic resonance can detect only ferromagnetic objects;
— Maximum penetration is limited and depends on meter design [372].

For best results, the spacing between two adjacent rebars must be greater than the concrete cover and since the method is based on the induction principle, results are affected by anything affecting the magnetic field within the instrument range (e.g. electrical cables, metal tie wires and iron content of cement). Figures 73 and 74 illustrate the application of the electromagnetic method to detect rebar location.

Infrared thermography is based on heat transfer and measures surface temperature differentials on a concrete member while the concrete undergoes heating and cooling [375]. The infrared thermography instrument senses thermal radiation emissions and produces a visual image from the thermal signal that can be related to the size

FIG. 73. Application of electromagnetic method to detect rebar location [373].

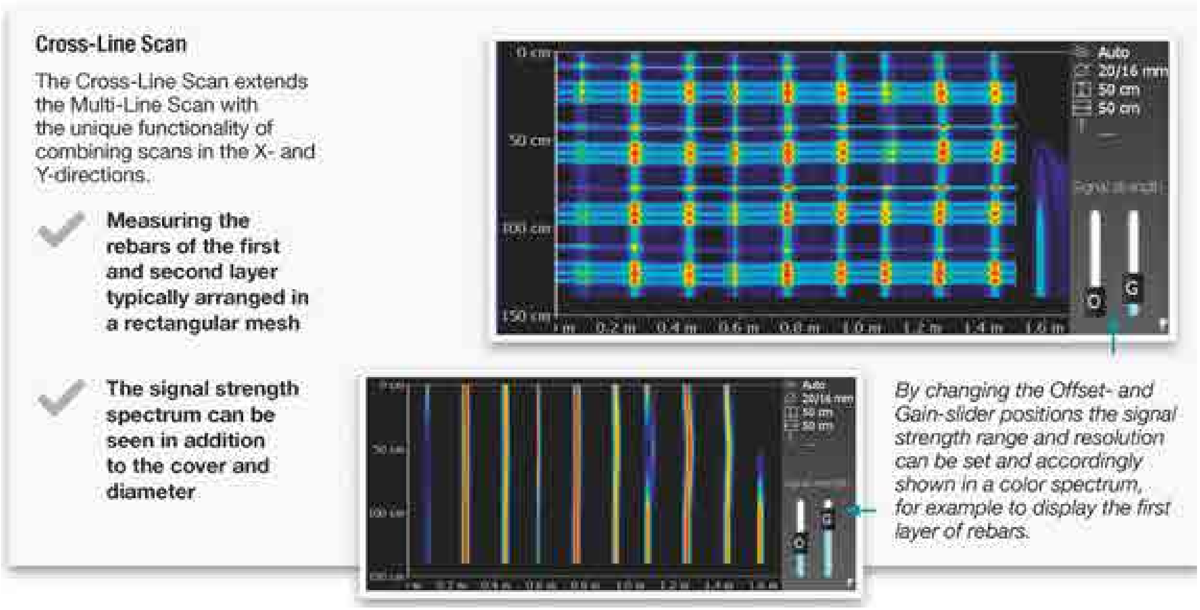


FIG. 74. Rebar mapping [374].

of internal defects (e.g. sound concrete is more thermally conductive than low density or cracked concrete). Infrared thermography for testing concrete utilizes two heat transfer mechanisms: conduction and radiation. The basic equipment involved includes an infrared scanner/detector head, a data acquisition/analysis device and a visual image recorder. The scanner head is an optical camera with lenses transmitting only infrared radiation in short and medium wavelength ranges. Since subsurface anomalies in a material affect heat flow through materials, heat transfer sensed through surface radiance variations can be used to locate subsurface voids, delaminations or other defects. The temperature difference between deteriorated and sound areas provides an indication of the depth of the defect.

The advantages of the method are that:

— Surface contact is not required;
— Equipment is very sensitive to small temperature changes;
— Results provide an indication of a deteriorated area in a survey region;
— A large concrete surface area can be covered within a short time.

The limitations of this method are that:

- The application is restricted to comparative situations;
- Complex and expensive equipment may be required;
- Surface textures and finishes will affect surface radiation properties;
- Depth to a subsurface anomaly cannot be accurately determined;
- It is necessary to produce a movement of heat in the structure, therefore, some in situ parameters such as surface moisture, ambient temperature and wind speed could influence reading accuracy.

Pulsed phase thermography provides a means to image the internal structure of a component. It combines the features of impulse thermography and lock-in thermography and records the cooling down process with an infrared camera so that at each surface position, a transient curve of temperature is recorded as a function of time [376]. Fast Fourier transformation analysis is used to separate information according to different frequencies of thermal waves and delivers amplitude and phase images. The analysis of transient curves permits a qualitative location mapping of defects.

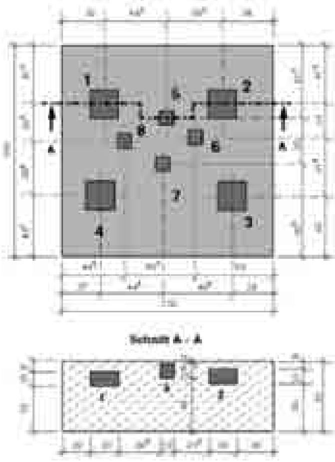
The advantages of pulse phase thermography include:

- Deeper probing to 10–15 cm;
- Less influence of surface infrared and optical characteristics (i.e. less sensitivity to non-uniform heating);
- Rapid image recording;
- Better defect shape resolution;
- Not needing to know positions of non-defect areas;
- Amplitude images show internal structure of specimen up to a maximum available depth depending on the frequency (band pass filter behaviour) [376].

Figures 75, 76 and 77 illustrate the application of impulse thermography to detect voids in a concrete test article [377].

Ground penetrating radar (GPR) is a high resolution near surface surveying tool. GPR is well developed in the geophysical field and involves the propagation and scattering of electromagnetic energy through materials and the electromagnetic analogue of sonic and ultrasonic pulse echo techniques. It has been adapted and can be used in its various forms to obtain information from concrete structures and their foundations and substrate.

FIG. 75. Impulse thermography test set-up [376].



(a)

FIG. 76. Impulse thermography test specimen with voids [376].

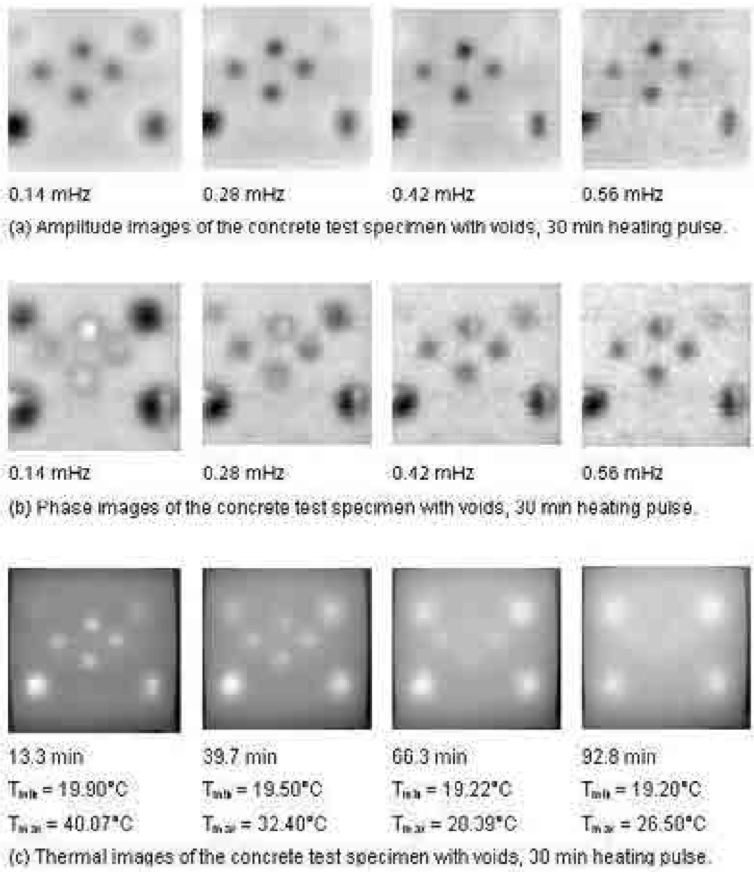


FIG. 77. Impulse thermography test results [376].

Uses for GPR include:

— Locating buried pipes, cables, rebar;
— Locating and size estimating embedded bars, caverns, cracks and flaws;
— Determining pavement thickness;
— Detecting groundwater and moisture, and determining its properties (hydraulic changes, contamination, etc.).

An advantage of using radar is that the antenna used for scanning does not require contact with the test surface, so large areas can be scanned rapidly. Short pulses of electromagnetic energy (microwaves) are transmitted through the structure. The energy is reflected by boundaries between layers of different dielectric properties, with some energy passing across an interface by refraction that may in turn be reflected from deeper interfaces. The strength and polarity of the reflection will be determined by the contrast in material properties at the interface, with buried metals providing strong reflections. The time delay before the reflected signal is received back at the surface is governed by the depth of the interface and velocity of the signal in the material. The resolution of GPR is primarily a function of antenna frequency and dielectric constant of the medium.

Figure 78 presents a GPR system and its principle of testing. The middle and right images show a schematic of the hyperbolic reflection image from a steel bar in concrete. The ability to detect reflector depth, such as reinforcing bars or tendon ducts, is dependent on the knowledge of concrete dielectric properties. The greatest penetration is possible when concrete is dry and frequency is low. A study of waveforms and patterns generated during a scan forms the basis for interpretation. Each signal pulse typically has two or three lobes, so an interface will appear as a series of bands at a particular reflection time. Reflected wave features of interest for NDT of concrete include:

— Rebar–waveform: Hyperbola with hyperbola top indicating rebar position;
— Steel plate–in-phase axes of waveform: Horizontal with no reflected wave information from below plate;
— Crack–in-phase axes of waveform: Unbalanced with the extent of unbalance increasing with crack depth [378].

A range of signal processing operations is available to assist in evaluating data (e.g. filtering, depth scaling, synthetic aperture focussing technique and automatic recognition of image features) [379]. Closely spaced rebar near the concrete surface tends to disrupt radar signals and mask deeper lying objects of interest (e.g. voids).

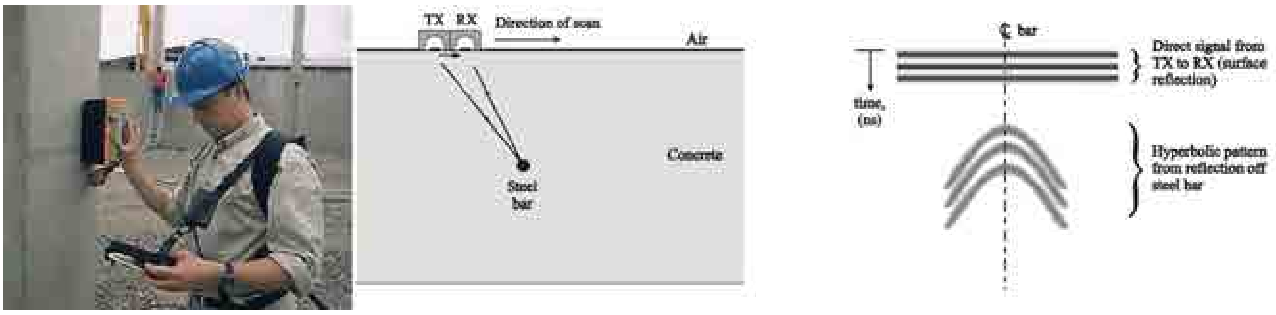
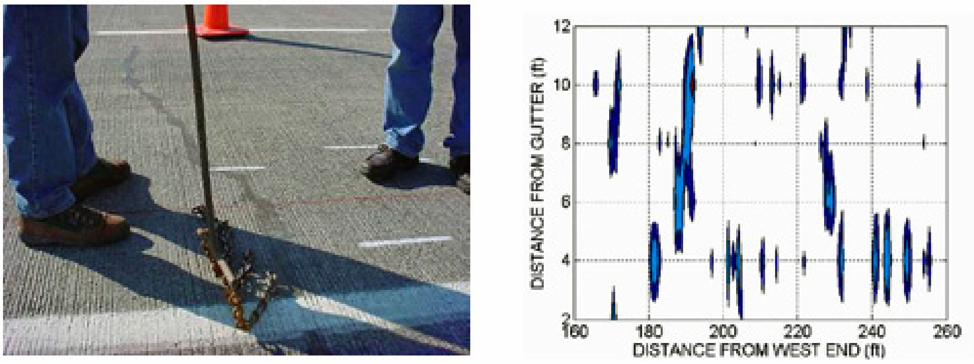


FIG. 78. Field application and principle of GPR left [380], middle and right (courtesy of Force Technology Inc.). TX: transmitter, RX: receiver.

(h) Audio/sonic methods

Audio methods are routinely used to detect delaminated areas in structures such as bridge decks. Hammers, steel bars and chains are often used. By dragging a chain across a concrete surface or using a metallic object to strike the concrete surface, it is possible to locate areas of delamination and voids through sound differentials that occur between good and defective concrete [381]. Solid areas of concrete will produce a characteristic metallic ringing sound when impacted, while defects in the form of debonds, cracks or other delaminations produce a hollow sound when struck. The operation is relatively fast and is performed over a grid to map the structure and provide a delamination profile. Method limitations are the large size of these structures (i.e. thicknesses up to several metres) and the fact that only these methods can be applied to local and selected test areas (because of accessibility constraints). Also, the technique relies on subjective judgment of the operator to differentiate between sound and unsound areas and results cannot be quantified. The technique is usually effective for defects not exceeding concrete cover depth, but it may miss small delaminations. Figure 79 presents a chain drag system scanning a bridge deck, and an example defect profile showing delamination areas.



(a) (b)

FIG. 79. (a) Use of chain drag to indicate delaminated regions of a bridge deck; (b) equipment and defect map showing areas of delamination [382].

(i) Surface hardness methods

Rebound hammers use the rebound distance (measured on an arbitrary scale) of a spring loaded weight that has impacted concrete to estimate quality or compressive strength.

The rebound hammer is used for the following purposes:

— To assess concrete uniformity in situ;
— To delineate zones (or areas) of poor quality or deteriorated concrete;
— To indicate changes of concrete characteristics over time;
— To classify abrasion resistance.

The effectiveness of the rebound hammer method is often enhanced through its combination with other techniques, such as ultrasonic pulse velocity measurements. The primary method limitations are that test results only measure surface characteristics and results may be influenced by parameters, such as:

— Test surface smoothness and moisture content;
— Orientation of hammer during impact;
— Type of cement used;
— Type of aggregate;
— The need to develop application specific calibration curves to provide reasonably accurate compressive strength results.

Surface treatments may exclude direct technique application. Figures 80 and 81 provide an illustration of the test method.

(j) Fluid penetrability and moisture content methods

Many concrete degradation mechanisms involve the penetration of aggressive materials, such as sulphates, carbon dioxide and chloride ions. In most cases, water must also be present to sustain the mechanism. Low permeability is important to concrete durability. As a result, the condition of the surface zone is a key factor in concrete durability [383]. There are three ingress mechanisms by which external agents can penetrate concrete:

FIG. 80. Rebound hammer test, Digi-Schmidt test hammer (courtesy of Proceq SA).

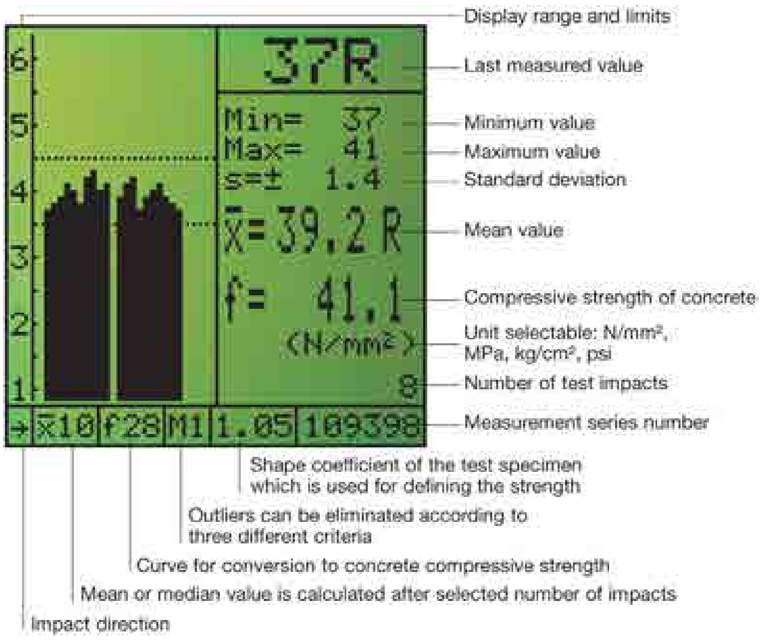


FIG. 81. Rebound hammer test; Digi-Schmidt example results displayed (courtesy of Proceq SA).

- — Absorption: ingress of liquids due to capillary forces;
- — Permeation: flow of fluid under action of pressure head;
- — Diffusion: movement of molecular or ionic substances from regions of higher concentrations to regions of lower concentrations.

Several methods have been developed to assess the ability of concrete surface zones to resist passage of external agents that may lead to concrete deterioration or depassivation of rebar. The methods are based on water absorption, water permeability or air permeability. Absorption tests measure rates at which water is absorbed into concrete under a relatively low pressure head. Initial surface absorption, Figg water absorption and covercrete absorption are examples of water absorption tests. Water permeability tests use higher pressures to obtain indications of permeability coefficients. Water permeability tests measure water flow into a concrete surface under a fixed pressure and include the Clam or Autoclam test [384] and the Steinert guard ring method [385]. Air permeability tests are based on the flow of air or other gases through concrete and include Figg air permeability, Schönlin and surface air tests. Both water and air based tests involve drilling a hole into the concrete or applying a chamber onto the surface.

The limitations of these methods include:

— Sensitivity to moisture and temperature changes;
— Changes in transport mechanisms during testing;
— Variance of air permeability with applied pressure;
— Influence of drilling on test values.

Figure 82 presents an initial surface absorption test apparatus for measuring water flow into a concrete test specimen through a known surface area, and a graph of example results obtained. Figures 83 and 84 present equipment for determining concrete water and air permeability.

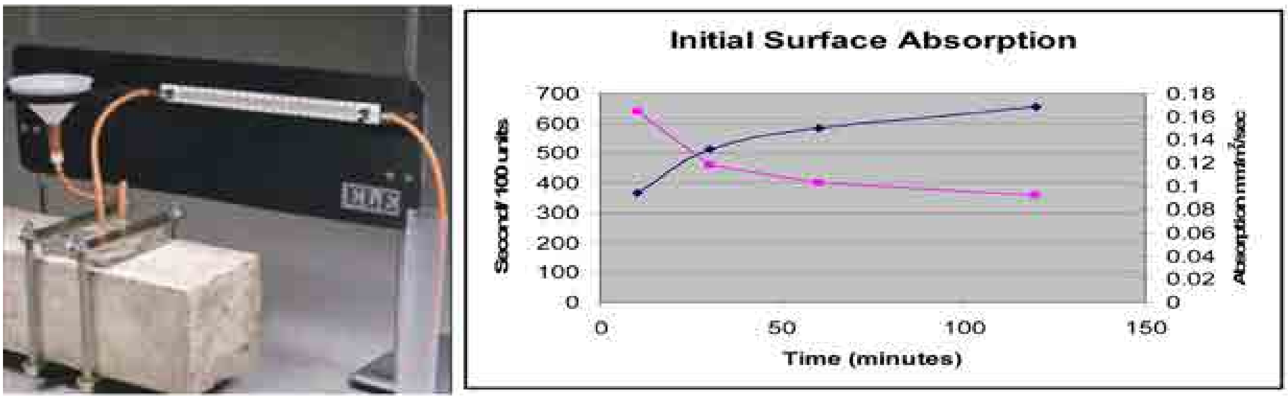


FIG. 82. Measurement of initial surface absorption of concrete (test set-up and results) [386].

FIG. 83. Water permeability test kit [387].

Neutron moisture gauges, chemically based humidity indicators, dew point sensors or electronic meter capacitance probes provide varying degrees of accuracy for quantitative moisture content assessments. With the exception of neutron moisture gauges, these techniques require insertion into surface drilled holes. Neutron magnetic

FIG. 84. Torrent air permeability apparatus (courtesy of Proceq SA).

resonance is based on counting low energy neutrons (resulting from hydrogen being present in water) upon a beam of high energy neutrons directed at the concrete [388]. Signal amplitude is a measure of hydrogen density and thus moisture content. It can be applied to one side of a structure and provides depth resolved information about structure internal makeup, such as moisture depth profile [389]. Figure 85 presents a neutron magnetic resonance measuring system for one sided access application and example results.

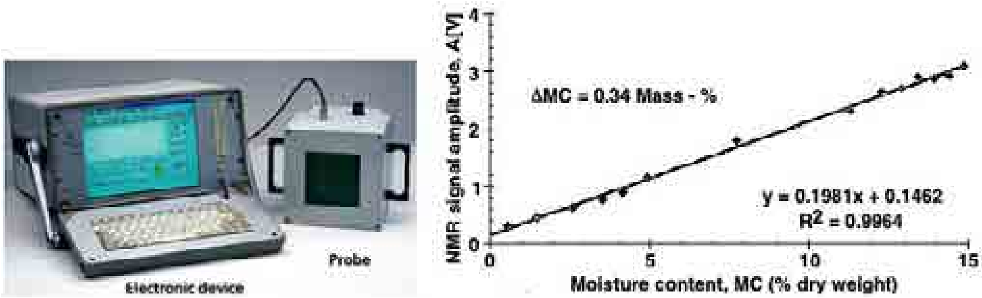


FIG. 85. Concrete moisture content determination using neutron magnetic resonance (equipment on left, water content calibration curve on right) [389].

7.2.1.2. Destructive or partially destructive methods

Destructive or partially destructive testing can be used to:

- Determine concrete strength, density and quality;
- Locate voids or cracks in concrete;
- Locate rebar and determine depth of concrete cover;
- Detect steel reinforcing material corrosion.

Testing techniques include:

- Core testing;
- Break-off;
- Probe penetration;
- Pull-out;
- Pull-off;
- Chemical analysis (e.g. chloride ion content, carbonation depth, alkali–aggregate reactions and sulphate attack);

— Laser induced breakdown spectroscopy;
— Petrography.

(a) Core testing

Removal and evaluation of concrete core samples from structures provides a direct method of concrete examination. Requirements for obtaining sufficient numbers of concrete samples for statistical evaluations are generally described in national codes and standards for building and construction [390]. When cores are removed from areas exhibiting distress, strength tests and petrographic studies (discussed later in this section) can be used to investigate causes and extent of deterioration. Concrete cores can be used to calibrate NDT devices, and down-hole cameras can be used to examine structure interiors where concrete cores were removed. The limitations of this method include:

— The number of samples that must be removed to meet statistical probability requirements (related to ensuring strength is below or above a certain level);
— The fact that results can be influenced by several factors (e.g. aggregate size, core diameter and slenderness ratio);
— The potential need to repair areas where cores are removed.

Figure 86 illustrates obtaining and testing a core sample.

FIG. 86. Obtaining and compression testing a concrete core sample [391].

A modern and advanced method for investigating core sample content is X ray micro tomography. Reference [392] discusses how this method has been used for determining the orientation of short fibres in steel fibre reinforced concrete. Core samples were taken from concrete slabs and an image of the sample fibres was obtained directly in 3-D. Orientation of each individual fibre was determined based on a skeletonized representation of this image.

(b) Break-off test

The break-off method is used in situ, primarily as a quality control test for concrete, and makes a direct determination of flexural strength in a plane parallel to and at a certain distance from the concrete surface. Break-off stress at failure can be related to concrete compressive strength using a predetermined relationship for a particular source of concrete. To perform the test, a specimen of 55 mm diameter and 70 mm deep is formed either by using a plastic cylinder placed into the fresh concrete, or drilling a core with the same outer dimensions into existing concrete. A load cell is placed into a circular groove at the top surface and load is applied using a hydraulic pump until failure of the specimen occurs in flexure. The pressure reading of the load cell is correlated to concrete strength by using calibration curves. Method limitations are that it cannot be used with concrete mixes with maximum aggregate sizes exceeding 19 mm, or on concrete structures with sections less than 100 mm thick. Figures 87 and 88 provide an illustration of the break-off test equipment [393] and method [388].

FIG. 87. Break-off test equipment [393].

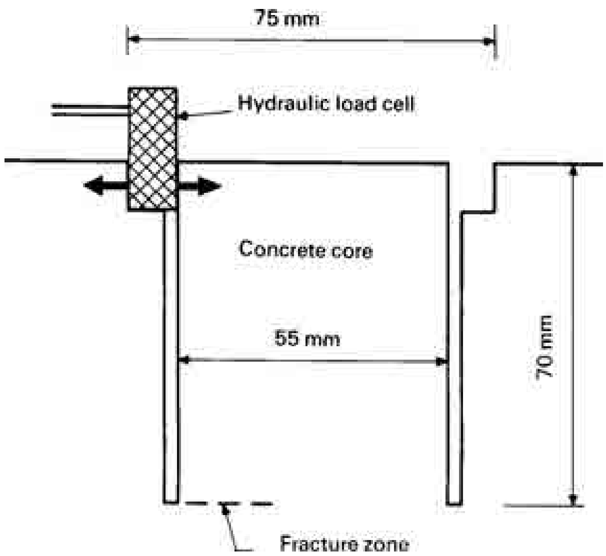


FIG. 88. Break-off test method schematic [388].

(c) Probe penetration

Probe penetration tests estimate concrete compressive strength, uniformity or general quality through measurements of concrete resistance to penetration of a steel probe that is driven by a given amount of energy [394].

Compressive strength is determined using calibration curves. The shallower the penetration depth, the stronger the concrete. The advantages of this method are that it is relatively simple and results correlate fairly well to concrete compressive strength. The limitations are that specimen thickness has to be at least three times the penetration depth, the method should ideally not be applied within about 200 mm of specimen edges or other tests and aggregate size and hardness influence results. Rebar must be avoided. Figures 89 and 90 illustrate the application of the Windsor probe [395, 396].

FIG. 89. Windsor probe test technique [395].

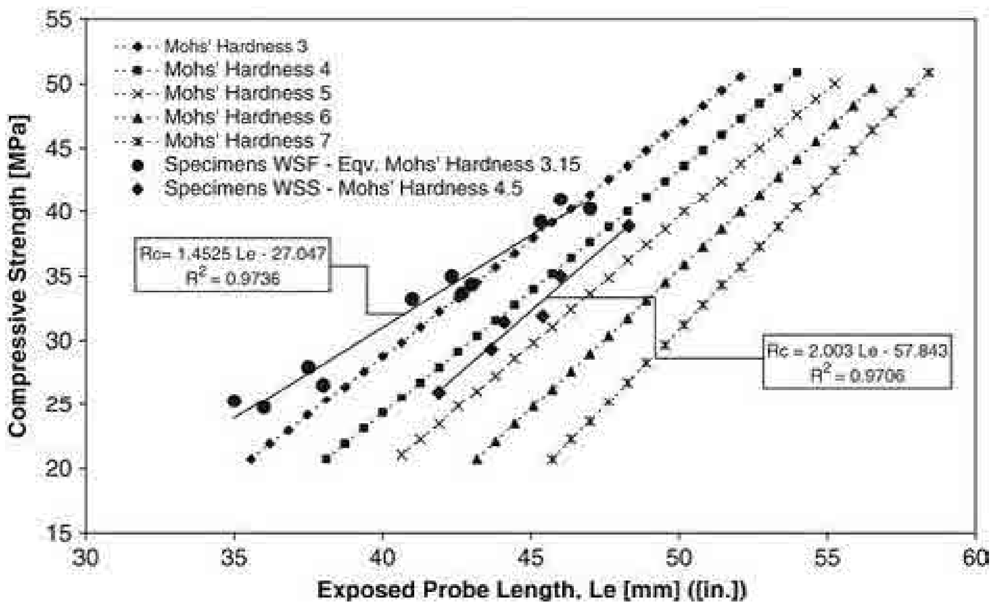


FIG. 90. Concrete compressive strength as a function of Windsor probe exposed probe length [396].

(d) Pull-out test

Originally known as cast-in-place pull-out, this test is performed by using a hydraulic device pulling an embedded metallic insert with an enlarged head from the concrete. Concrete compressive strength is related to pull-out force through calibration curves. Failure involves fracture, and often removal, of an approximately cone-shaped portion of concrete. Developments have eliminated the requirement that pull-out inserts be cast into the specimen. Several forms of the test exist: internal fracture test, LOK test and CAPO test. The internal fracture test involves drilling a hole about 30–35 mm deep into the concrete, tapping a 6 mm wedge anchor bolt with an expanding sleeve into the hole until the sleeve is 20 mm below the surface and loading the bolt at a standard rate against a reaction ring until the load required to fracture the concrete is achieved [388]. Calibration charts are used to relate force to compressive strength. The LOK test was developed in Denmark to estimate workmanship effects in addition to potential strength as measured by cylinders or cubes [397]. An insert consisting of a steel sleeve attached to a 25 mm diameter, 8 mm thick anchor plate located at a depth of 25 mm below the concrete surface is cast into the concrete. Estimated concrete cube or cylinder compressive strength is obtained through calibration charts using the force required to cause failure by pulling the disk out of the concrete. The CAPO test was developed as a version of the LOK test that can be applied to existing concrete [388, 397]. A 45 mm deep, 18 mm diameter hole is drilled, after which a 25 mm groove is cut at a depth of 25 mm using a portable milling machine. An expanding ring insert is placed into the groove and expanded using a jack until concrete pull-out occurs. Compressive strength is estimated from correlation curves. Test limitations are that results are affected by size of coarse aggregate, and a correlation relationship between pull-out strength and compressive strength is generally required for each application. Also, some repair may be required. Figures 91, 92 and 93 provide illustrations of the internal fracture test, LOK test, and CAPO test, respectively. Accuracy of the internal fracture test method is on the order of ±30% with a coefficient of variation of about 15%. Accuracy of the LOK test is on the order of ±20% with a coefficient of variation of about 8%. Accuracy of the CAPO test method is on order of ±20% with a coefficient of variation of about 7% [388].

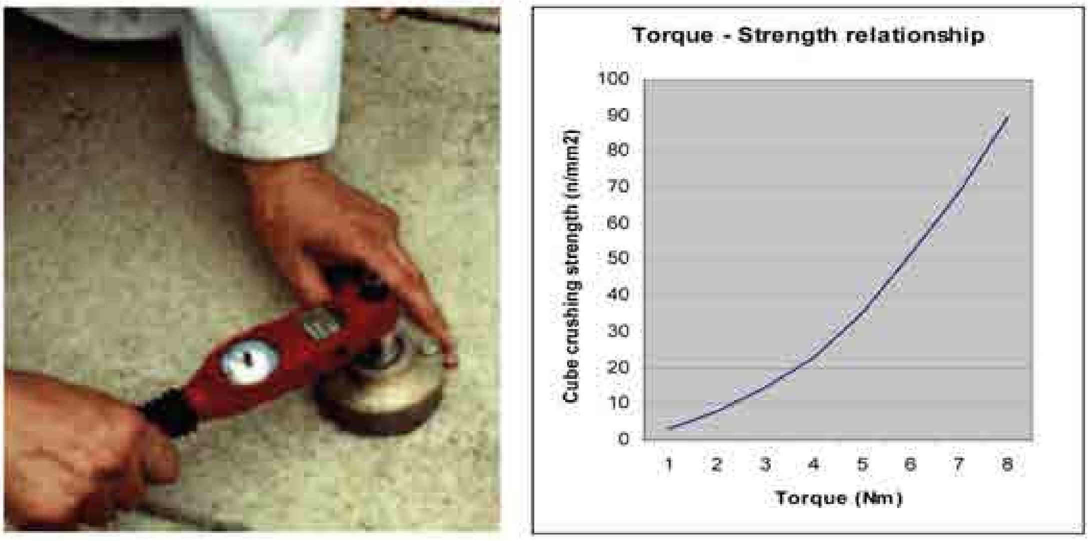


FIG. 91 . Internal fracture test fixture [398] and strength–torque relationship example (courtesy of Testconsult [399]).

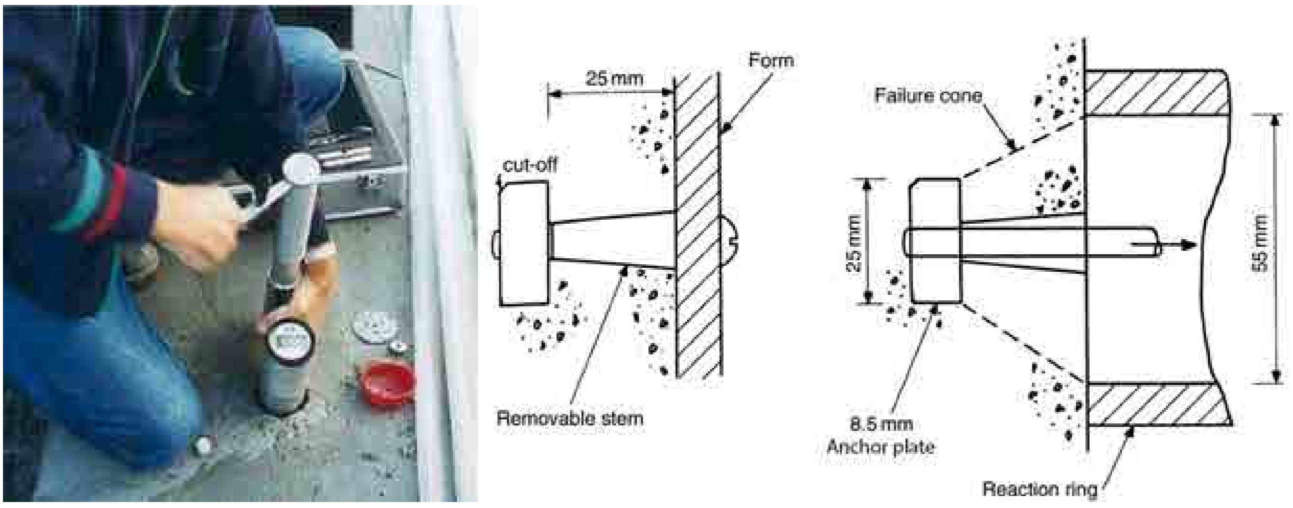


FIG. 92. LOK test equipment (left) [400]; process (right) [388].

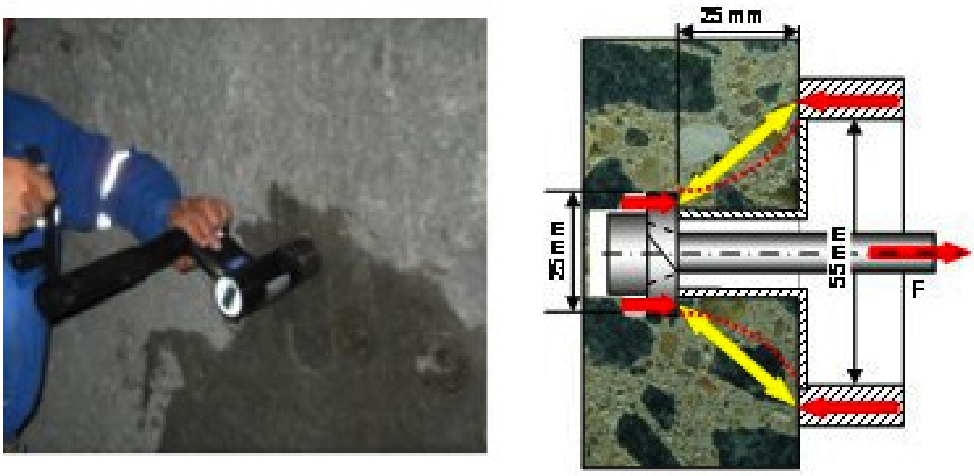


FIG. 93. CAPO test [401].

(e) Pull-off test

The pull-off method has been developed to measure in situ tensile strength of concrete through the application of direct force. A disk is glued to the concrete surface with an epoxy resin and jacked off to measure the force necessary to pull a piece of concrete away from the surface [388]. Partial coring to an approximate depth can be used if surface carbonation or skin effects are present. Nominal concrete strength is calculated on the basis of probe diameter (this may be converted to compressive strength using a calibration chart appropriate to the particular concrete) and whether coring was used. In conjunction with partial coring, the test is particularly suited to assess bond strength of repair overlays. Figure 94 provides illustrations of the pull-off test.

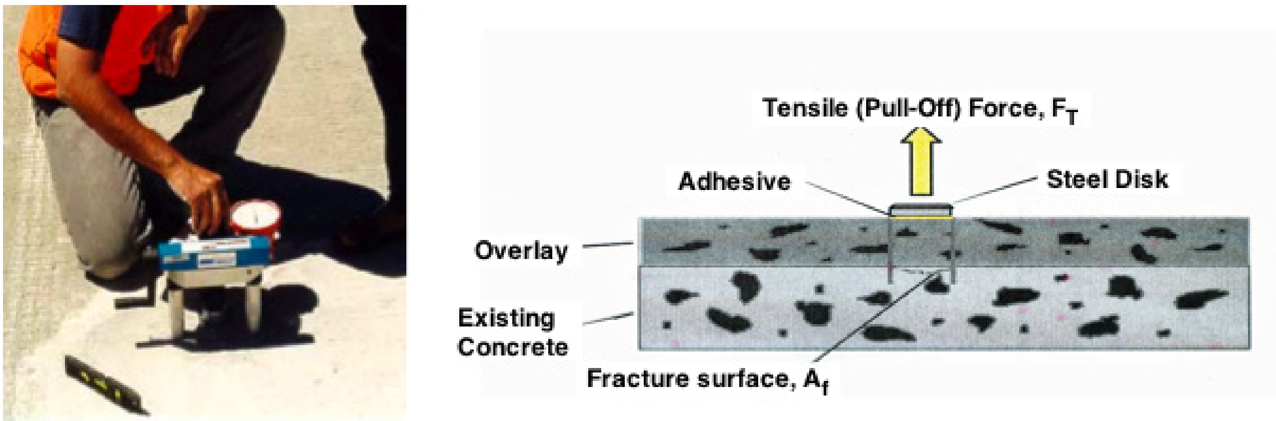


FIG. 94. Pull-off test: equipment (left); schematic of test to determine bond strength of overlay (right) [402].

7.2.1.3. Chemical analysis

Chemical analysis can be used to identify causes of deterioration such as chlorides, carbonation, AAR or sulphate attack. Chemical analysis of concrete will generally require use of specialized laboratory facilities and an experienced concrete analyst. Most likely sources of imprecise results are:

— Aggregate contributions;
— Unusual and unknown cement composition;
— Chemical attack;

— Extraneous materials;
— Inadequate sampling procedures [388].

In addition to the items discussed in the following, chemical analysis is useful for identifying cement type and content, aggregate type and gradation and original water content.

(a) Chloride ion content

In good quality, well-compacted concrete, reinforcing steel with adequate cover should not be susceptible to corrosion because the highly alkaline conditions present within the concrete (pH>12) causes a passive iron oxide film (gamma Fe_2O_3) to form on the surface (i.e. metallic iron will not be available for anodic activity). Chloride ions, however, that may be present in the concrete mix as an admixture, be absorbed from the surface, or enter through cracks, can induce local disruption of the passive steel layer, leading to pits or localized attack. The chloride ions exist in two forms: chemically bound and soluble in concrete pore water. Determination of concrete chloride ion content, therefore, is an important aspect of the analysis of concrete structures relative to the potential for corrosion of embedded rebar. Two of the most commonly used methods for determination of chloride content in concrete are the water-soluble and total-chloride tests [403, 404].

The water-soluble test involves obtaining concrete samples by coring or drilling and grinding the sample to produce a powder. The powder is boiled in water for five minutes and soaked for twenty-four hours. The water is then tested for dissolved chlorides and is presented as a percentage of cement or concrete. The water-soluble chloride test measures only free ions soluble in the pore water. These ions are linked to initiation of corrosion. However, the water-soluble chloride test is not very accurate or repeatable, so the total-chloride test is normally used [405].

The total-chloride test is an acid-soluble test and involves digesting a ground sample of hardened concrete in nitric acid. The solution is then tested for chloride content and is presented as a percentage by weight of the material being analysed. The acid-soluble test measures the sum total of all chemically bound and free chloride ions in concrete. Results of chloride content analysis are reported as either percentage of chloride by weight of concrete, parts per million of chloride ions, percentage of chloride per weight of cement, or weight of chloride per volume of concrete. Other methods used to determine concentrations of chloride ion in concrete include:

— X ray fluorescence;
— Visible spectrophotometry;
— Atomic absorption spectrophotometry;
— Neutron activation analysis;
— Potentiometric titration;
— Potentiometry by standard additions;
— Ion chromatography;
— Quantab chloride titrator strips;
— Laser induced breakdown spectroscopy;
— Measurements of speed of propagation and damping characteristics of electromagnetic waves in concrete [388, 406–409].

Primary limitations of these methods are that they may require coring or drilling to obtain samples at locations in a structure where chloride ion contents are desired, and the chloride content reported includes chlorides that were present in the concrete mix constituents.

(b) Carbonation depth

Although carbonation has an initial effect of increased compressive strength, modulus of elasticity, surface hardness and resistance to frost and sulphate attack, it reduces concrete alkalinity leading to rebar corrosion. Depth of carbonation can be easily determined either in situ or in a laboratory by treating a freshly broken concrete surface with phenolphthalein. Carbonated portions will be uncoloured. Periodic determinations can be used to establish penetration rates. Incrementally drilled powder samples can also be extracted and sprayed with phenolphthalein.

Presence of carbonation can also be assessed by acid etching [410] or microscopic analysis of thinly cut concrete sections [411, 412]. Carbonation testing often is performed in conjunction with chloride ion content determinations. The limitation of this method is that it requires the exposure of a fresh concrete surface for each test. Figure 95 shows a phenolphthalein reaction with carbonated concrete.

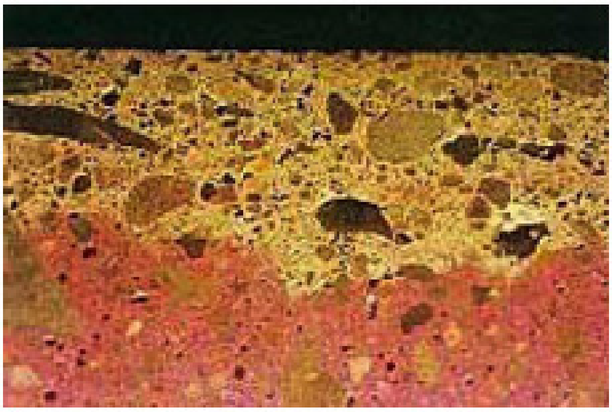
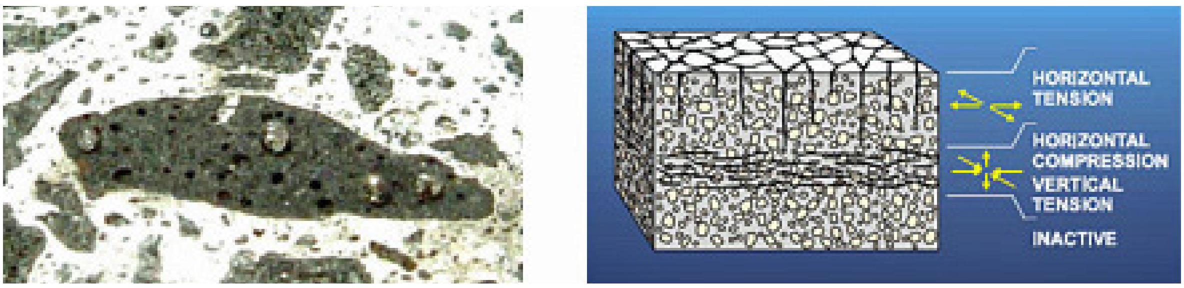


FIG. 95. Carbonation of concrete (carbonated portion is uncoloured) [413].

(c) Alkali–aggregate reactions

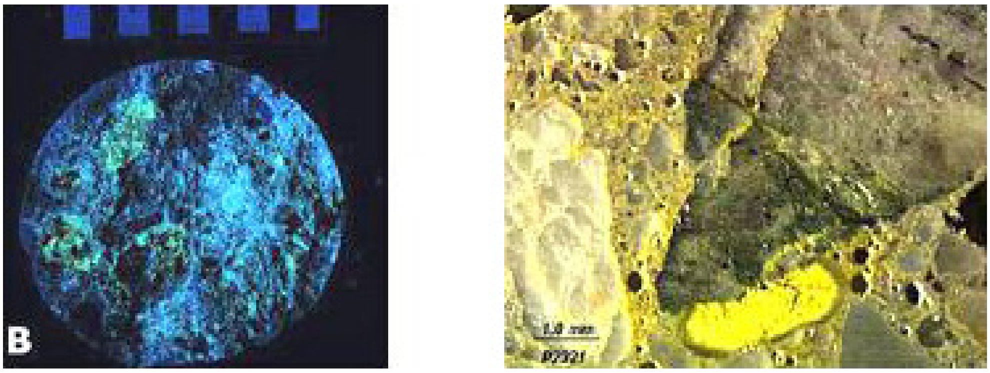
Expansion and cracking leading to loss of strength, stiffness and durability of concrete can result from chemical reactions involving alkali ions from Portland cement, calcium and hydroxyl ions and certain siliceous constituents in aggregates that form a calcium alkali-silicate gel. This gel takes up pore solution water due to the forces of attraction between the polar water molecules and the alkali-silicate ions and expands, which can disrupt the concrete. Visual examination is a critical part of diagnosing ASRs in concrete structures. Key features to address include: environmental conditions, nature and extent of cracking, pop-outs, movements, displacements and deformations, surface discolouration and surface deposits [414]. Cores can be obtained in suspect areas for petrographic studies and mechanical testing. Two tests are available for assessing the potential presence of ASR. One test is for sodium using a uranyl acetate solution [415] and another for potassium using a sodium cobaltinitrite solution [416]. The uranyl acetate test uses the principal of ultraviolet fluorescence of the uranyl ion. The sodium cobaltinitrite solution has a strong yellow colour in normal light. Figure 96 shows the gel resulting from AAR that causes expansion and cracking and the cracking patterns that can develop [413, 417].



(a) (b)

FIG. 96. Alkali–silica reactivity: (a) droplets of alkali gel emerging from porous basaltic aggregate; (b) idealized sketch of cracking pattern in mass concrete caused by internal expansion due to AAR ([417] and [413], respectively).

Figure 97 presents examples of the application of these solutions to freshly broken concrete surfaces. These tests serve as ancillary to petrographic examinations, as positive indications in these tests merely signify the presence of the particular ion in an exchangeable form in the substance being tested [413].



(a) (b)

FIG. 97. Alkali–silica reactivity: (a) droplets of alkali gel emerging from porous basaltic aggregate; (b) idealized sketch of cracking pattern in mass concrete caused by internal expansion due to AAR ([417] and [413], respectively).

(d) Sulphate attack

All sulphates are potentially harmful to concrete. Sulphate attack is caused by exposure of concrete products or structures to an excessive amount of sulphate from internal or external sources. The degree of sulphate attack depends on water penetration, the sulphate salt and its concentration and type, the means by which the salt develops in the concrete (e.g. rising and drying causing crystallization) and the chemistry of the binder present in the concrete. Examples of laboratory techniques for identification of the presence of sulphates include chemical analysis, optical microscopy, scanning electron microscopy, electron probe microanalysis, differential thermal analysis and X ray powder diffraction. Sulphate compounds often exist as cryptocrystalline, clear to white secondary deposits in voids or cracks and can be distinguished using a barium chloride potassium permanganate solution [418]. If sulphate ions are present, $BaSO_4$ will precipitate, trapping permanganate in the crystal structure, thus staining purple as shown in Figs 98 and 99.

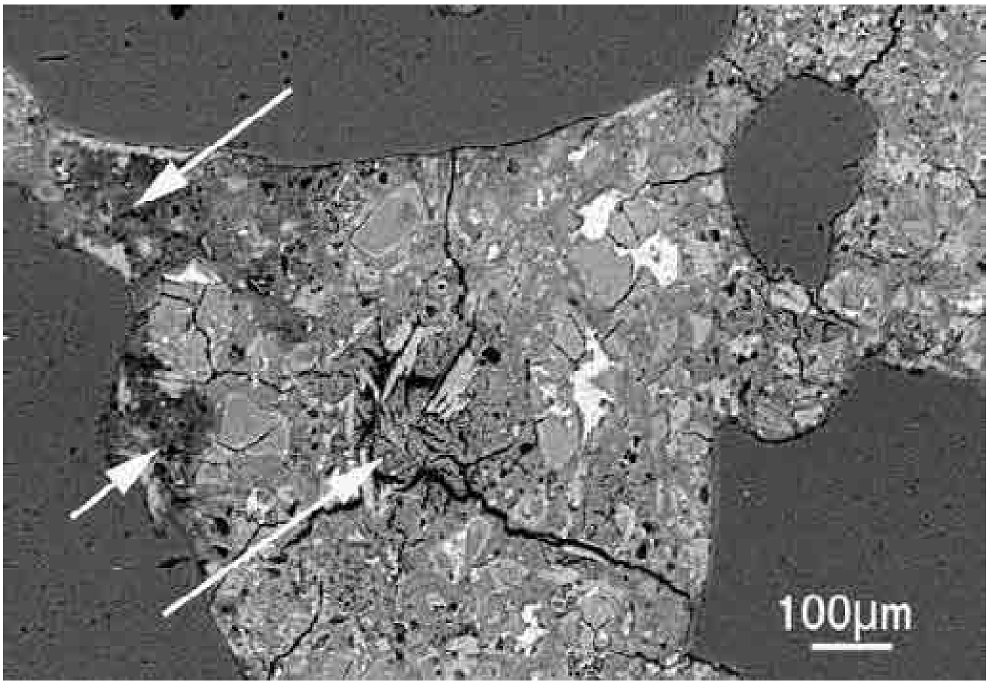


FIG. 98. Sulphate attack in concrete where ettringite (arrows) has replaced some of the calcium silicate hydrate in cement paste [129].

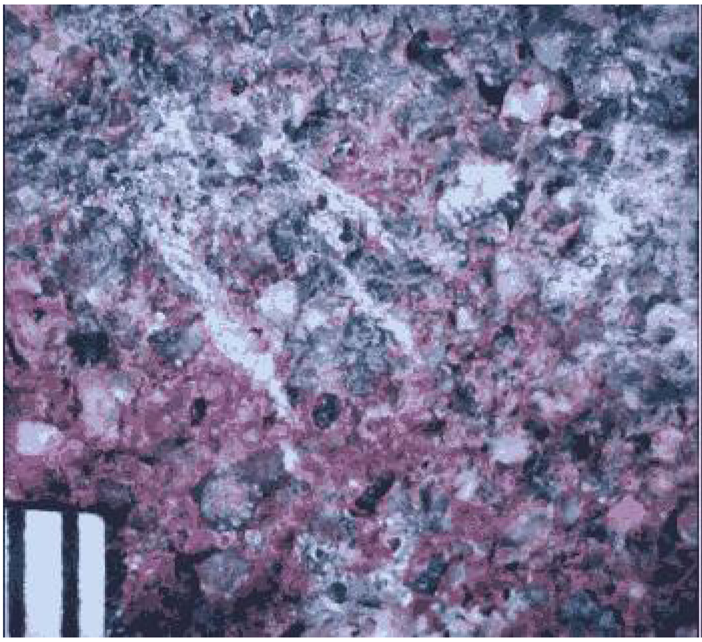


FIG. 99. Fractured surface of concrete treated with KMnO4 (secondary deposits containing sulphate are stained purple) [413].

(e) Laser induced breakdown spectroscopy method

Laser induced breakdown spectroscopy is a real time method that can be used to identify the elemental composition on the surface of building materials (i.e. to characterize concrete) [419]. A short intense infrared laser pulse is used to ablate and vaporize a small amount of material. A plasma plume is formed and during the relaxation process, atomic emission occurs and element specific spectra features can be observed. The emitted fluorescence light is directed to a detection unit by an optical fibre. Light intensity is measured as a function of wavelength (i.e. spectrum) [420]. Results from measurements on concrete specimens can be represented on a Rankin diagram. Points are plotted according to determined contents for Si, Ca, Mg, Al and Fe for each spectrum. Cement type can be identified by positions in the Rankin diagram. In order for quantitative determinations to be made of element contents, a database containing calibration measurements, on reference materials with defined composition, is required. In addition to identification and characterization of concrete and cements, applications of laser induced breakdown spectroscopy include:

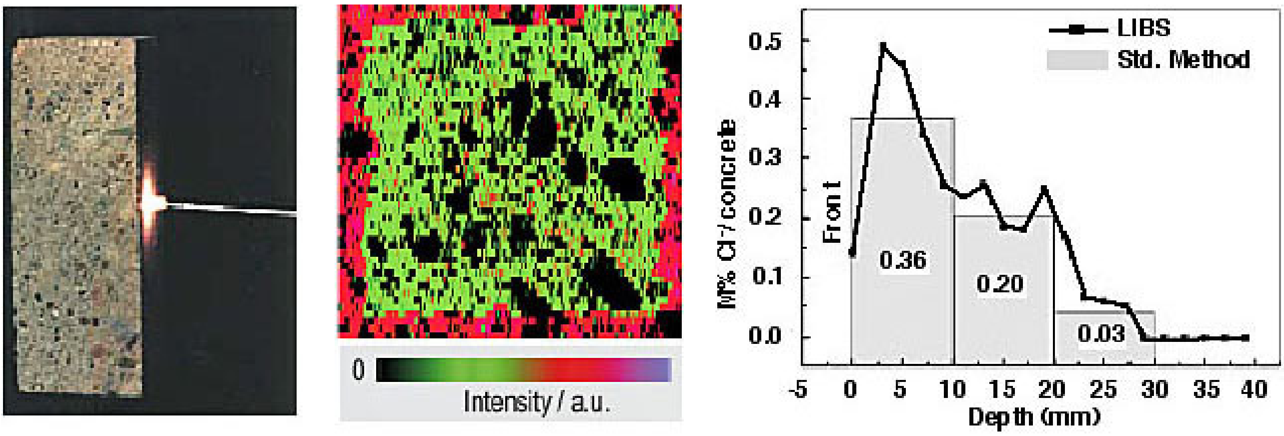
— Measurement of chloride contents at different depths for durability assessments;
— Measurement of sulphur content at different depths;
— Determination of heavy metal contents;
— Identification of surface layers [420].

Figure 100 presents the application of the laser pulse to a concrete specimen. Also shown is an area scan of a broken concrete surface showing carbon distribution and a comparison of chloride profiles obtained by standard chemical analyses and laser induced breakdown spectroscopy.

(f) Petrography

Petrographic examinations of samples of hardened concrete removed from existing concrete structures can provide valuable information for use in an AMP. Petrographic methods combine unaided eye inspections and microscopic examinations using stereo and metallographic microscopes. Several purposes for which petrographic examinations of structures may be conducted include:

— Detailed determination of concrete condition in a structure;
— Determination of causes of inferior quality, distress or deterioration;



(a) (b) (c)

FIG. 100. Application of laser induced breakdown spectroscopy with examples of obtainable results: (a) application of laser pulse to concrete [419]; (b) carbon distribution in carbonated concrete [421]; (c) comparison to chloride profiles obtained by chemical analysis [420].

— Determination of whether the concrete in the structure was or was not as specified;
— Description of cementitious materials matrix (e.g. kind of binder, degree of hydration, nature of hydration products and presence of mineral admixtures);
— Determination of presence of alkali–aggregate reactions;
— Determination if concrete has been subjected to chemical attack or early freezing;
— Determination of the nature of the air void system;
— Survey of a structure relative to its safety [410].

Although approximate water to cementitious materials ratios and cement content can be estimated, more accurate values require applications such as chemical techniques, X ray diffraction analysis or scanning electron microscopy techniques. The disadvantage of petrographic examinations is that they require removal of samples from the structure for testing. Figure 101 presents a flow chart of the petrographic examination process and its application to determination of air void content in hardened concrete.

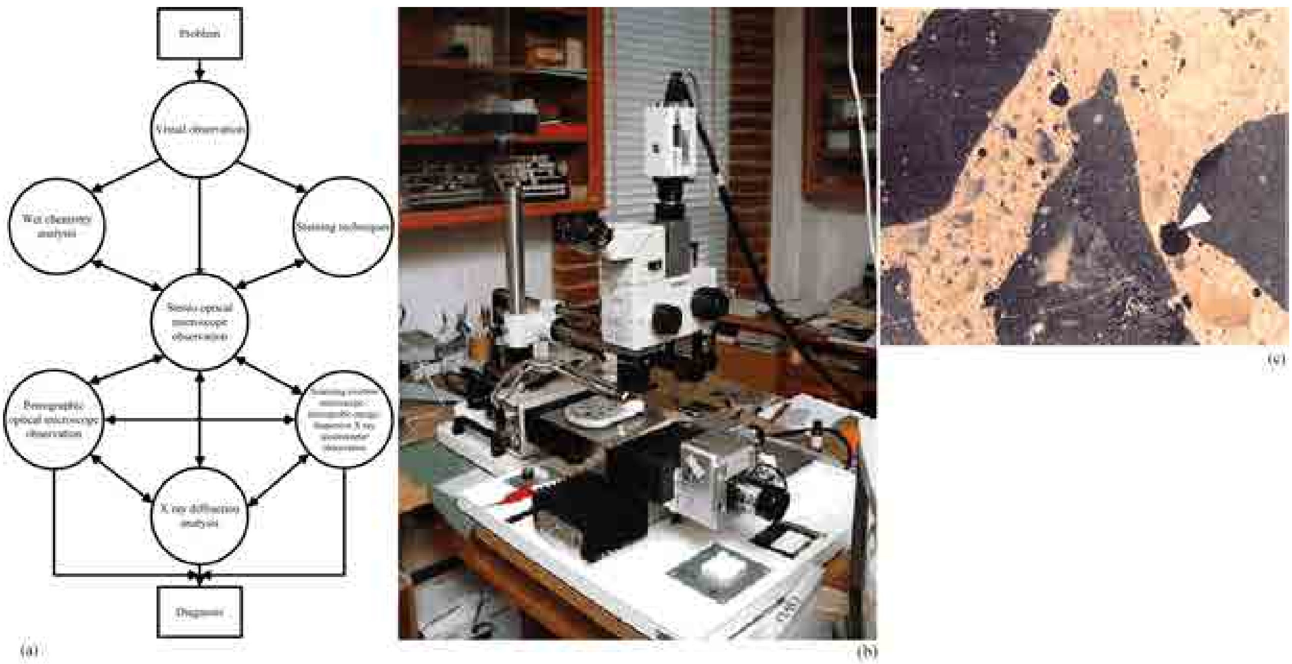


FIG. 101. Images related to petrographic examination: (a) the process flowchart; (b) linear traverse equipment for determining air void parameters; (c) a concrete surface with air void [413].

7.2.2. Mild steel reinforcement (rebar)

Corrosion of reinforcing steel is probably the main threat to durability of reinforced concrete structures. The mechanism of corrosion in aqueous media is of electrochemical nature with the two main causes being carbonation of concrete and presence of chlorides [422]. The identification and mitigation of corrosion occurrence are important to structural performance because corrosion can cause a loss of structural capacity due to a reduction in rebar cross-section, a reduction of bond strength between the rebar and concrete, and a loss of concrete integrity resulting from cracking and/or spalling of cover concrete.

Assessments of mild steel reinforcing are primarily related to determining its characteristics (e.g. location and size) and evaluating corrosion occurrence. There are a number of techniques that can be used to carry out a condition survey of reinforced concrete structures and to assess the potential for corrosion [423]. Table 19 presents a summary of the more widely used techniques for conducting a condition survey [424]. In addition to identification of the technique, information is provided in the table on what the technique detects, user requirements and approximate application speed. Each technique, whether used in isolation or in combination, provides an assessment of the structure at the time of measurement. Corrosion monitoring, or condition surveys conducted over a period of time, are required to provide an indication of changing conditions.

Several methods identified in Table 19 have been addressed previously. Only techniques related to the evaluation of corrosion are addressed in this section. These include:

- Half-cell potential measurements;
- Surface potential measurements;
- Electrical resistivity measurements;
- Polarization methods;
- Tafel extrapolation;
- Electrochemical impedance spectroscopy;
- Embeddable corrosion monitoring sensors [425].

TABLE 19. TYPICAL TECHNIQUES USED FOR CORROSION ASSESSMENT OF REINFORCED CONCRETE [424]

Technique	Item detected	User requirements	Application speed
Visual	Surface defects	General	1 m^2/s
Hammer/chain drag	Delaminations	General	0.1 m^2/s
Covermeter	Rebar depth and size	General	1 reading in 5 min
Phenolphthalein	Carbonation depth	General	1 reading in 5 min
Chloride content	Chloride induced corrosion	General + laboratory	1 reading in 5 min + lab/site analysis
Permeability	Diffusion rate	Corer + specialist	Time for coring + laboratory
Impact/ultrasonics	Defects/concrete quality	Specialist	1 reading in 2 min
Petrography	Concrete properties	Corer + specialist	Time for coring + laboratory
Half-cell potential	Corrosion risk	General/specialist	1 reading in 5 sec
Linear polarization	Corrosion rate	Specialist	1 reading in 5–30 min
Resistivity	Concrete resistivity	General/specialist	1 reading in 20 sec

7.2.2.1. Half-cell potential

Electrical methods are used to evaluate rebar corrosion activity [426, 427]. When rebar is corroding, electrons flow through the bar and ions flow through the concrete. As a result, the corrosion potential of rebar will shift negatively if the surface changes from the active to the passive state [428]. When the bar is not corroding, there is no flow of electrons and ions. The half-cell potential method is used to detect this negative charge and to thereby provide an indication of corrosion activity. Figure 102 illustrates the half-cell potential measurement technique. Corrosion rate depends on the potential difference between anode and cathode, which is affected by a number of factors (e.g. O_2 content, Cl^- concentration and stray electrical currents). Potential measurements at a number of locations on the concrete surface using a reference half-cell (e.g. copper–copper sulphate) connected to the rebar are used to indicate likelihood of corrosion occurrence (i.e. greater than 90% probability of no corrosion, corrosion activity is uncertain or greater than 90% probability that corrosion is occurring).

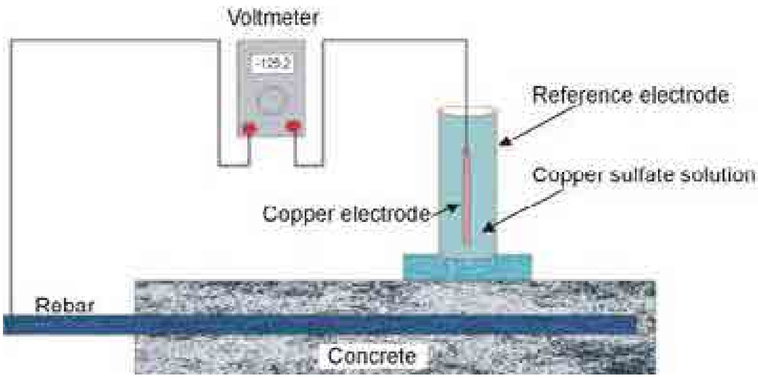


FIG. 102. Half-cell potential technique [429].

The concrete surface being investigated is usually divided into a grid system to define measurement locations. Results generally are plotted in the form of an equipotential diagram so that areas potentially exhibiting corrosion can be readily identified. Examples of a half-cell instrument and half-cell potential results are provided in Figs 103 and 104.

Since potential criteria for corrosion likelihood are not applicable to all structures and apply to particular environments, potential gradients (contour maps) are commonly used to indicate areas of active corrosion (e.g. >100 mV warrants further investigation and >200 mV indicates corrosion activity) [427]. Modified types

FIG. 103. Half-cell potential instrument [429].

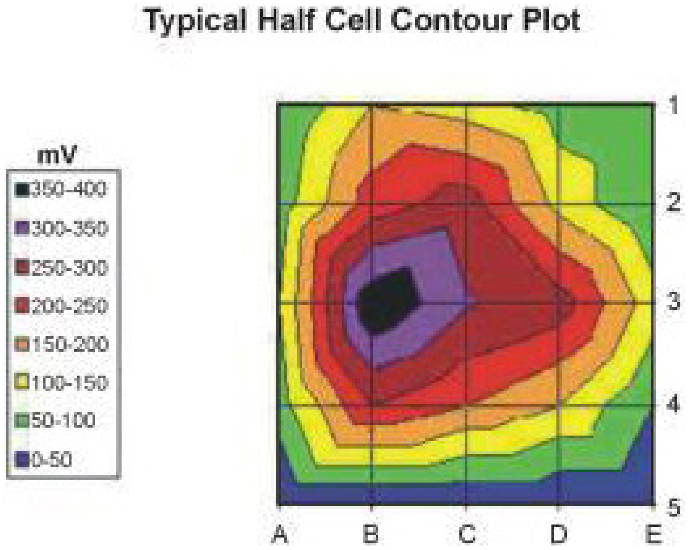


FIG. 104. Example of half-cell potential survey (corrosion suggested at high negative voltage locations) [429].

of instrumentation, consisting of a number of half-cells mounted in parallel or on a roller bar, have been developed to accelerate the examination process. Primary limitations of this method include the following:

— Neither the magnitude nor rate of corrosion is provided;
— Surface coatings or coated rebar present problems;
— Measurements are affected by temperature and moisture;
— Electrical continuity is required;
— Concrete constituents can affect results (e.g. type of cement and chloride ingress).

The half-cell potential method, despite its sensitivity to moisture level, is a useful indicator for locating concrete surface areas where risk of rebar corrosion is high. It is a useful complement to other techniques in selecting test points for further analysis such as chloride measurements. It may be used to detect adverse external effects such as leakage currents that may be detrimental to rebar, or to monitor the effect of cathodic protection systems. Results in Sweden from potential mapping of bridge piers in a marine environment indicate that there may be a linear relationship between measured chloride content and observed half-cell potential [430].

7.2.2.2. Surface potential measurements

While corrosion is occurring, an electric current flows between cathode and anode through the concrete. This flow of current can be detected by measuring the potential drop in the concrete (see Fig. 105). Two reference

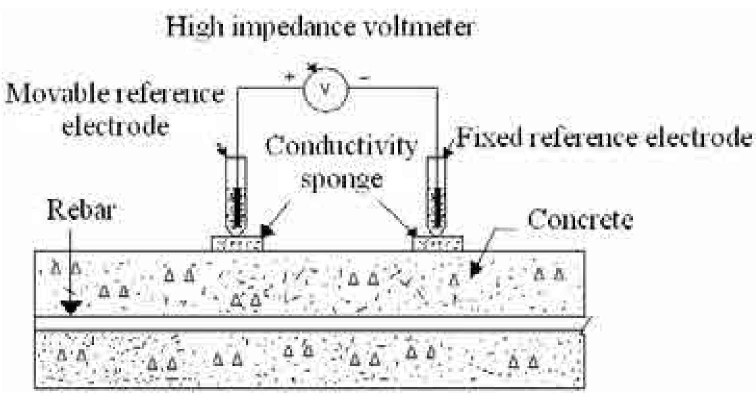


FIG. 105. Schematic of surface potential measurement [425].

electrodes are used for surface potential measurements, with one electrode remaining fixed while the other electrode is moved over the structure according to a grid. A more positive potential reading represents an anodic area where corrosion is possible. The greater the potential difference between anodic and cathodic areas, the greater the probability of corrosion [425]. This technique differs from the half-cell technique in that an electrical connection to the steel reinforcement is not required.

7.2.2.3. Electrical resistivity measurements

Electrical resistivity is defined as the ratio between applied potential and current circulating between two electrodes, providing the arrangement enables the calculation of geometrical characteristics [422]. Electrical resistivity provides an indirect measurement of pore porosity and connectivity (e.g. measures ease with which ions migrate through concrete) [431]. Electrical resistivity measurements are used to identify wet concrete areas and thus corrosion risk. Highly permeable concrete will have high conductivity and low electrical resistance. Under field conditions, there is a correlation between concrete resistivity and steel corrosion rate [432]. Conditions such as high pore water content and electrolyte salt presence, which lead to low resistivity, usually favour corrosion.

Concrete resistivity can be measured using a four point technique, as illustrated in Fig. 106. Four equally spaced probes are installed in a straight line on the concrete to be tested with electrode spacing equal to the depth to which measurement of average resistivity is desired. Average resistivity is a function of voltage drop between the centre pair of probes with current flowing between the outside probes. Resistivity is then determined (e.g. in ohm·m or ohm·cm). Corrosion likelihood is related to the resistivity value measured (i.e. >200 ohm·m is negligible, 100–200 ohm·m is low, 50–100 ohm·m is high and <50 ohm·m is very high).

Results may provide an indication of concrete quality, as indicated by the amount of moisture present, and allow evaluation of other concrete characteristics such as chloride ion diffusivity [433, 434]. The limitation to this method is that resistivity measurements are obtained relatively close to the concrete surface, and when electrode spacing is increased to allow evaluations at deeper concrete depths, rebar may interfere with results obtained.

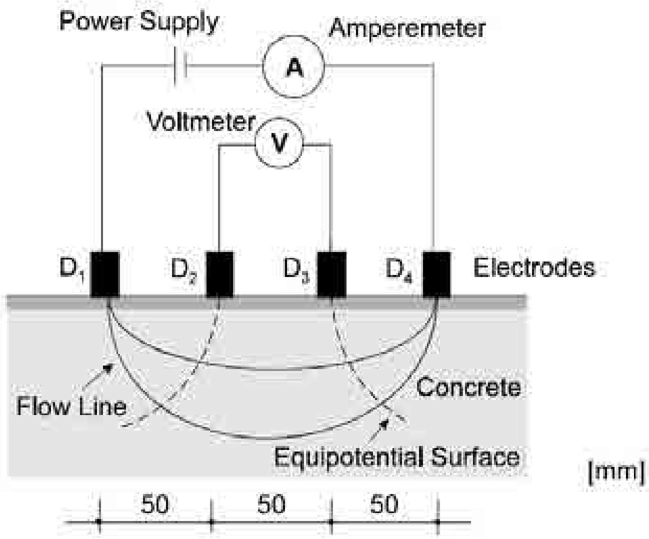


FIG. 106. Electrical resistivity measurements — four point technique [422].

Figure 107 shows an example Wenner four point probe instrument.

7.2.2.4. Linear polarization resistance methods

Polarization resistance of rebar embedded in concrete is defined as the ratio between applied voltage ΔE (shift in potential from E_{corr}) and the step in current ΔI, when the metal is slightly polarized (about 20–50 mV) from its free corrosion potential, E_{corr} [435]. It can be measured by means of alternating or direct current techniques. In polarization resistance testing, the current to cause a small change in the value of the half-cell potential of the

FIG. 107. Wenner four point probe (courtesy of Proceq SA).

corroding bar is measured. For small perturbations in the open circuit potential, a linear relationship exists between the change in voltage and the change in current per unit area of bar surface. A relationship exists between the corrosion rate of the rebar in concrete and the polarization resistance. Corrosion rate is usually expressed as the corrosion current per unit area of bar. Instantaneous corrosion current density, I_{corr} (mA/cm^2), is obtained by dividing a constant B (results from a combination of anodic and cathodic Tafel slopes (see Section 7.2.2.6)) by the polarization resistance value. Values of I_{corr} can be used to assess concrete structure degradation rates affected by rebar corrosion. Guidance related to rebar service life has been developed relative to the value of I_{corr} and is provided in Table 20. It is possible to convert the corrosion rate into the mass of steel that corrodes per unit of time [422]. If bar size is known, corrosion rates can be converted to losses in bar diameter. However, the method cannot be simply applied to real size structures because the area polarized is unknown. This is because the applied electrical signal tends to vanish with the distance from the counter electrode rather than spread uniformly across the working electrode. If metal is actively corroding and polarization resistance is low, the current applied from a small counter electrode located on the concrete surface is drained very efficiently by the rebar and it tends to confine itself to a small surface area. However, if the metal is passive and polarization resistance is high, the current applied tends to spread far away (e.g. ~50 cm) from the application point [422]. Therefore, an electrochemical method may provide reliable corrosion rates for actively corroding rebar, but difficulties are imposed in estimating the, typically, very low corrosion rates of passive rebar. Methods to determine the polarization resistance in large reinforced concrete structures can be classified into three groups: confinement of applied electrical signal, measurement

TABLE 20. RANGES OF CORROSION CURRENT VALUES RELATED TO SIGNIFICANCE IN TERMS OF SERVICE LIFE OF REINFORCEMENT [435]

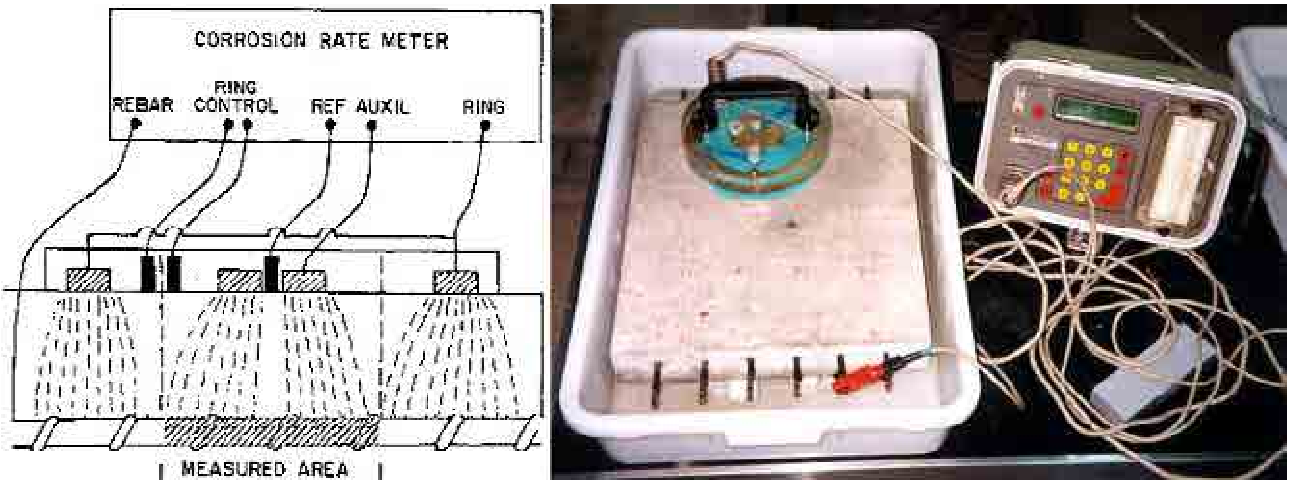
I_{corr}* (mA/cm^2)	V_{corr}** (mm/yr)	Corrosion level
≤0.1	≤0.001	Negligible
0.1–0.5	0.001–0.005	Low
0.05–1	0.005–0.010	Moderate
>1	>0.010	High

* I_{corr}: corrosion current density
** V_{corr}: corrosion rate

or estimation of the lateral spreading of the electrical signal and minimization of the effect of the lateral spreading of the electrical signal [422].

One of the most widely adopted solutions to limit the electrical current lines from the counter electrode to the working electrode is the modulated guard ring, whereby the applied signal is confined by using a ring shaped counter electrode surrounding the main electrode [436]. A measurement is made using the central counter to apply a galvanostatic step, lasting 30–100 seconds. Then, another counter current is applied from the external ring and the external current is modulated by means of two reference electrodes in order to equilibrate internal and external currents, enabling a correct calculation of the polarization resistance. Electrical current field lines that originate from the central counter electrode are confined within a known area of rebar by means of two reference electrodes placed between central and guard electrodes in order to control the confinement by modulating the electrical current applied from the external ring. Efficiency of confinement is continuously monitored by means of two extra reference electrodes placed between the central and external counter electrodes.

Figure 108 presents the principle of modulated confinement of electrical current [436]. Performance of the guard ring has been shown to be an improvement upon that of a single unconfined auxiliary probe.



(a) (b)

FIG. 108. (a) Principle of modulated confinement of electrical current; (b) application to corrosion rate measurements [436].

7.2.2.5. Galvanostatic pulse technique

The galvanostatic pulse technique is a transient polarization method working in the time domain for in situ application and can be used for detecting corrosion in wet and anaerobic environments [437–439]. The set-up is similar to the half-cell potential method and involves use of a counter electrode and reference electrode placed on the concrete surface above the rebar (see Fig. 109). A short duration anodic current pulse (typically ~3 s duration) with an amplitude of about 0.1 mA is impressed galvanostatically from the counter electrode that in turn shifts the rebar potential, with the shift recorded by a data logger [440]. Rebar is polarized in the anodic direction relative to its free corrosion potential. Extent of polarization depends on the corrosion state. Rebar is easy to polarize in the passive state, as noted by the large difference between free corrosion and polarized potential. The difference is much smaller when corrosion is occurring. The method is superior to the half-cell potential method, in particular when testing wet concrete where there is a risk of misinterpretation of results. Together with more reliable qualitative information concerning classification of passive and corroding areas, the galvanostatic pulse technique allows quantitative information to be obtained through calculation of corrosion current. If the area of polarized rebar is known, then corrosion current can be converted to a corrosion rate. It is possible in this way to estimate corrosion rate at the time of measurement in the case of general corrosion, but not in the case of local pitting corrosion. Galvanostatic pulse technique measurements take considerably longer to execute than half-cell potential measurements and require an experienced person to perform the technique.

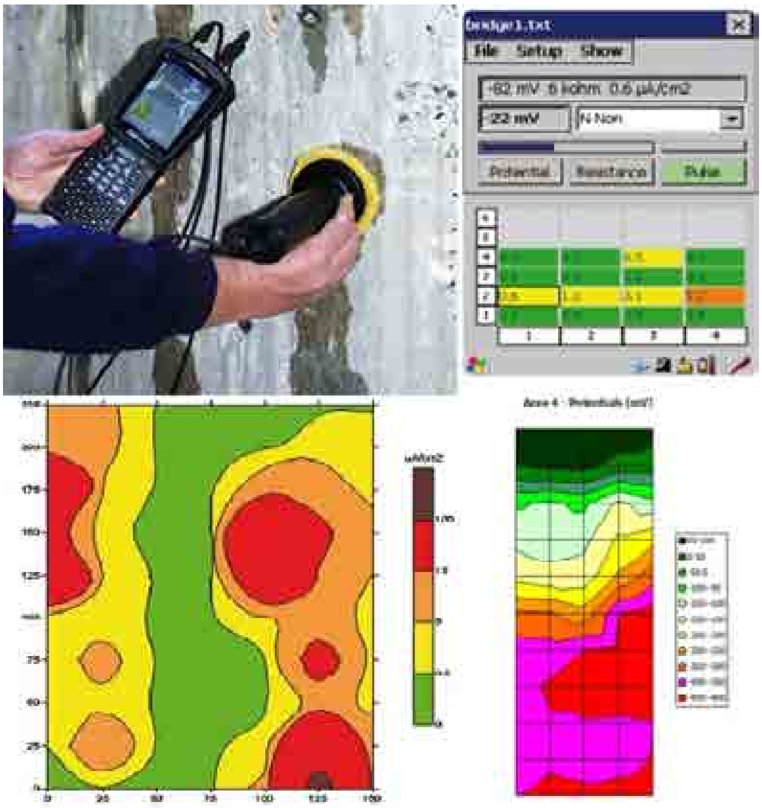


FIG. 109. Galvanostatic pulse technique: CorroMap tool for measuring half-cell potential, corrosion rate and resistance mapping (courtesy of Force Technology).

7.2.2.6. Tafel extrapolation

Tafel extrapolation is an electrochemical method for calculating corrosion rate through use of current intensity (I_{corr}) and Tafel slopes. The technique is based on application of either steady fixed levels of current followed by monitoring of potential (galvanostatic), or application of specific potential followed by monitoring of current (potentiostatic) [425]. This method differs from the linear polarization technique in that the change in potential is kept to less than ±25 mV for the linear polarization technique, while the change in potential can go up to ±250 mV for Tafel extrapolation. Tafel slope values for anodic and cathodic curves in the Tafel graph (see Fig. 110) are used to obtain corrosion current, which is then used to calculate corrosion rate. The accuracy of the Tafel extrapolation technique is equal to, or greater than, conventional weight loss methods. With this technique, very low corrosion rates can be measured, monitoring can be continuous and corrosion rate can be measured directly [425].

7.2.2.7. Electrochemical impedance spectroscopy

Alternating current (AC) electrochemical impedance spectroscopy is a technique for characterizing the frequency dependent electrical behaviour of cement based materials and interfaces. It has been investigated as a tool for quantifying corrosion of steel rebar embedded in concrete [24, 441] and chloride diffusivity in concrete [442]. The technique enables information to be obtained about mechanisms occurring within the system by applying a perturbation to reinforcing steel and measuring current flow and phase shift in the resulting current [443]. Typically, AC of about 10–20 mV is applied to the rebar and the resultant current and phase angle is measured for various frequencies. Response to an AC input is a complex impedance that has both real (resistive) and imaginary (capacitive or inductive) components [425]. By studying variations of impedance with frequency, an equivalent electrical circuit can be determined that provides the same response as the corrosion system studied [428]. AC electrochemical impedance spectroscopy can provide more information than linear polarization resistance methods, can estimate a steady state corrosion rate from the impedance spectrum, and can be used to study

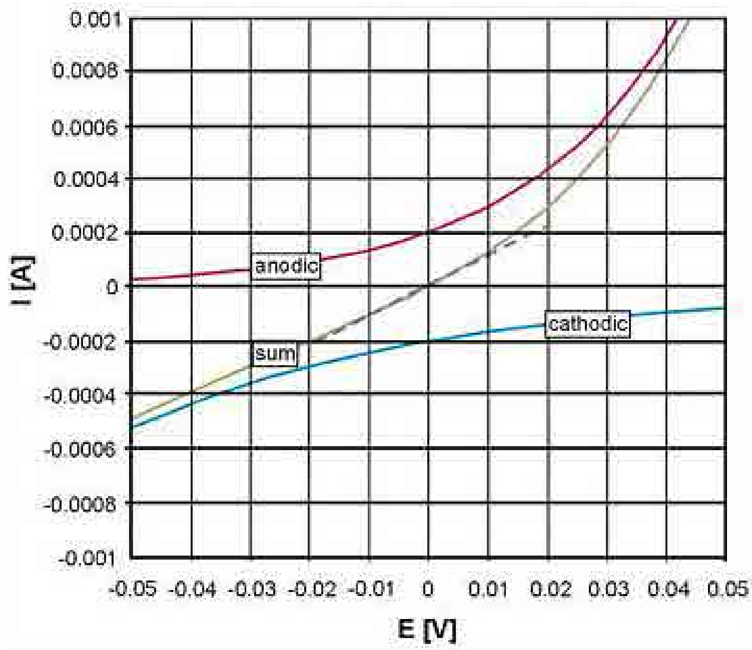


FIG. 110. Example Tafel graph of current intensity (I[A]) versus voltage (E[V]) [429].

effects of corrosion inhibitors, coatings and pitting corrosion [444]. It can, however, be labour intensive and time consuming, requiring lengthy data acquisition times, a physical connection to the steel embedded in concrete, and is presently more suitable for laboratory studies. In calculation of penetration rates, an assumption that uniform corrosion is taking place has to be made [428].

Figure 111 presents an experimental set-up for AC impedance spectroscopy measurements of corrosion resistance of coated steel dowels in concrete and results from the application of AC impedance spectroscopy to determine chloride diffusivity.

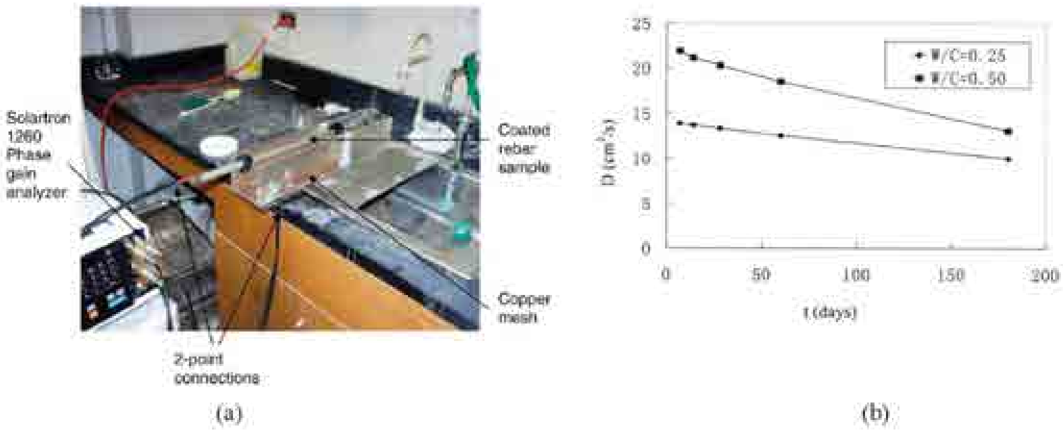


FIG. 111. AC electrochemical impedance spectroscopy: (a) experimental set-up; (b) change of chloride diffusion coefficients (D) with time (t) for mortars with different water to cement (W/C) ratios ([442] and [441], respectively).

7.2.3. Prestressing system

Primary potential ageing mechanisms associated with prestressing systems used in NPP CCBs are concrete shrinkage, creep, stress relaxation and corrosion of prestressing steel. Concrete shrinkage, creep and stress relaxation result in loss of prestressing force, and corrosion results in a reduction of load carrying capacity. Inspection methods associated with the detection of both of these manifestations are discussed in this section.

Corrosion protection is provided by filling ducts with wax, corrosion-inhibiting grease (unbonded or non-grouted) or Portland cement grout (bonded or grouted). Both grouted and non-grouted prestressing systems have been used in construction of CCBs.

Approaches for surveillance of grouted and non-grouted prestressing systems are different. In non-grouted systems, tendons are available for inspection, testing and retensioning. For grouted prestressing systems, cementitious grout provides excellent corrosion protection for the tendons when used properly. Being a one-time operation, the main emphasis is on material selection, qualification of the grouting process and quality control during construction. Requirements for grouts and grouting are defined in an NRC regulatory guide (see Ref. [301]). Guidelines for grout material and grouting operation are also provided in Refs [245] and [246], respectively. Although a grouted post-tensioning system cannot be inspected directly, indirect methods have been used to ensure continuous integrity over the plant service life.

Guidelines for developing ungrouted and grouted tendon surveillance programmes acceptable to the NRC, and for providing reasonable assurance (when properly implemented) that structural integrity of containment is being maintained, are provided in Regulatory Guides [303] and [259], respectively. Clarification with respect to determination of prestressing forces and prediction of prestressing force losses over structure service life is available [72]. Additional information on inspection of post-tensioning systems is available in Subsection IWL of Section XI of the ASME Boiler and Pressure Vessel Code [445]. A CSA standard [249] provides guidelines for ensuring integrity of both grouted and ungrouted post-tensioning systems of the concrete containment structure, including options for utilizing monitoring instrumentation and test beams.

7.2.3.1. Ungrouted post-tensioning systems

(a) Prestressing force determinations

Loss of prestressing force with time is not completely predictable and needs to be determined at regular intervals to ensure that the CCB retains adequate capacity to resist accident pressure and coincident design loads within acceptable margins. The CCB design establishes the minimum prestressing force necessary to maintain concrete in compression (full prestressing), with a reasonable margin, under postulated loads. Determination of the prestressing force level is performed routinely as part of the in-service inspection programme that is mandated by regulators in many Member States. Results obtained are compared to design calculations of prestressing force versus time, and if determined to be unacceptable, specific actions are required (e.g. increased inspection, retensioning or replacement).

Most CCBs with ungrouted post-tensioning systems use lift-off load determinations to indicate levels of prestressing force. Tendon end anchorage force is determined by measuring the jacking load required to lift the tendon anchor head clear of the shim stack or bearing plate. Limitations of this method are that it can only be performed on ungrouted tendons, and there may be some uncertainty associated with determination of the actual point of lift-off of the anchor head.

Some utilities monitor prestressing force using installed instrumentation.

(b) Mechanical tests on tendon materials

Associated with some ungrouted tendon in-service inspection programmes is the requirement to remove representative samples of tendon materials to monitor for ageing effects, notably corrosion. After removal, sections of the wire or strand, depending on tendon type, are taken from each end and the mid-length of the selected tendons is cleaned and visually examined for evidence of corrosion and tensile tests conducted (e.g. tensile strength, yield strength and elongation). The primary limitation of this procedure is that the number of tendons examined represents a small percentage of the total population. A study by ORNL in the 1980s [446] looked at sample size criteria under the proposed ASME Section XI and Reg. Guide 1.35 (Rev. 3) and concluded the guidelines were adequate with respect to providing high confidence in the containment force level. However, for the required sample sizes, the probability of including at least one defective tendon in the sample was low, when several defective tendons existed in the population.

(c) Tests on corrosion protection media

To provide a corrosion protection medium for ungrouted tendons, tendons are coated or the space between the post-tensioning tendon and metal sheath is filled with specially formulated grease. As part of some tendon in-service inspection programmes, samples of grease are taken at both ends of the tendons selected for examination and are analysed for free water content, reserve alkalinity and presence of aggressive ions (i.e. chloride, sulphide and nitrate ions). Limitations of this procedure are that only a limited sample size is evaluated and the samples may not reflect conditions at tendon mid-length.

(d) Visual inspection

Visual inspections of critical areas (such as anchor regions of a post-tensioning system, joints, areas of high stress concentration as identified by analysis) are performed periodically as part of in-service inspection programmes for some CCSs. Visual inspections may include areas where leakage was detected during integrated leakage rate testing, areas where problems were found previously and those areas where repairs were performed.

The end anchorage system (e.g. end cap, exposed bearing plate surfaces and anchorheads) is visually examined for evidence of cracking, distortion, major corrosion and broken or protruding wires. Visual inspection also includes examination of concrete adjacent to the bearing plates for cracking or spalling that would be indicative of bearing failure. End anchorage areas need to be examined for leakage of protective grease from the prestressing tendon ducts.

The primary limitations of this procedure are related to limited sample sizes and the fact that only visible locations can be examined.

Prestressing system channel internals can be inspected via camera recording systems. One such system has been developed by the Nuclear Research Institute Řež (Czech Republic) for the WWER-1000 Temelin NPP containment envelope (see Fig. 112). The system is autonomous and can record various parameters on a standard secure digital memory card for future evaluation.

The system is designed to be towed by means of the front towing wire. A securing suspender in the rear allows the system to be pulled out in case the towing wire is torn or mechanical obstacles appear in the direction of motion.

Technical parameters

— Cable channel/duct inside diameter a minimum of 70 mm;
— Internal camera battery life of 2.5 h, and lighting battery life (6 high intensity LED diodes) of 3.5 h;
— Speed of movement not to exceed 25 mm/s;
— Objects reliably displayed starting from 15 cm;
— Significant mechanical resistance while preserving low weight and good controllability.

FIG. 112. Prestressing system inspection camera for Temelin NPP; front and rear views (courtesy of Nuclear Research Institute Řež).

7.2.3.2. Grouted post-tensioning systems

Some containment structures with grouted post-tensioning systems have several tendons left ungrouted to enable direct measurement of prestressing force.

Monitoring systems are used on some containment structures with grouted post-tensioning systems. These consist of instruments installed during construction inside the concrete, or later retrofitted to the existing containment structure. They enable measurement of structure response to pressure loads during partial pressure testing and leak rate testing and enable monitoring of concrete long term characteristics in order to estimate prestressing force losses.

The main parameters monitored are strain, temperature and deformation. Instrumentation for measurements includes vibrating wire strain gauges, fibre optic sensors, extensometers, thermocouples, pendulums and invar wires. Information related to various types of instrumentation devices, including their advantages and limitations, is provided in Section 7.3.2.

Heat treated steel is sensitive to stress induced corrosion, which may lead to localized, non-ductile failure of a tendon. The main functions of grout are to protect prestressing steel from corrosion and to bond tendons to the surrounding concrete. Due to the bond between tendon and grout, grouted tendons are less vulnerable to local damage than ungrouted ones. If a tendon breaks, part of the prestress remains transmitted to the concrete.

The possibility of tendon corrosion is low if the grout and concrete surrounding tendons are in good condition. Voids within tendon ducts may be due to blockages, inadequate grouting procedures, grouting material problems or poor construction. Inadequate grouting may not fully protect tendons against corrosion and can lead to reduced durability, possibly causing loss of the cross-section and tendon breakage. In grout with high water to cement ratio, bleeding may occur, causing water pockets, particularly in higher areas of the prestressing ducts. High operating and ambient temperatures tend to facilitate corrosion once it is active.

Techniques for detection of corrosion in post-tensioning tendons are available and have been successfully used on civil structures such as bridges. Corrosion detection methods, however, are not widely used in the nuclear industry. Large areas of the containment structure would have to be covered to perform a thorough investigation. If investigation is performed locally, detected corrosion would raise concerns for the rest of the structure. However, if no corrosion is detected, the results would only be applicable to the area tested. With instrumented monitoring, local capacity reductions would be detected by the sensors (assuming they are of sufficient accuracy and are located strategically in appropriate quantities). In this case, corrosion detection methods could be applied to further investigate an identified location. Corrosion detection can also be used if there is a concern over grouting quality.

Ultrasonic tomography is successfully used to detect voids in grout of post-tensioning systems. It is based on the ultrasonic echo method. The transducers applied to the concrete surface introduce low frequency pulses of shear waves. A 3-D tomographer system, with transducers sending and receiving the reflected pulse, is able to scan and detect the voids in the grout at a distance of up to about 0.5 m from the surface, depending on the amount of reinforcing steel (i.e. heavy reinforcement reduces visibility).

The Post-Tech corrosion evaluation method, used by Vector Corrosion Technologies, is based on the fact that the possibility of corrosion depends on relative humidity at the tendon location. This method is partially destructive, as small holes (about 16 mm in diameter) need to be drilled into the structure to access the tendon sheath. Dry gas is injected at a low pressure at an inlet and measurements of relative humidity and temperature are taken at the outlet port(s) to determine the possibility of corrosion. Other methods for corrosion detection of grouted post-tensioning systems are described elsewhere [447, 448].

Some utilities utilize test beams fabricated during containment structure construction for indirect monitoring of prestressing system performance. These beams use the same materials and are stored under similar conditions as those of the concrete containment structure. As part of the in-service inspection programme, to confirm grout effectiveness as a corrosion protective medium, some utilities destructively test some of these test beams periodically and then perform visual inspection of their tendons for signs of corrosion.

Some beams are fabricated with ungrouted tendons to enable lift-off measurements. When interpreting results, consider that major contributors to prestressing force loss, such as creep and shrinkage, greatly depend on the size of the concrete element and environmental conditions (i.e. temperature and humidity). Therefore, prestressing force loss determined by testing of beams might not directly represent prestressing force losses of the massive concrete containment structure.

In-service inspection programmes need to include visual inspections of critical areas similar to those described for ungrouted systems (Section 7.2.3.1(d)), with the exception that camera systems and inspections for grease leakage are not applicable.

7.2.4. Penetrations

There are various penetrations through the containment structure (e.g. electrical penetrations, mechanical penetrations and airlocks). Since these penetrations are part of the containment pressure boundary, in-service inspections are periodically performed. Methods that have been used to demonstrate the integrity of these penetrations (outside of integrated leakage rate testing, described in Section 7.3.1) include visual examination and local leakage tests.

7.2.4.1. Visual examination

Containment in-service inspections generally require periodic visual examinations of the following:

— Accessible metal surfaces;
— Seals, gaskets and moisture barriers;
— Dissimilar metal welds;
— Pressure-retaining bolting.

These inspections are intended to detect problems that could adversely affect the structural capacity of the containment and its leaktightness. Visual inspection covers:

— Detection of corrosion on exposed metal surfaces;
— Weld defects (e.g. cracks);
— Wear, damage, erosion, tear, surface cracking or other forms of degradation that could affect the leaktight integrity of seals and gaskets.

The primary limitation of this method is that inaccessible regions cannot be examined.

7.2.4.2. Local leakage tests

Local leakage tests are conducted to detect leaks and to measure leakage rates across penetrations with flexible metal seals, bellows expansion joints, airlock door seals, doors and penetrations with resilient seals or gaskets and other components.

Three primary methods are used to perform leakage tests:

(a) Differential pressure measurement;
(b) Soap bubble testing;
(c) Gas detector.

The differential pressure measurement tests generally involve pressurization of the penetration being examined to a predefined magnitude using air or nitrogen and monitoring the decay in pressure with time. Soap bubble testing is used to identify locations of leakage by pressurizing the component and applying a thin film of liquid soap to exposed surfaces. Bubbles will develop at locations where leakage is occurring. The gas detector method uses a tracer gas that normally is not part of the surrounding air (e.g. halogen) to slightly overpressurize the component (e.g. bellow or similar). Any leakage is identified with a tracer detector. Limitations of these methods are that detection of soap bubble formation may be difficult in certain areas because of geometry or accessibility constraints and the lower limit on detectability of the tracer gas is dependent on the natural gas content of the surrounding air (e.g. helium: 5 ppm).

7.2.5. Liners

Both non-metallic and metallic liner systems have been used in CCBs. Containment leaktightness is verified periodically through leakage rate testing. Visual examinations of exposed liner surfaces are conducted periodically. Supplemental testing (e.g. ultrasonic, magnetic particle, liquid penetrant or electromagnetic) can be performed as part of this procedure, or if abnormal conditions are identified.

7.2.5.1. Integrated leakage rate test

The integrity of both metallic and non-metallic liners used to form the pressure boundary of many CCBs is evaluated by pressurizing the containment with air to a pre-established level. Information on this method is provided in Section 7.3.1.

7.2.5.2. Visual examination

General visual examination of accessible liner surfaces can uncover evidence of possible degradation that could affect containment integrity or cause unacceptable leakage. Corrosion of uncoated areas of metallic liner, due to excessive moisture or hostile inside containment environments, may also be revealed so that a maintenance programme can be implemented prior to significant loss of liner section. Examination of dissimilar metal welds will identify material defects in the weld and base metal (adjacent to weld edges). However, without removal of material, inaccessible areas that may be vulnerable to corrosion, such as liner portions embedded in concrete, cannot be inspected.

Visual examination of non-metallic liners and coatings is effective in identifying potential problem areas (e.g. blistering, peeling, delaminations and voids). However, testing is required in order to assess adhesion and determine coating thickness (see Section 7.2.5.9). Flexible fibre optic borescopes, charge coupled device cameras and computer based image processing software can assist in examining corners, bent surfaces and otherwise inaccessible surfaces [336].

7.2.5.3. Ultrasonic test

Ultrasonic testing is a non-destructive method in which beams of high-frequency sound (0.1–25 MHz) are introduced into materials for the detection of surface and subsurface flaws. The sound waves travel through the material examined with some attendant loss of energy (attenuation) and are reflected at interfaces. The reflected beam is displayed and then analysed to define the presence and location of flaws or discontinuities. Ultrasonic testing has particular application to metallic liner materials in that it can measure the thickness and extent of corrosion. The limitations of this method are as follows:

— At least one surface has to be accessible;
— Experienced personnel are required;
— Discontinuities just below the surface may not be detectable;
— Parts that are rough or thin are hard to inspect.

7.2.5.4. Magnetic particle test

Magnetic particle testing (see Fig. 113) is a method for locating surface and subsurface discontinuities in ferromagnetic materials. The part to be tested is magnetized by means of an electromagnet. Magnetic discontinuities that lie in a direction generally transverse to the direction of the magnetic field and, therefore, the presence of the discontinuity is detected by spraying ferromagnetic particles onto the surface. Some of the particles gather and are held by the leakage field. This magnetically held collection of particles forms an outline of the discontinuity and generally indicates its location, size, shape and extent. The limitations of this method are as follows:

— The method can only be used on ferromagnetic materials;
— The surfaces to be examined must be accessible;

— Operator experience and skill are required for best interpretation of results;
— Large currents may be needed for very large parts;
— Extraneous magnetic fields (e.g. from power cables) influence the test results.

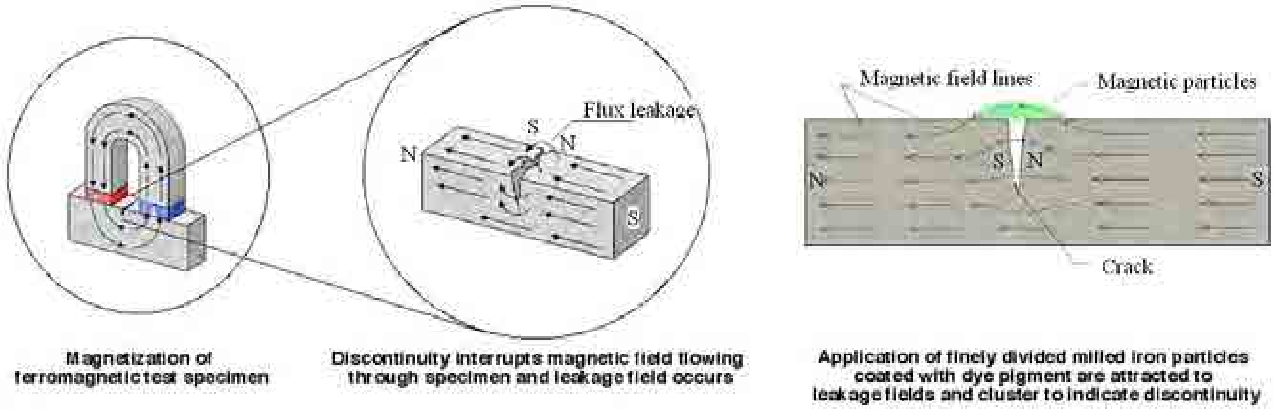


FIG. 113. Magnetic particle testing [449].

7.2.5.5. Liquid penetrant test

Liquid penetrant testing (see Fig. 114) can be used to reveal discontinuities in solid and non-porous materials. A wide spectrum of flaw sizes can be found regardless of test article configuration or flaw orientation. Liquid penetrants seep into minute surface openings on test specimen surfaces due to capillary action. After allowing time for penetration, excess penetrant is removed and a developer is applied to enhance the visibility of flaws. The limitations of this technique are that it can only detect surface flaws and results are affected by surface roughness and porosity.

FIG. 114. Flaw revealed by liquid penetrant testing [450].

7.2.5.6. Electromagnetic or eddy current testing

Electromagnetic testing, especially eddy current, is commonly used to inspect objects throughout their life cycle. Eddy current inspection methods are based on electromagnetic induction and can be applied to electrically conductive materials for detection of cracks, porosity, inclusions and to measure the thickness of non-conductive coatings on a conductive metal. Other applications include material thickness measurements, alloy sorting and monitoring of metallurgical conditions, such as hardness and heat treatment [449].

7.2.5.7. Radiography

Radiography is discussed in Section 7.2.1.1(f) on concrete inspections, and is additionally applicable to metallic liners where access to both sides of the liner is possible.

7.2.5.8. Infrared and thermal testing

Infrared and thermal testing was discussed in Section 7.2.1.1(g) on concrete inspections, and is additionally applicable to metallic liners.

7.2.5.9. Non-metallic liner and coating tests

The integrity of corrosion inhibiting coatings applied to metallic liners can be evaluated visually, with supplemental testing of suspect areas to measure dry film coating thickness and bond, and to determine if uncoated areas exist.

Liner coating thickness can be determined using the following:

— Magnetic pull-off gauges, which use a calibrated spring to determine the force required to pull a permanent magnet from a ferrous base coated with non-magnetic film.
— Magnetic flux gauges, which relate coating thickness to change in magnetic flux due to differences in distance between the instrument probe and the substrate.
— Tooke gauges, which use microscopic observations of a small V-groove cut into the coating film. Locations where Tooke gauges have been used may require recoating.

Evaluation of sealing and coating systems can be done through adhesion measurements by tape adhesion (e.g. ASTM D3359 [451]), direct tension (elcometer) or direct peel (e.g. ASTM C794 [452]).

A cross cut test is an example of a tape adhesion test described in ASTM D3359. In this test, six equally spaced cutting edges, with the distance between cutting edges dependent on coating thickness, cut through to the substrate. Tape is then applied and removed from the coating. Bond performance can be determined by comparing the amount of coat peeling from its backing.

Test methods for testing pull-off adhesion of non-metallic liners are described in CSA N287.2 [245].

7.2.6. Spent fuel pools

Spent fuel pools are massive reinforced concrete structures with typically limited access for surface inspection. Surveillance is used to monitor pool water levels, temperature, chemistry and presence and amount of any leakage. Inspections of pool liners can be done visually, for example using remotely operated vehicles (see Section 7.2.1.1(a)). Accessibility is limited due to radiation hazards and obstructions such as spent fuel, fuel racks and equipment.

Where external surfaces of SFPs are in contact with soil, indirect methods of monitoring environmental parameters (i.e. evaluation of air, soil, or groundwater conditions) can be used as described in Section 7.2.12 to assess degradation potential.

7.2.7. Cooling towers

Cooling towers in NPPs are typically not safety related. However, due to the investment involved and the role of cooling towers in electricity production, they are frequently inspected with procedures similar to those used for safety related structures. These involve visual inspection of concrete surfaces, a thorough check of tower shape (geometry checks and laser scanning) [453] and related preventive maintenance activities.

Typical specific inspection methodologies include:

— Checking for structure tilt using topography equipment;
— Detailed visual inspections of concrete surfaces to map the shape and size of cracks and other damage;

— 3-D laser scanning.

Section 7.2.1.1(a) and 7.3.4 provide further details of methods used.

Increasingly regular assessments are being augmented by embedded monitoring systems to enhance diagnostic capabilities. For example, vibration, temperature and clogging of cooling tower fills are being monitored with wireless sensors in numerous jurisdictions.

(a) *French inspections*

Electricité de France (EDF) has a long history of cooling tower assessment and maintenance. Based on experience, the tower's concrete shell is considered a sensitive component under EDF's Lifetime Project (see Appendix III.4). An assessment of cooling tower failure margins was conducted through analysis of ageing factors, evaluation of safety and durability factors, and improvements in inspection. Ageing factors were analysed with respect to:

— Outside surface (rates of concrete carbonation and aggressive ion penetration);
— Internal surface (attack by algae and condensate);
— Concrete structure (structure of cement hydrates and moisture and temperature gradients).

Safety and durability factor evaluations included site surveys (e.g. soil characteristics, ultimate resistance and buckling). Surveillances are accomplished through annual topographic surveys (formerly photogrammetry, now 3-D laser scanning).

Since the early 2010s, EDF has set-up a sample collection programme by drilling some parts of cooling towers in order to draw a map of carbonation progress and thus corrosion risk. This mapping will be taken into account for probabilistic studies, which will incorporate finite element models to help ensure cooling tower functionality and prevent collapse.

(b) *Belgian inspections*

In Belgium, the AMP objective for cooling towers is early detection of abnormal changes, so as to limit required repairs. Belgian cooling tower inspections are conducted in collaboration with EDF and include topographic examinations and visual inspections, among others.

Topographic inspections are conducted to monitor foundation displacement or shell deformation. Reference benchmark rings are glued to the top of the foundation system at five shell levels. A change in elevation of each of these benchmarks shows differential settlement, and allows for estimations of structural effects on the shell. The measurement of horizontal movement of each benchmark is performed by planimetry and the deformed shape of each level is drawn and analysed. Due to the shape and height to thickness ratio of the shell, its global stability and dynamic behaviour are very sensitive to top and bottom ring rigidity. Deformation of these rings from circular can induce excess structure flexibility.

Visual inspections are primarily associated with crack mapping, though other defects in shell outer faces are also mapped. To emphasize defects during mapping, the tower is kept in operation and the inner surface is not accessible. Photographs covering the entire shell surface are taken from ground level. Computer aided interpretation of the photographs permits mapping of cracks and defects and their evolution and the development of crack distribution, length and orientation statistics. Cataloguing of photographs shows shell changes with time. Additional inspections include measurements to determine rebar corrosion and depth of concrete carbonation. Laboratory tests are performed on samples removed from the towers. Results of inspections (e.g. deformation of the shell and degradation in material properties) are used to perform analyses to demonstrate that required safety factors are being maintained.

7.2.8. Water intake structures

Water intake structures may be safety or non-safety related and thus be subject to different inspection requirements. Guides exist for condition based maintenance of general purpose structures [454]. Routine monitoring involves visual inspections of above water level structures and video recording of underwater structures. Inspection frequency varies with the type of plant, the water environment (e.g. aggressiveness of water and presence of bio-fouling agents that can impact plant operation) and observed degradation.

7.2.9. Concrete pipe

Assessment of the condition or health of pipeline systems is very difficult given that they are typically buried and are, therefore, not readily accessible for visual inspection. Inspections are complicated by a typical lack of redundancy built into many systems, making system isolation and internal access difficult.

Three basic types of sensing technologies for concrete pipelines include:

(a) Internal sensing (using technology deployed inside the pipe, such as remote video cameras or ultrasonic transducers);
(b) Fibre optic sensing (e.g. fibre Bragg gratings);
(c) Remote sensing (e.g. infrared thermography systems, GPR).

EPRI has produced a guide for NDT techniques that can be applied to underground piping, including concrete pipe [455].

Specific inspection technologies typically applied to concrete pipe include:

- *Internal visual inspection and sounding:* For highly distressed pipe; can suffer from a high number of false positive readings; see Fig. 115 for an example.
- *Electromagnetic inspection:* Provides locations, distribution and number of wire breaks; broken prestressing wires show as signal distortion in electromagnetic field.
- *Acoustic monitoring:* Records noise due to wire breaks, slippage and other events as they occur.
- *Acoustic leak detection:* Provides size of leak and location; may indicate joint problems or imminent pipe rupture.
- *External inspection:* Wire continuity testing; mortar is removed at crown of pipe and wire resistivity measurements are plotted over the pipe length. This method can verify results of electromagnetic inspection.
- *Galvanostatic pulse measurement:* Detects location of corrosion of rebar; see Section 7.2.2.5.
- *Material testing, petrographic analysis and soil corrosiveness tests*.
- *GPR:* Pipe, structure, void and leak location.

FIG. 115. Pipeline engineer conducting a visual and sounding inspection (courtesy of Pure Technologies Inc.).

7.2.10. Masonry structures

Traditional evaluation methods for condition and properties of masonry structures have been, excluding visual inspection, largely destructive in nature, as they involve testing of specimens removed from a structure [456]. The application of destructive methods to NPP masonry structures, however, is limited because specimen removal may be structurally damaging. The evaluation of masonry structures uses NDT methods to complement visual inspections to determine flaw location and for assessing material properties. The most common NDT techniques for masonry structures are listed in Table 21 (on p. 166). In addition to the techniques provided, relatively recent developments include the use of scanning laser Doppler vibrometry, GPR and acoustic tomography.

The Schmidt (or rebound) hammer method (discussed in Section 7.2.1.1.) provides a measure of relative material surface hardness. Its primary application is in assessment of material uniformity over large areas. It is generally not recommended that Schmidt hammer be used as a direct indicator of compressive strength, but more for indicating relative changes in compressive strength between locations.

Flatjack methods provide information on stresses in a masonry structure and provide a direct measurement of material and structural properties (i.e. deformation, shear strength and compressive stress) [457– 459]. A flatjack is a flexible steel envelope, thin enough to fit within a masonry mortar unit that is hydraulically pressurized to apply stress to the surrounding masonry. Flatjack tests use either single or two flatjacks. The evaluation of in situ compressive stress is accomplished by removal of a portion of the mortar bed joint, measuring the magnitude of the resulting deformation, inserting a flatjack into the joint and restoring the original state of stress by pressurization of the flatjack until the original position of the measuring points is restored.

The in situ deformability test is conducted by inserting two parallel flatjacks separated by several courses of masonry, pressurizing the jacks equally and measuring masonry deformation between the two jacks (see Fig. 116). This method provides a measure of masonry compressive modulus.

The in-place shear test measures joint shear resistance between masonry units or masonry joints. A single masonry unit is removed, as well as a head joint on opposite sides of the test unit. The test unit is then displaced horizontally relative to the surrounding masonry using a hydraulic jack inserted into the location where the masonry unit was removed. The horizontal force required to cause first movement of the test unit is determined. A modified version of this technique has been developed to provide more reliable results. In this approach, vertical stress on the wall at the test unit is determined using the single flatjack test procedure. Normal stress during the in situ shear test is then controlled using flatjacks above and below the test unit. Figure 117 presents a schematic of the set-up for the modified in situ shear test.

FIG. 116. In situ deformability test using two flatjacks (courtesy of Feld, Kaminetzky & Cohen, P.C.).

The ultrasonic pulse velocity method utilizes acoustic transducers (~50 kHz) to pass high frequency stress waves through masonry. Both direct and indirect tests can be conducted (see Fig. 118). Direct tests, or through wall tests, are useful in establishing material uniformity and identifying locations where voids are present

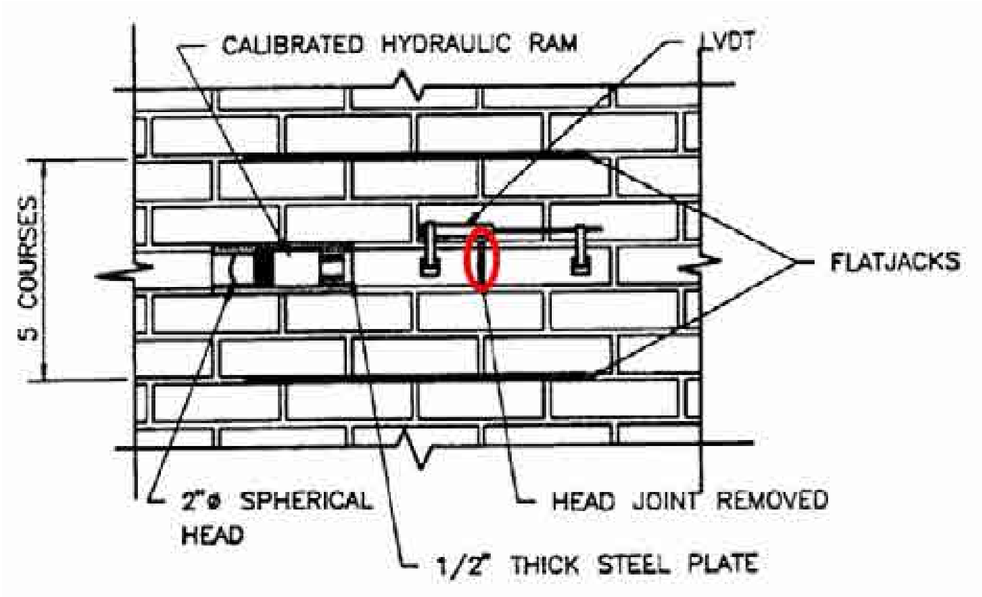


FIG. 117. Modified in situ shear test of masonry [456] (LVDT is linear variable displacement transformer).

(e.g. locations exhibiting longer than average arrival times). Indirect tests are used to determine average velocity through an accessible masonry unit to detect voids or lower quality material (e.g. reduction in pulse velocity). Heterogeneity of masonry construction limits applicability of ultrasonic pulse velocity methods due to signal attenuation (see Section 7.2.1.1.).

Magnetic (or electromagnetic) methods provide a fairly rapid means to locate vertical, horizontal and joint steel reinforcement, metal ties and metal connectors. This technique was discussed in Section 7.2.1.1.

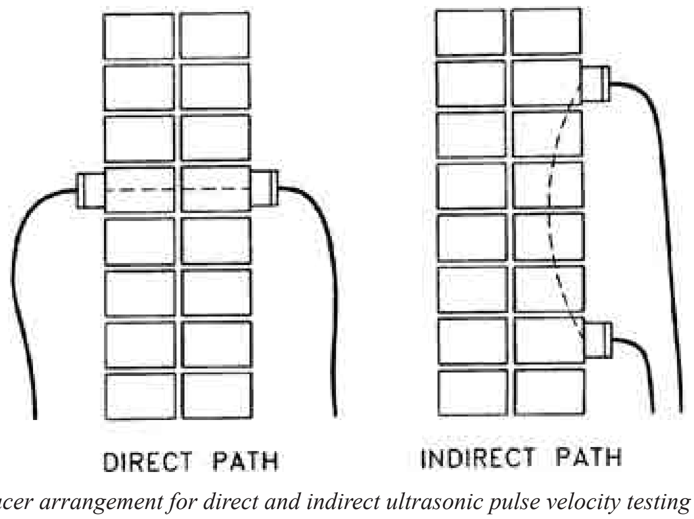
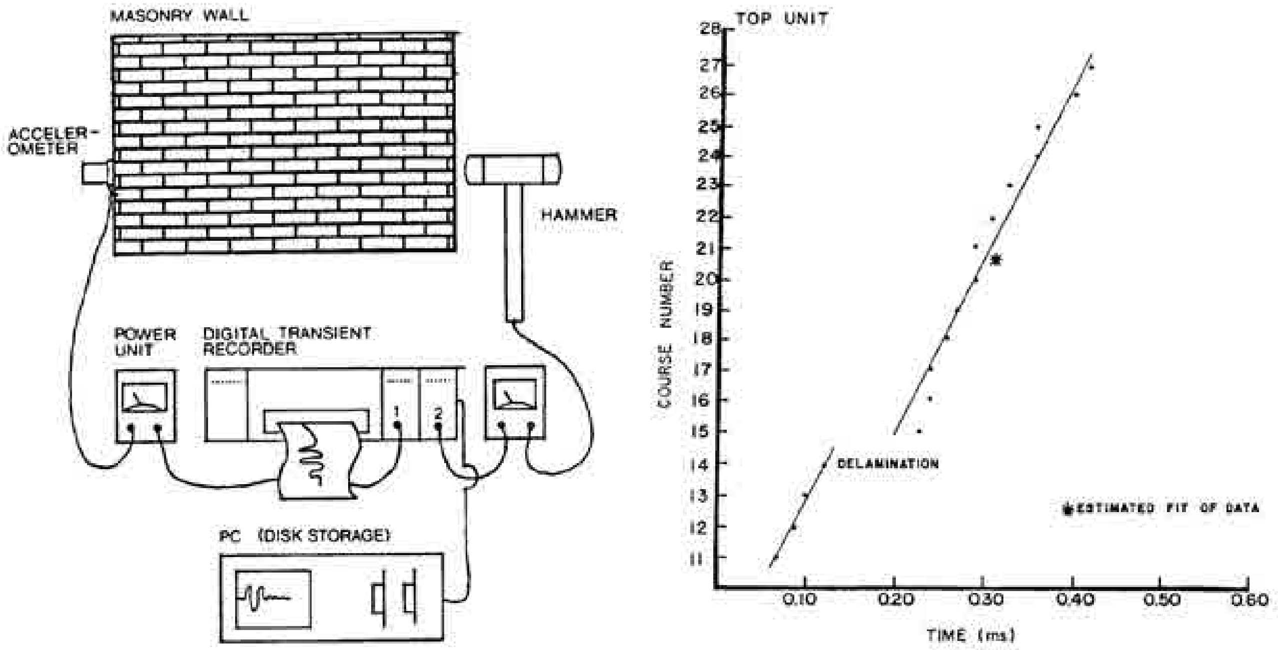


FIG. 118. Transducer arrangement for direct and indirect ultrasonic pulse velocity testing of masonry [456].

The mechanical pulse velocity test utilizes an object (e.g. a hammer) to input a stress wave into the masonry wall, and then subsequent vibrations are monitored using an accelerometer. A digital transient recorder is used to record both the hammer input signal and accelerometer output signals. Stress waves input in this manner are low frequency with high amplitude, and of longer wavelength than those input by the ultrasonic pulse velocity method. The quantity of interest is the pulse arrival time, which, when path length is known, provides the pulse velocity. Pulse velocity can be correlated to material properties. The technique can also be used to locate material flaws and discontinuities. Figure 119 presents a schematic of the method and an example application.



(a) (b)

FIG. 119. (a) Schematic of mechanical pulse velocity testing apparatus; (b) detection of masonry wall delamination [456].

TABLE 21. MASONRY STRUCTURE NON-DESTRUCTIVE EVALUATION TESTS [456]

NDT Technique	Advantages	Disadvantages
Rebound hammer	Simple to use and inexpensive Establishes uniformity of properties	Point evaluation only No direct relation to strength or deformation properties Unreliable for flaw detection or inaccessible areas
Single flatjack in situ strength	Establishes state of compressive stress in situ Easy to use and inexpensive equipment ASTM standards available	Time consuming to prepare test Requires removal of mortar Repair required after test
Double flatjack in situ deformation	Establishes deformation properties in situ Easy to use and inexpensive equipment ASTM standards available	Time consuming to prepare test Requires removal of mortar Repair required after test
In-place shear strength	Establishes joint shear strength in situ Easy to use and inexpensive equipment ASTM standards available	Time consuming to prepare test Requires removal of masonry unit and head joint Restricted to masonry with low cement content mortar Requires unit and mortar replacement after test State of compressive stress on test unit must be estimated Contribution of collar joint unknown
Two flatjack modified in-place shear strength	Can establish joint shear strength in situ Permits control of compressive stress of test unit Determines Coulomb failure surface for material	Time consuming to prepare test Requires removal and replacement of two masonry units and two mortar joints Restricted to masonry with low cement content mortar Large amount of equipment is required Contribution of collar joint unknown

TABLE 21. MASONRY STRUCTURE NON-DESTRUCTIVE EVALUATION TESTS [456] (cont.)

NDT Technique	Advantages	Disadvantages
Ultrasonic pulse velocity	Simple to use and establishes uniformity of properties Can detect flaws, cracks and voids Equipment readily available and moderately priced Trace of stress wave recordable for analysis	Requires access to both sides of wall for direct measurements Signal attenuation in older or soft masonry restricts distance between transducers for indirect and direct application Grinding of rough surface required Coupling material needed between masonry material and transducers No direct correlation with material properties
Mechanical pulse velocity	Simple to use with no damage and establishes uniformity of properties Can detect flaws, cracks and voids Equipment readily available and moderately priced Trace of stress wave recordable Capable of testing over long distances	Several equipment items are required Requires separate instrument to record wave arrival time with wave analysis complicated No direct correlation between results and material properties
Magnetic methods	Equipment portable and inexpensive Large areas of masonry easily evaluated Locates metal ties and connectors and accurately maps location and orientation of reinforcing steel	Readings can be ambiguous requiring operator interpretation or supplemental destructive tests Possible misidentification of metal conduit and reinforcing steel, among others Accuracy in determination of bar size and depth questionable

Scanning laser Doppler vibrometry was developed for use in diagnosis of civil and historical buildings [460]. The technique measures point-by-point surface velocities using interferometric techniques with galvanometric driven mirrors steering laser beams. Structural excitation is radiated by acoustic waves from high power loudspeaker systems. Surface vibrations induce a Doppler frequency shift on the impinging laser beam and this shift is linearly related to the velocity component in the direction of the laser beam. Most diffused scanning laser Doppler vibrometers have a maximum velocity range of 10 m/s, with a frequency upper limit of 200 kHz, a resolution of about 1 μm/s, and a base accuracy on the order of 1–3%. Laser power is less than 1 mW, so no special safety measures are required. Working distances of tens of metres are possible with a spatial resolution of 1 mm. Figure 120 illustrates the application of scanning laser Doppler vibrometry to the inspection of a block wall of 1.8 m × 1.5 m × 0.25 m. The wall was damaged by hitting it with a hammer to create an irregular groove with a width of some centimetres and a depth of about 2 cm. Figure 120 presents results for the undamaged wall and, as seen in Fig. 120, the vibration mode shifts as a result of the induced damage.

GPR (see Section 7.2.1.1(g)) has been used for NDT of masonry structures, primarily historical buildings [461]. The majority of these applications have been conducted by using 2-D or 3-D profiles with the system in echo configuration (i.e. transmitter and receiver on same side of structure) and using frequencies normally in the 500–1 GHz range [462, 463]. These investigations demonstrated that pertinent information may be obtained about the physical properties of brick and stone masonry walls (e.g. thickness, number of layers and position of detachments, structural discontinuities or voids and metal inclusions) [464]. The GPR technique, with the capability of transmitting and receiving radio frequency waves linearly polarized along the direction of the dipole (HH (horizontal) or VV (vertical) directions), has been used for non-invasive retrieval of information on the internal structure of masonry [465]. Figure 121(a) presents a test specimen, which consisted of three sections:

— A section 12 cm thick with complete bricks and lime;
— A section 12 cm thick with full bricks, but without lime, that contains a cavity 50 cm × 20 cm × 12 cm;
— A concrete wall section 50 cm thick.

Post processed images (HH, VV and combined HH-VV) at a depth of about 15 cm are provided in Fig. 121. Both HH and VV results clearly indicate the cavity. Subtraction of the two focused phase maps (HH-VV) provides a cleaner image of cavity edges.

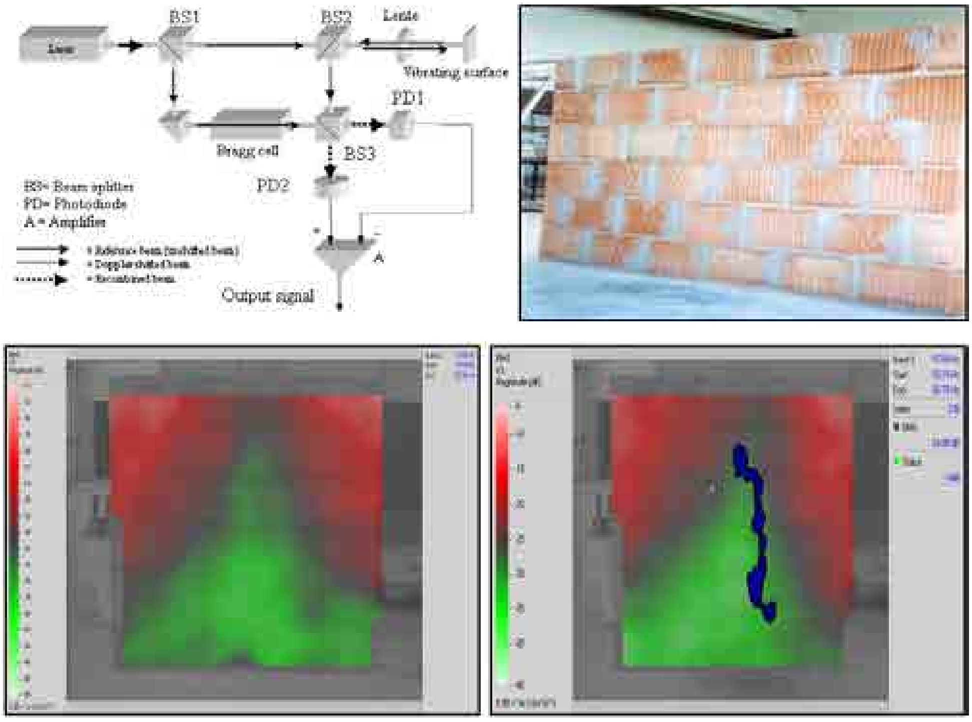
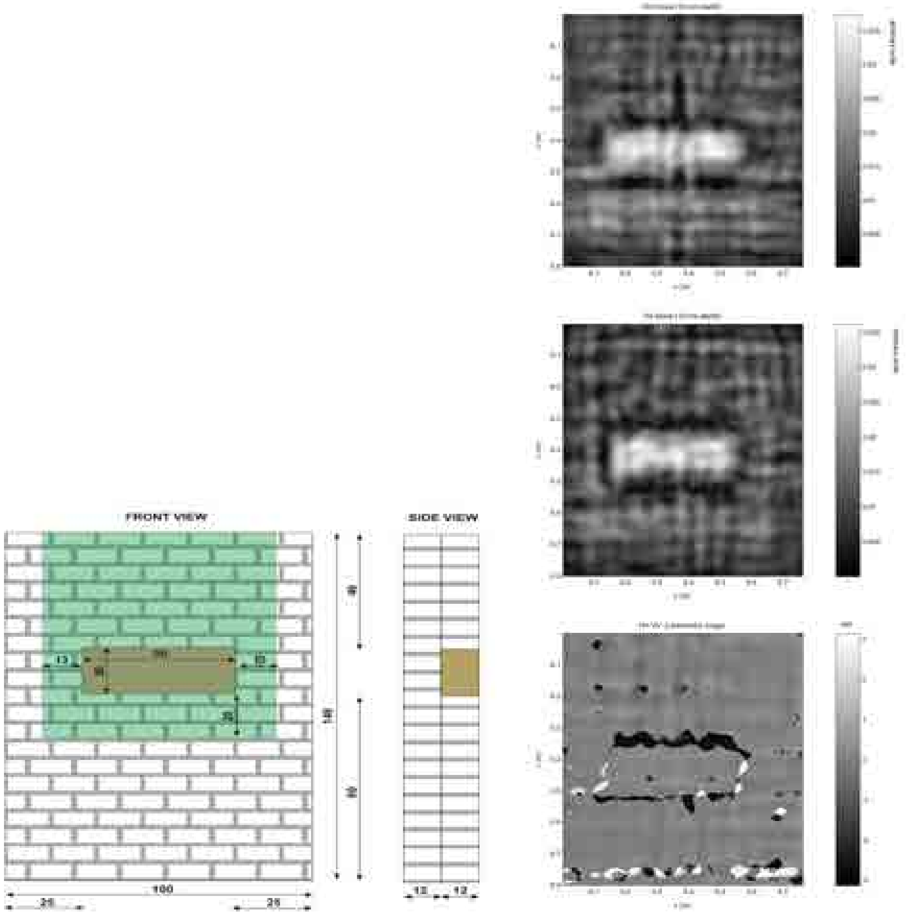


FIG. 120. Application of laser vibrometry to inspect a block wall [460].



(a) (b)

FIG. 121. Use of polarized GPR to investigate a masonry wall with an internal cavity: (a) masonry test facility with internal cavity; (b) GPR images at 15 cm depth [465].

In another application of GPR to masonry structures, a test specimen (see Fig. 122(a)) was constructed of solid brickwork except for the presence of air voids (6 cm × 11 cm × 12 cm, 6 cm × 11 cm × 24 cm and 12 cm × 11 cm × 24 cm) simulated by skipping bricks, part or all, at three different depths [464]. The test specimen was investigated using GPR systems having nominal frequencies of either 1.5 GHz or 1.7 GHz. Tomograms of results of radar measurements are shown in Fig. 122(b), with void locations marked by dashed lines. Results demonstrate that GPR can identify and locate voids 12 cm × 11 cm × 24 cm, or larger, in masonry structures. GPR has also been used to detect rising moisture levels in brick work as illustrated in Fig. 123 [466]. In this application, signal delay, due to a higher dielectric constant resulting from moisture presence, provides a visualization of the qualitative effect of rising moisture and moisture content. In order to determine moisture content quantitatively, calibration of dielectric constant and moisture content have to be performed [467]. The influence of dissolved salts also needs to be considered.

(a)

(b)

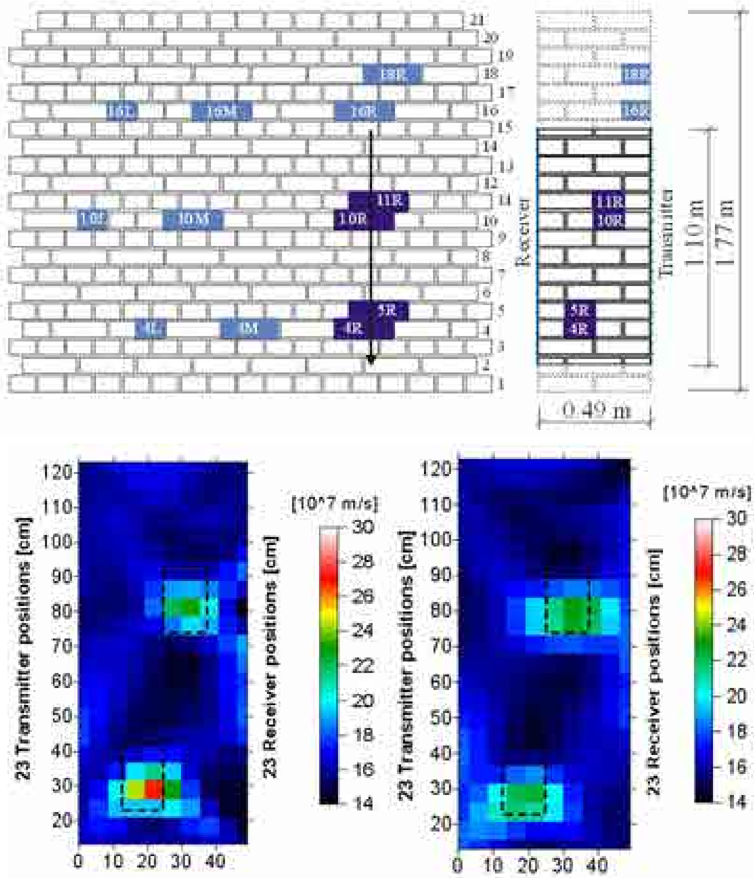


FIG. 122. GPR testing of masonry wall with simulated air voids: (a) test masonary wall showing location of air voids; (b) detected air voids using GPR [464].

Tomographic imaging uses sonic or ultrasonic pulse velocity information taken through a section of a structure to develop a 2-D or 3-D reconstruction of velocity distribution. The reconstruction image is used to locate features concealed beneath the material's surface [468]. Acoustic methods (e.g. acoustic tomography) have been found to be suitable to detect structural inhomogeneities in brick and stone masonry [461]. Ultrasonic tomography was also applied to the masonry test specimen shown in Fig. 122(a). Ultrasonic transmission measurements were performed by using two different pairs of transducers emitting longitudinal waves with nominal frequencies of 85 kHz and 25 kHz. The transmitter was fixed on one side and the position of the receiver was changed to cover a distance of 110 cm in 5 cm increments (23 received positions). The transmitter was moved and the process repeated until the monitored section was traversed (23 transmitter locations).

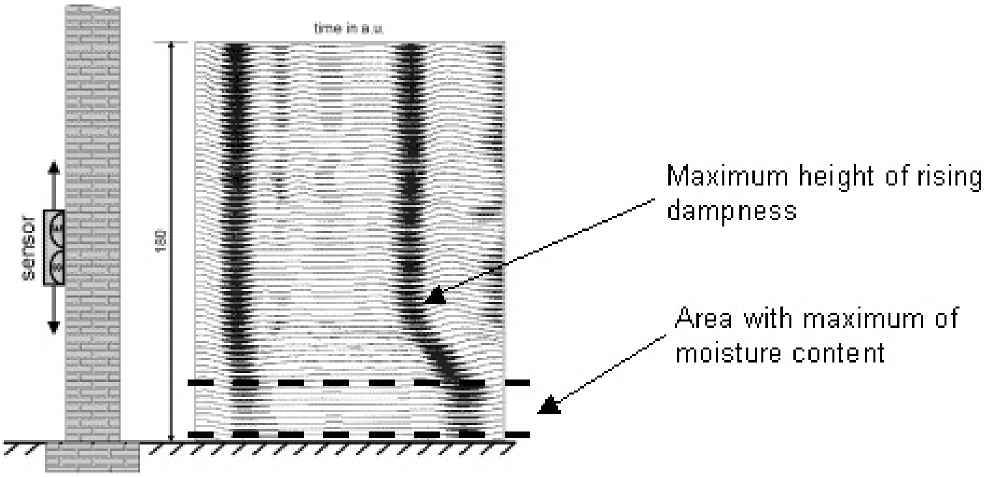


FIG. 123. Application of GPR to detect moisture level [466].

Figure 124 presents an example of a test set-up and theoretical travel paths for different receiver locations when a void is present, thus indicating an increase in wave travel time due to the existence of the void. The acoustic tomograms shown in Fig. 125 were obtained from the masonry test specimen containing simulated air voids. The tomograms show two significant zones of lower velocity that correspond to void positions. The results demonstrate the ability of acoustic tomography to identify and locate voids of 12 cm × 11 cm × 24 cm, or larger, in masonry structures.

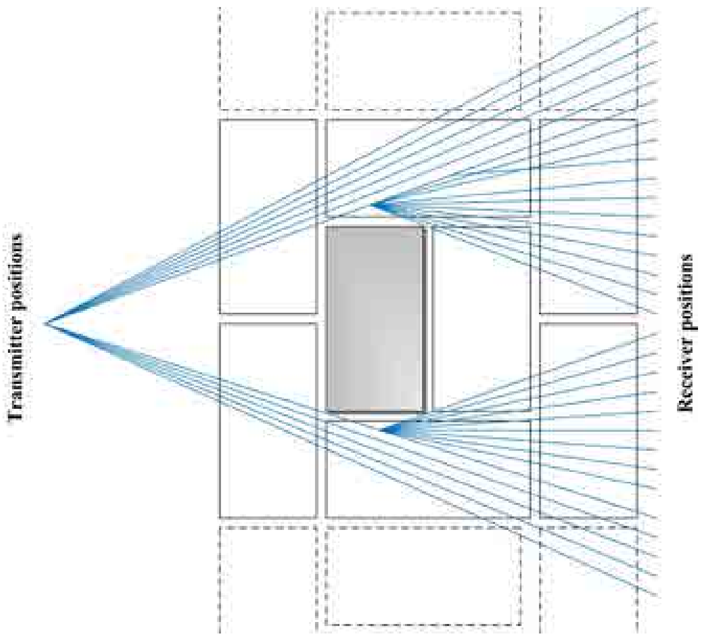


FIG. 124. Illustration of effect of void in masonry travel path of ultrasonic wave.

(a)

(b)

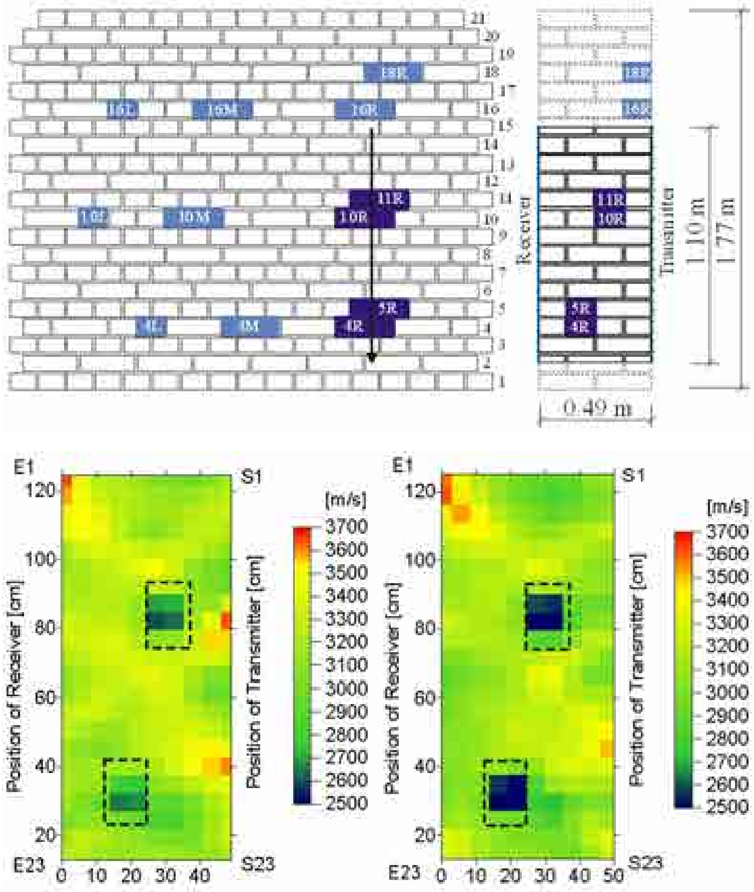


FIG. 125. Acoustic testing of masonry wall with simulated air voids: (a) masonry wall specimen; (b) detection of simulated air voids in masonry wall using acoustic tomography [464].

7.2.11. Structural anchorages

Visual inspection of structural anchorages typically consists of examining concrete adjacent to anchors for degradation, such as cracking or spalling. Other checks need to be made for corrosion of exposed embedded metal surfaces, corrosion stains around embedded metal, detached embedments or loose bolts and indications of degradation due to vibratory loads from piping and equipment. Guidance for evaluation of these conditions is available in Ref. [241].

A discussion of the inspection of anchorages associated with prestressing systems is in Section 7.2.3.

7.2.12. Foundations, piles and underground structures

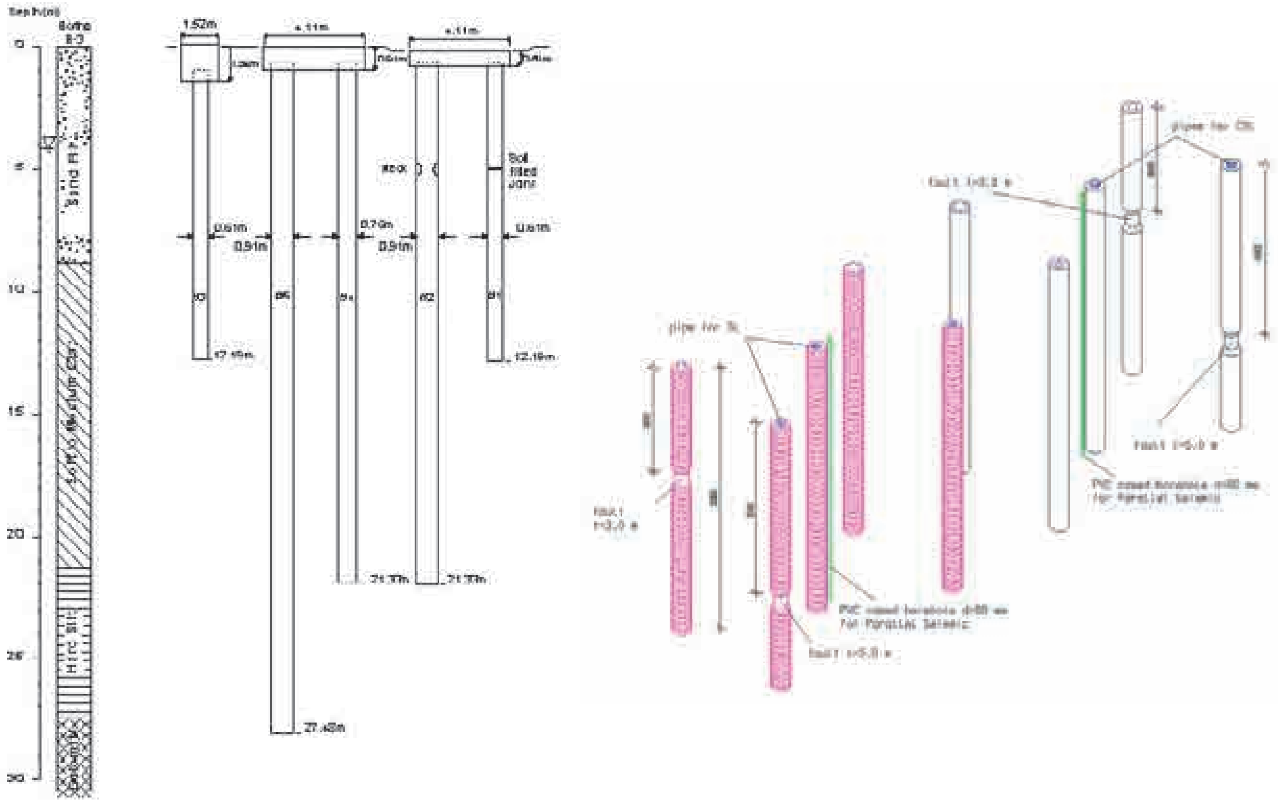
If upper surfaces of a concrete foundation (or pile) are available for inspection, a number of NDT techniques are available, including:

— Impulse echo;
— Sonic mobility;
— Cross-hole seismic logging or tomography;
— Parallel seismic testing;
— Single-hole sonic logging;
— GPR [469].

A discussion of the impulse response method was previously provided in Section 7.2.1.1. Advantages and disadvantages of these methods for evaluating structural integrity of deep foundations (i.e. driven piles, cast-in-place piles and drilled shafts) are documented in Ref. [470].

The applicability of NDT methods to evaluate deep foundations under inaccessible head conditions in a controlled environment has been investigated through a study at the National Geotechnical Experimental Station at Northwestern University [471]. Figure 126(a) presents a schematic of the site. In this study, five drilled shafts with lengths between 12.2 m and 27.4 m and diameters between 0.61 m and 0.914 m were constructed; some purposely with defects (a reduced cross-section in one shaft to represent a typical construction deficiency and a thin, soil filled joint in another to represent a performance related defect induced by excessive lateral loads). Reinforced concrete caps were cast on the piles. Piles were evaluated using sonic echo and impulse response testing under both accessible and inaccessible head conditions. Additional tests were performed using the parallel seismic method for inaccessible head conditions.

Another large scale, deep foundation site for optimization and validation of NDT methods has been built at Horstwalde, Germany [472]. This site consists of a foundation connected to bored piles, including both sound piles and piles with flaws or geometrical anomalies. The site is also being used in the European Community Fifth Framework Programme, Re-Use of Old Foundations on Urban Sites (RUFUS), for personnel training [473]. Accurate knowledge of exact geometry and structure of piles and the presence of embedded instrumentation (i.e. temperature, strain and vibration) allows for the evaluation and calibration of measurement devices. Figure 126(b) presents a schematic of the site at Horstwalde, Germany. The relative performance of NDT methods for deep foundations can be obtained from results being developed under the RUFUS project and from the two large scale test facilities.



(a) (b)

FIG. 126. Large scale deep foundation test facilities: (a) schematic of National Geotechnical Experimental Station site; (b) schematic of Horstwalde, Germany site ([471] and [474], respectively).

The most common foundation for both concrete and steel NPP containments is a basemat, which is a flat, thick slab supporting the containment, its interior structures and any shield building surrounding the containment [475]. As such, concrete NPP foundation elements are typically either partially or totally inaccessible for inspection. Under these conditions, foundation structures are only accessible for inspection after removal of adjacent soil, coatings, waterproof materials or portions of neighbouring components or structures. As a result, indirect methods related to environmental conditions are often used to indicate a foundation's degradation potential [318]. This is generally done through an evaluation of surrounding medium (e.g. air, soil, humidity, groundwater or cooling water). Methods employed are based on a chemical evaluation to assess the presence and concentration of potentially aggressive ions (e.g. sulphates or chlorides). In addition to an assessment of the aggressiveness of the surrounding environment, the US Code of Federal Regulations (CFR) requires a complete description of groundwater level and other hydrodynamic effects on the design bases of the basemat and SSCs important to safety [476].

Table 22 provides test methods and references related to the evaluation of air, soil or groundwater conditions adjacent to a concrete foundation element. Information is available on chemicals in the ground or in groundwater that are potentially harmful to concrete [96, 306]. Detailed guidelines on assessing and classifying aggressiveness to concrete of chemicals in the ground have been developed [477].

TABLE 22. EXAMPLES OF TEST METHODS AND REFERENCES FOR ENVIRONMENTAL ASSESSMENTS

Item measured	Candidate method(s) or reference(s)
Air	
Acidity	ASTM D1654, G50 and G92
Carbon dioxide content	CO_2 sensors
Humidity	ASTM D4230 and E337
Temperature	Temperature sensors
Soil	
Corrosivity/pH	ASTM G51; BS 1377-3, Section 9; BR 279
Sulphate ion content	ASTM D4542; BS 1377-3, Section 5; BR 279
Chloride ion content	ASTM D4542; BR 279; BS 1377-3, Section 7
Resistivity	ASTM G57
Moisture content	ASTM D2216 and D3017; DIN 18121-1, -2
Nitrate	BR 279
Permeability	ASTM D2434; DIN 18130-1; EN 1997-2, EN 7; BS 8004; BS 5930
Groundwater	
Water table elevation and sampling	ASTM D512, D1293 and D4448; observation wells; piezometers
Corrosivity/pH	ASTM D1067, D1293 and E70; BR 279; BS 1377-3, Section 9
Hydrostatic pressure	Standard sensors

TABLE 22. EXAMPLES OF TEST METHODS AND REFERENCES FOR ENVIRONMENTAL ASSESSMENTS (cont.)

Item measured	Candidate method(s) or reference(s)
Dissolved oxygen content	ASTM D888
Soluble sulphate	ASTM D516, D4327, D4130 and D4327; BR 279; BS 1377-3, Section 5; DIN 38405-5
Nitrate ion	ASTM 4327, BR 279
Chloride ion	ASTM D4458, D4327 and D512; BR 279; BS 1377-3, Section 7
Carbon dioxide content	EN 13577
Microorganisms and bacteria	ASTM D4412

Note: ASTM: Standard Test Methods or Practices, American Society for Testing and Materials, West Conshohocken, Pa.;
BS: British Standards, British Standards Institute, London, United Kingdom;
EN: European Committee for Standardization, Brussels, Belgium;
BR: Building Research Establishment report BR 279: [478];
DIN: German Standards, Deutsches Institut für Normung e.V., Berlin, Germany.

7.2.13. Waterstops, joint sealants and gaskets

Visual inspections of waterstops, joint sealants and gaskets are effective in identifying potential problem areas. Degradation of these components can also be detected through periodic leak testing. Durometer testing is used to indicate the condition of elastomeric materials. Durometers have a calibrated spring that forces an indentor point into the test specimen (see Fig. 127). An indicator scale provides a direct reading that can be related to the suitability of the elastomer material for its intended application.

FIG. 127. Durometer for testing elastomeric materials (courtesy of Instron).

7.3. MONITORING

7.3.1. Integrated leakage rate testing

Containment leak rate tests are performed periodically to ensure that leakage through containment, or systems and components penetrating containment, does not exceed specified allowable leakage rates. Test pressure and frequency of leak rate tests vary. The trend in some countries is to extend the period between leak rate tests where appropriate based on past performance, ageing management activities and assessed risks. Containment leak rate testing can also be used to ensure that containment structure integrity is maintained during its service life. IAEA publications, Refs [29, 479, 480], provide general guidance for performing containment leak rate tests. Further detail is being established under IGALL AMP304 [225] (see Section 4.9.3).

Concrete containment leakage rates are evaluated by pressurizing containment with air to a pre-established level (e.g. peak pressure associated with a design basis accident) and monitoring leakage as a function of time. Both full and partial pressure testing have been used. Pressure, temperature and vapour pressure sensors are used during testing to sample containment atmosphere. Changes in contained air mass define the leakage rate. A typical leak rate test sequence is shown in Fig. 128.

Tests often have a step whereby a systematic inspection of containment penetrations for leakage is performed, with repairs being made as required. Cracks can be checked for leakage by spraying a thin film of soap solution on cracked areas and visually monitoring the formation of bubbles during pressurization. The presence and rate of bubble formation indicate openness of cracks. Using air flow detection ‘smoke’ that is chemically produced in tubes, or using light plastic bags taped to cracks, can also assist in assessing crack or containment leakage during testing. Other methods, such as fogging of the RB (using smoke generators), use of tracer gasses, or personnel searches for unusual sounds, can be used for leak searches. Monitoring the response of the containment structure to the pressure load using strategically located instrumentation enables integrity evaluation of the post-tensioning system (see Section 7.3.2).

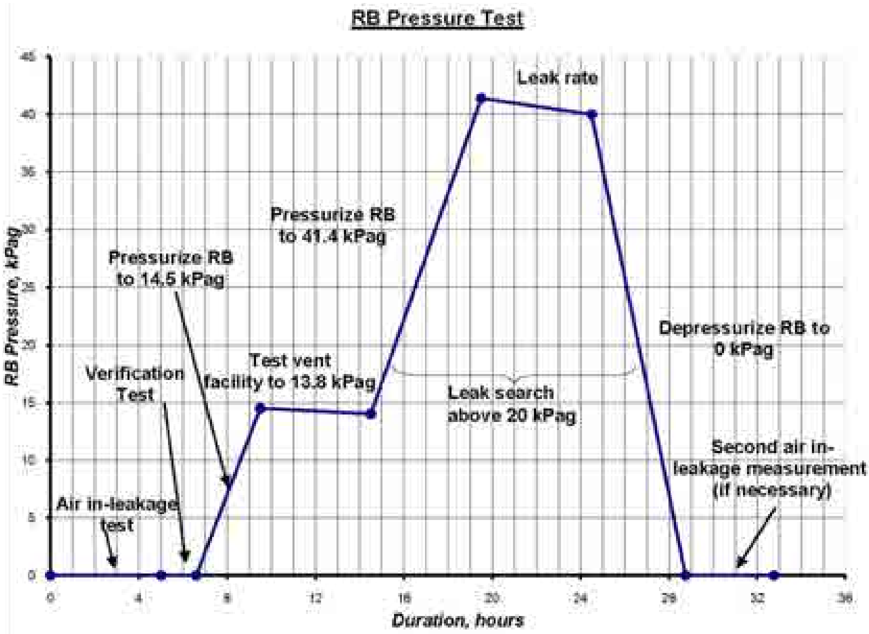


FIG. 128. Typical RB leak rate test sequence (courtesy of Ontario Power Generation).

A typical step in an ILRT is to confirm that leakage rates measured at high pressure are a laminar extrapolation of those measured at a lower pressure. Absorption/release of air by walls accompanying pressure changes is a laminar-like process and the ideal leakage characteristic of a containment structure is one that exhibits laminar behaviour [248]. This lends confidence that the leak rate test itself has not initiated containment boundary failures.

A limitation of leakage rate testing is that it is done while the plant is shut down. For CANDU multiunit stations with a vacuum building, ILRTs of vacuum building containment structures require a multiunit outage. A generic concern has been raised that each pressurization imposes a high magnitude, low frequency cyclic load on the containment that may facilitate ageing. In the USA, containments that exhibit a history of acceptable integrated leakage rate testing results have the option to extend the interval between tests to 10 years. Some plants with good integrated leakage rate testing results have been granted relief by the NRC to extend the interval to 15 years.

Advanced digital methods of monitoring containment behaviour during leak testing have been developed. Reference [481] describes one technique, called digital image correlation, used during a 2011 containment test at the R.E. Ginna plant. During this test, digital images of a portion of the containment wall were used to analyse cracks and concrete strain. Some pictures associated with the test are in Figs 129, 130 and 131.

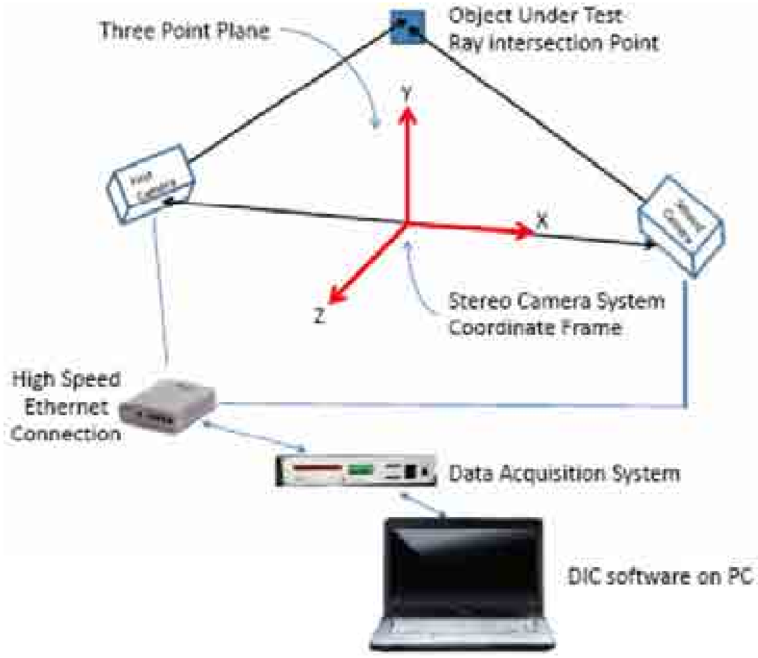


FIG. 129. Digital image correlation test set-up, photogrammetry location principle [481].

FIG. 130. Digital image correlation cameras and inspection location on outside containment wall [481].

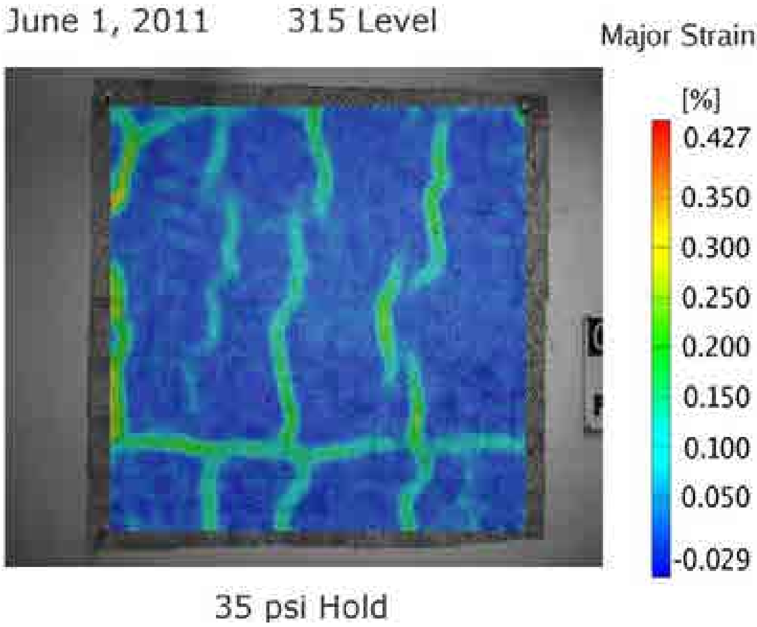


FIG. 131. Digital image correlation principal strain overlay at a hold point. Significant strain 'fingers' are indications of hairline cracks in concrete surface [481].

7.3.2. Instrumentation

Instrumentation in NPP structures is aimed at providing verification of design assumptions, monitoring short term performance (e.g. during initial structural integrity testing) and/or monitoring long term performance. For these purposes, instruments are normally cast into the structure during construction. The first structure of a new design, or the first of multiple structures at a new site, are usually instrumented extensively. Past practice was that subsequent structures were normally not instrumented or contained only nominal instrumentation. However, as it is now recognized that subsequent structures are not always identical to the prototype, new constructions are normally instrumented. While retrofitting instruments for long term performance monitoring is still not widely used in the nuclear industry, it is fairly common (though locally destructive) for hydroelectric power dams.

Typically, installed instrumentation enables measurement of concrete strains/stresses, structure deformation and ambient conditions. Instrumentation can also be used to measure tendon prestressing force (in case of ungrouted tendons). These measurements can provide real time information on structure performance to allow timely detection and management of degradation. They would also allow for monitoring of prestressing force losses to confirm the integrity of the post-tensioning system and containment availability over the life of the plant.

Instrumentation 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 prestressing losses in order to verify design assumptions and to establish baseline data for the subsequent measurements. This is particularly important for grouted (or bonded) systems where direct measurements of prestressing force are not possible after grouting is complete; thus, a reference point is required for monitoring time dependant prestressing force losses.

For existing prestressed concrete structures with grouted post-tensioning systems, retrofitted sensors need to be considered to enable monitoring in order to confirm the structure's response to applied loading (e.g. in case of the pressure loading during leakage rate testing) as well as monitoring of prestressing force losses going forward. When sensors are installed on the outside surface of the containment structure, sufficient protection needs to be provided from the environment, as temperature fluctuations and ultraviolet exposure significantly affect sensor readings, making interpretation of results difficult.

Instrumentation for measuring response of the structure needs to be strategically located to allow assessment of the integrity of the post-tensioning system. The locations should preferably include those for which theoretical values are available to facilitate comparison. Direction of the monitored strain needs to match the directions of the post-tensioning tendons. Temperature probes need to be incorporated with the sensors to allow for temperature compensation of the sensors, as well as isolation of thermal effects.

The possible failure of some of the embedded sensors during construction and over the lifetime of the structure needs to be considered when determining the quantity of the instruments. It is preferred to use different types of instruments for measurements of the same parameters to ensure redundancy and proper functionality of the sensors.

Frequency should be established based on the monitored phenomenon. For example, when response of the structure during the leakage rate test is monitored, frequent (typically hourly) measurements are required. Monitoring needs to start a few days before pressurization and end a few days after depressurization is complete. Less frequent measurements (e.g. monthly) would suffice for monitoring time dependant changes of the containment structure and detecting any unusual trends during operation.

Instruments commonly used in NPPs include:

— Vibrating wire strain gauges;
— Thermocouples;
— Pendulums;
— Extensometers (e.g. invar wire);
— Load cells (also called dynamometers);
— Liquid level gauges;
— Humidity gauges;
— Precision level surveying.

Fibre optic sensors have been used by some utilities. Instrumentation is useful in detecting and monitoring overall and local strain, stress, deformation and temperature changes in CCBs. Particular results may be helpful in assessing concrete volumetric changes (i.e. creep and shrinkage) and loss of force in prestressing systems. These are changes that cannot be detected through common visual inspection techniques.

The advantages of instrumentation are that it can:

— Confirm design assumptions and provide baseline data;
— Provide frequent quantitative measurements of structural performance;
— Be automated;
— Obtain results without plant outages;
— Detect changes at a level below those detectable by visual and many other NDT methods.

The limitations of this method include the following:

— Instruments typically provide structure data only at discrete points.
— Redundancy of instruments is required to protect against individual instrument failure or damage.
— Reference conditions apply when instruments are placed in service and read (which may not reflect full structure history).
— Post-construction installation may cause local damage.

Specific usage, advantages and limitations of various instruments are described below.

7.3.2.1. Vibrating wire strain gauge

Vibrating wire strain gauges (see Fig. 132) may be either embedded in concrete during construction or retrofitted. Vibrating wire strain gauges enable local strain measurements of concrete in the gauge's vicinity. A key gauge component is a tensioned steel wire within the vibrating wire transducer's body, which forms the core of the gauge. An electrical coil around this wire electromagnetically plucks the wire and monitors its vibration frequency. Wire frequency is a function of wire length, diameter, density and tension. For a given wire with a specified length, diameter and density, the frequency (f) of the wire is proportional to the square root of its tension (T) or strain ($f \propto \sqrt{T}$).

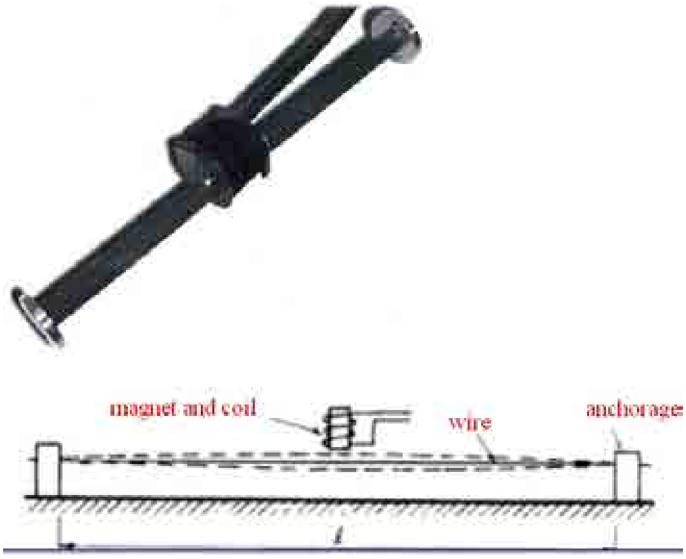


FIG. 132. Vibrating wire strain gauge [482].

A special readout controls the coil and converts feedback into a strain reading. These gauges can lose accuracy with time due to zero drift and radio or electromagnetic interference can affect results. Gauges will not properly reflect concrete strain if they are not dimensioned to avoid the inclusion effect. Sample data from a nuclear application are in Fig. 133.

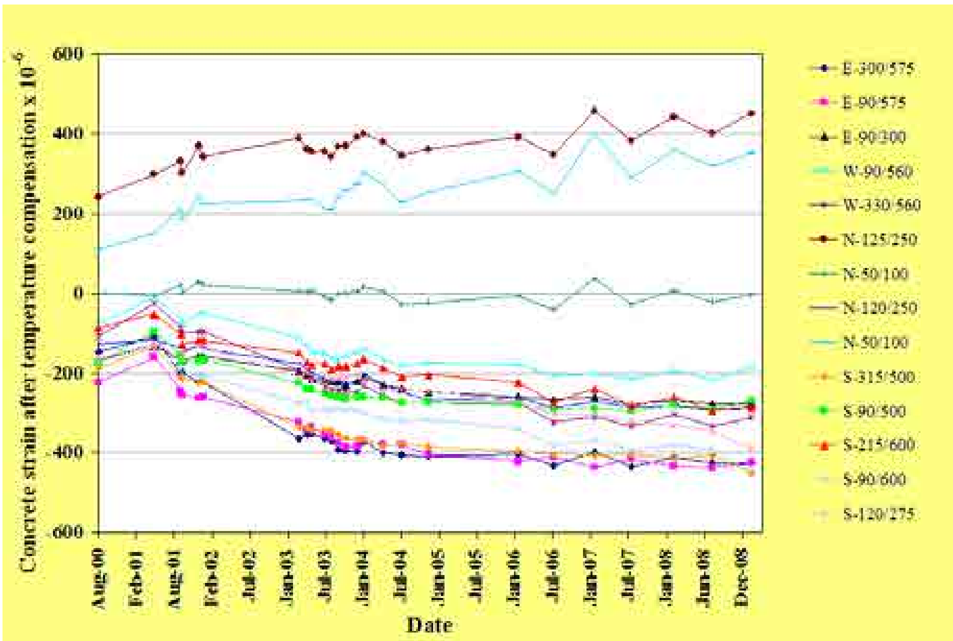


FIG. 133. Example of vibrating wire strain gauge data for a nuclear containment application [483].

7.3.2.2. Thermocouples

Thermocouples consist of two wires of dissimilar metals joined at the ends to form a complete circuit. Materials are selected on the basis of maximum anticipated temperature at the location monitored. Whenever the junctions of the circuit are at different temperatures, an electromotive force exists that is dependent on the difference in temperature between the junctions. Thermocouples measure temperature by keeping one junction in contact with the body to be measured and the other at a known reference temperature, and measuring the electromotive force.

7.3.2.3. Pendulums

Hanging pendulums monitor horizontal movement (e.g. radial and tangential displacements) between two points in the CCB. Typically, a small diameter stainless steel wire hangs from an upper anchor point connected to a weight below a reading table at the unattached lower end.

The weight is usually immersed in an oil bath to dampen movement. Several different reading methods may be used, including steel measuring scales, depth micrometres, optical micrometres and industrial optical vision systems. The instrument requires a clear vertical path between its upper and lower points and protection against draughts (or other environmental effects) and accidental physical damage.

7.3.2.4. Extensometers

While there are many different forms of extensometers, the wire and weight type is most easily applicable to use in CCBs. It measures deformation along its length between the anchor and measuring points. Invar wire is commonly used to limit the gauge's susceptibility to temperature change. Readout methods range from steel measuring scales to depth micrometres to linear variable differential transformers. The limitations of extensometers are similar to those of pendulums.

7.3.2.5. Load cells

Load cells (or dynamometers) are inserted between structural components to measure load transfer within the structure. Types of load cells include devices based on mechanical, hydraulic, electrical resistance and vibrating wire principles. For a description of the principles of operation, and advantages and limitations of each type, see table 13.1 of Ref. [332].

7.3.2.6. Humidity gauges

The two types of humidity sensors commonly used during containment integrated leakage rate testing are:

(1) Chilled-mirror dew point hydrometers;
(2) Lithium-chloride electrical resistance hydrometers (dew cell) [484].

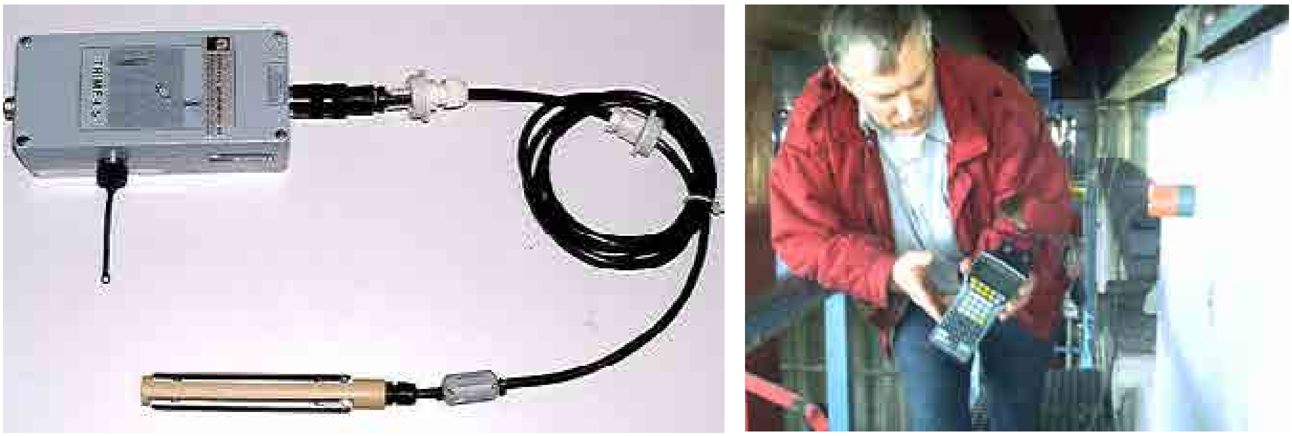
Chilled-mirror dew point hydrometers use photoelectric cells to detect condensation on a mirror with temperature being controlled by a thermoelectric heat pump. A platinum resistance temperature detector senses mirror surface temperature, which is a direct measure of dew point.

Dew cells maintain vapour pressure equilibrium between a saturated lithium-chloride salt solution and the surrounding air sample. A wick of woven glass tape wound around an electrically non-conducting tube contains the aqueous solution. It is heated by two parallel helically wound gold wires with a potential of 25 V (AC) maintained between them. A resistance bulb embedded in glass tape measures the element temperature, which is then converted to dew point by means of the unique relationship between vapour pressure of the saturated salt solution and its temperature.

Without modification, both sensors do not function reliably in the presence of condensing moisture.

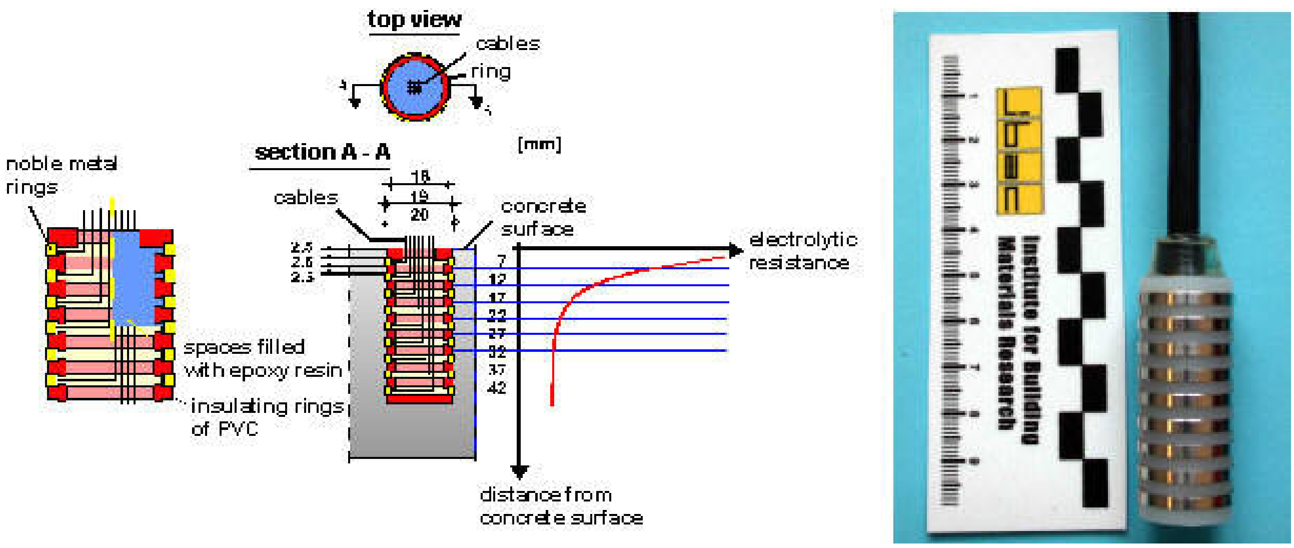
Capacitance probes monitor changes in dielectric constant and dissipation factor caused by moisture in the air to provide relative humidity. Time domain reflectometry is based on electromagnetic wave reflections by discontinuities in electrical impedance of the material (i.e. the dielectric constant is related to concrete moisture content). By measuring an electromagnetic wave's speed of propagation in concrete, the dielectric constant can be determined [485]. Figure 134 presents an example meter based on time domain reflectometry that can be embedded in concrete structures for determination of moisture changes. Reference [486] provides an application of such probes in a networked environment for online measurement of relative humidity and temperature of repaired sandwich type concrete façades.

A multiring electrode method has been developed that can be embedded in new construction or placed into existing structures. It uses resistivity in conjunction with temperature measurements to determine moisture distribution in the concrete cover zone [487]. Figure 135 presents the multiring electrode and sensor. Moisture and salinity of concrete can also be evaluated based on determination of dielectric permittivity in the microwave frequency range of 1–10 GHz using a microwave resonator. This requires positioning of antennas on opposite concrete sides or into two parallel boreholes [488].



(a) (b)

FIG. 134. Application of dielectric measurements to indicate concrete moisture content: (a) TRIME (Time domain Reflectometry with Intelligent MicroElements) sensor for moisture determination (IMKO GmbH); (b) measuring concrete dielectric properties [485].



(a) (b)

FIG. 135. Schematic of multiring electrode set-up and photograph of sensor: (a) schematic of multiring electrode set-up; (b) sensor [487].

7.3.2.7. Embeddable corrosion monitoring sensors

Permanently embedded corrosion monitoring devices are electronic sensors providing real time early warning of conditions that can lead to rebar corrosion damage. In addition to providing information relative to planned maintenance and life prediction of reinforced structures, embeddable sensors are useful for assessing repair effectiveness and determining future repair cycles [424]. Embeddable reference electrodes are of several designs.

The MMO Ti probe (see Fig. 136) consists of a titanium probe activated with iridium enriched metal oxide, cast into a specially developed cementitious filler that maintains constant pH around the probe and guarantees long term electrochemical potential stability [489]. The probe and filler are housed in a plastic protective cover. Electrical contact with surrounding concrete is through the exposed bottom part of the cementitious filler. The ERE 20 probe (see Fig. 137) uses a magnesium dioxide electrode in a corrosion resistant steel housing with an alkaline, chloride free gel and a porous cement plug in front [490]. The gel pH corresponds to that of pore water in normal concrete.

The ERE 20 can be installed in new construction by attaching it to rebar, or in existing structures by drilling holes to the required depth and casting it in place with mortar.

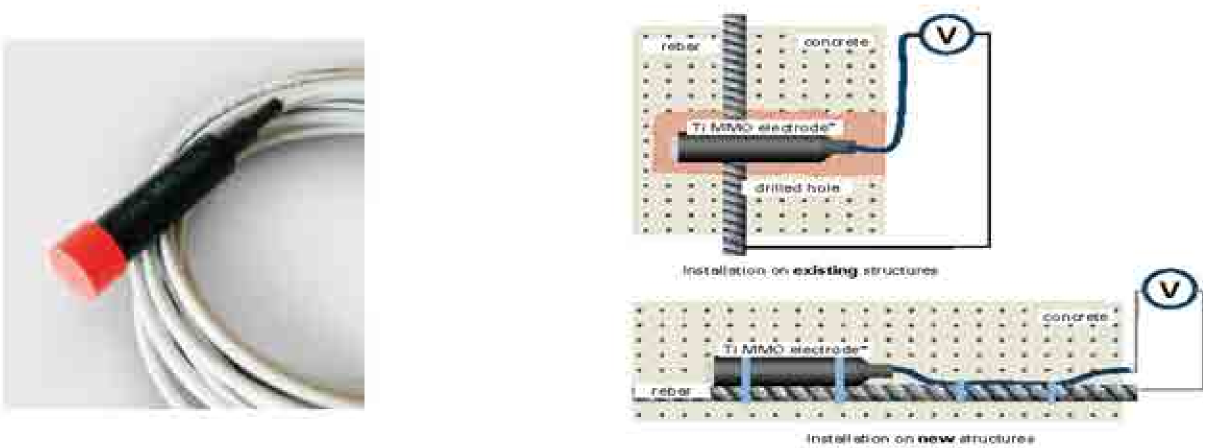


FIG. 136. MMO Ti electrode corrosion sensor [490].

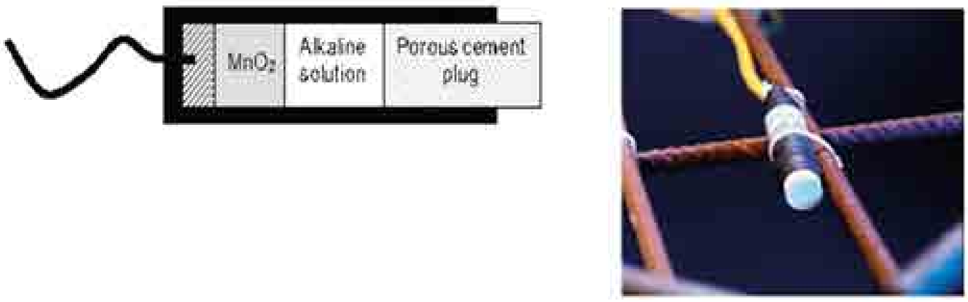
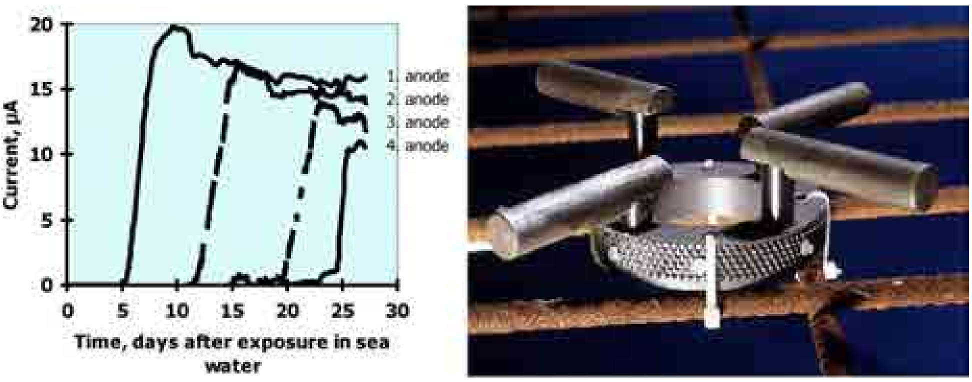


FIG. 137. ERE probe [490].

The wire sensor (silver–silver chloride wire, see Fig. 138) is wrapped around the steel to be monitored. This method has increased sensitivity, making it suitable for measurements of pitting corrosion in large concrete structures.

The CorroWatch multisensor consists of four black electrodes and one noble metal cathode [492]. Anodes are placed in varying, but defined, distances from the concrete surface. Anode heights can be adjusted according to concrete cover. To predict when rebar will start corroding, current between the single anodes and cathode is measured, either with an ammeter or a specially designed data logger. When corrosion starts, current will increase significantly, as illustrated in Fig. 139(a). In the anode ladder system, electrodes are made of steel with a similar composition to that of rebar to ensure they will start corroding at the same time a rebar at the same depth would corrode [493]. Typically, six anodes are used and positioned 50 mm (see Figs 140 and 141) from one another to prevent interaction between anodes. Using adequate calibration models, the time to corrosion can be determined at any time related to cover depth of the rebar. Embeddable probes have been developed for corrosion macrocell current measurements, providing a direct indication of electrochemical activity [494]. Figures 139(b), 140, 142 and 143 are examples of these embeddable probes.

FIG. 138. Wire sensor [491].



(a) (b)

FIG. 139. CorroWatch multisensor and example result: (a) laboratory result showing corrosion initiation at each anode; (b) CorroWatch Multisensor [492].

FIG. 140. Anode ladder system installation for corrosion monitoring [495].

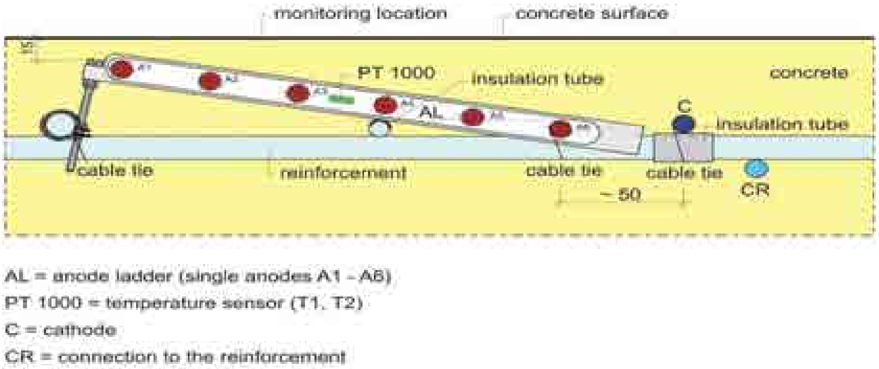


FIG. 141. Anode ladder system in relation to concrete surface [495].

Figure 142 presents an expansion ring system version consisting of the expansion ring anode and a cathode bar [496]. Similar to the six bar anode ladder system, the expansion ring anode system consists of six measuring rings at different distances from the concrete surface in 1 cm steps from 1 to 6 cm. The expansion ring anodes and cathode bar are inserted into holes drilled into the concrete and a turning nut expands the expansion rings to ensure the sensors make good concrete contact. All inner open spaces are then filled with resin-sealing material to ensure that water or chlorides do not penetrate the inner part of the sensor. The expansion ring system is recommended for non-submerged conditions only. The basic principle for determining time to corrosion for the multianode systems is illustrated in Fig. 142.

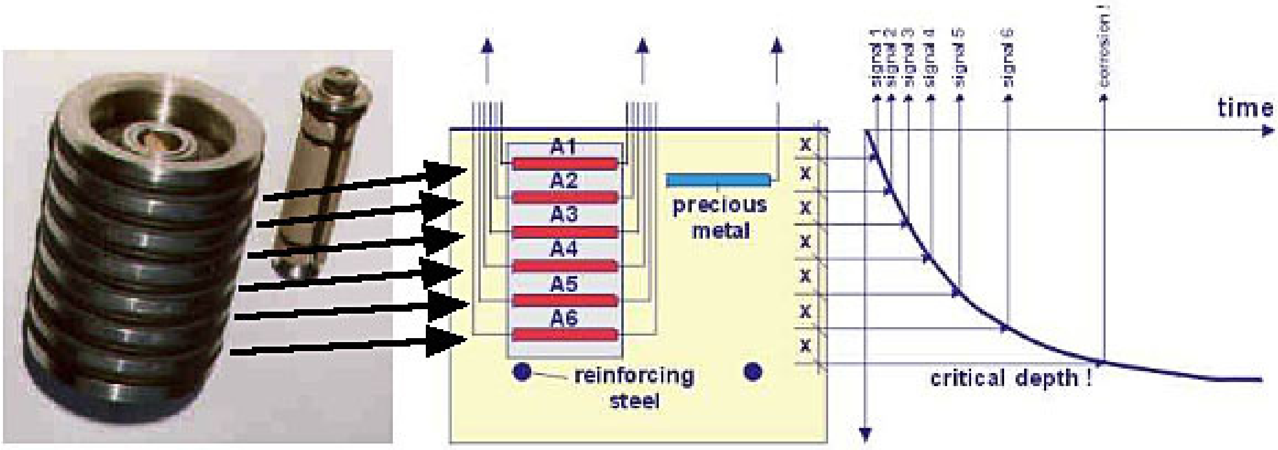


FIG. 142. Expansion ring system and its use to determine time to corrosion [496].

The embedded corrosion instrument, see Fig. 143, comprises sensor electrodes and processing electronics integrated within a moulded plastic enclosure [497]. It is installed during construction at the embedded top level of rebar and monitors five key factors related to rebar corrosion, including:

(1) Linear polarization resistance;
(2) Open circuit potential;
(3) Resistivity;
(4) Chloride ion concentration;
(5) Temperature.

Linear polarization resistance is measured by using a steel working electrode, stainless steel counter electrode and manganese dioxide reference electrode. The unit initiates measurement of open circuit potential between the working and reference electrodes in the potentiostat circuit and applies an appropriate potentiostat drive potential between the counter and working electrodes. A zero resistance ammeter in the potentiostat circuit measures the cell current. Four stainless steel electrodes are used to measure concrete resistivity. A silver–silver chloride ion specific electrode, in combination with its reference electrode, is used to measure chloride ion concentration. Concrete temperature is determined using a solid state sensor. After signal conversion from analogue to digital, measurements are automatically transmitted to a data logger.

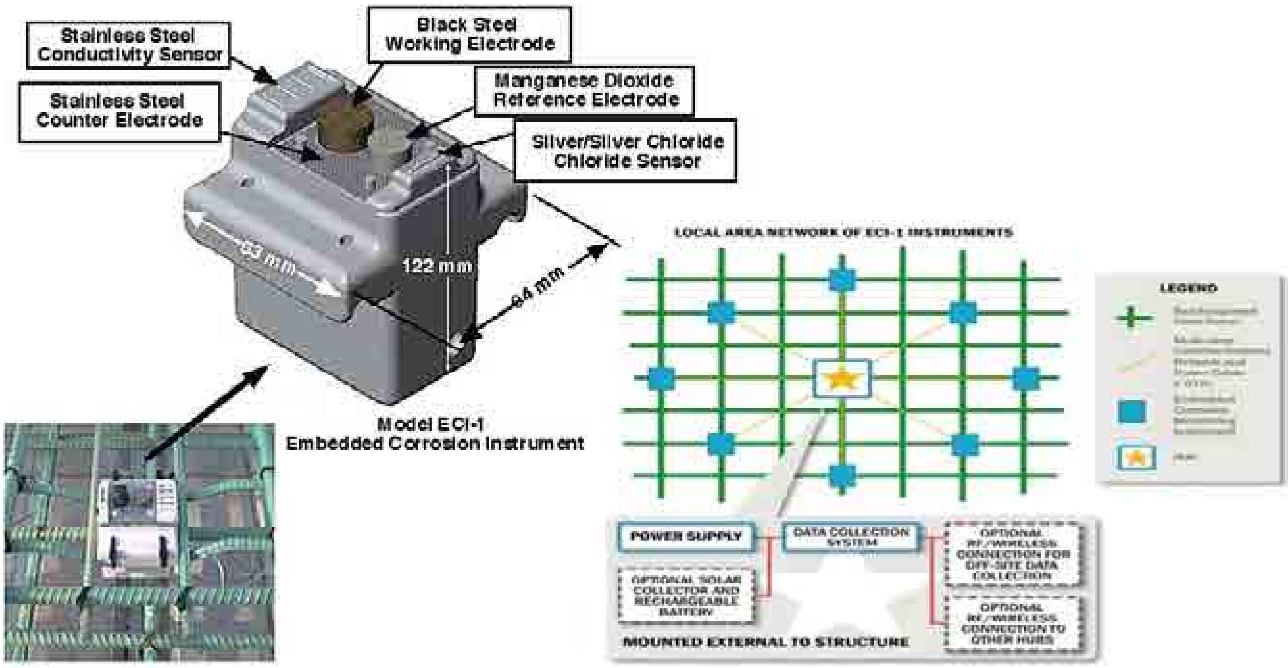


FIG. 143. Embedded corrosion instrument [497].

7.3.2.8. Liquid level gauges

Liquid level gauges incorporate a liquid filled tube between two reservoirs. The relative elevation between end reservoirs is determined by manometer readings or reservoir pressure measurements. The liquids used are water, oil or mercury. Accuracy of the device is affected by tube diameter, measurement method and presence of gas in the tube. Accuracy ranges from 0.1 to 10 mm.

7.3.2.9. Fibre optic sensors

Fibre optic sensors can be either embedded into, or surface bonded onto, different materials, such as concrete, rebar, steel plates and fibre reinforced plastic strips, among others. They can be used for structural strain, crack or temperature monitoring, as well as for overall deformation measurements. They may be installed as part of original construction or during rehabilitation or retrofits. Systems are also available to track acoustic activity in PCCP pipelines to identify events associated with prestressing wire failures (acoustically sensitive fibre sensors inside the pipe can identify deteriorating pipe sections and determine deterioration rates).

Two types of fibre optic sensors are commonly used in civil applications, the Fabry-Pérot sensor (see Fig. 144) and the fibre Bragg grating sensor.

Fabry-Pérot sensors measure strain as a result of a phase change of reflected light resulting from length variations of a cavity between two mirrors at the end of an optical fibre. Sensors are read via a computer linked data acquisition system. Software is used to define time and rate of data acquisition and to store data. To estimate concrete strain due to temperature variations, temperature can also be measured by thermocouples bonded in nearby locations. Examples of sensor results from a nuclear application are shown in Fig. 145.

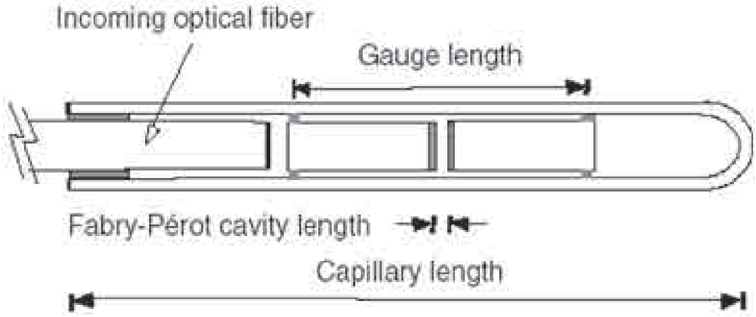


FIG. 144. Principle of the Fabry-Pérot sensor [498].

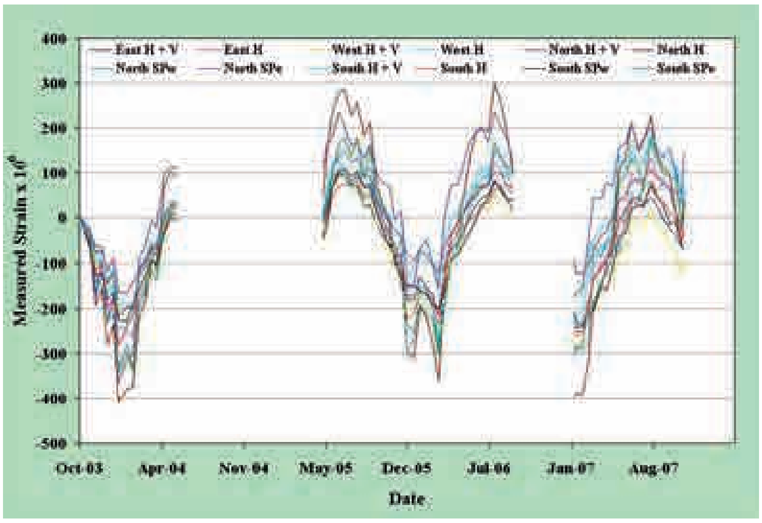


FIG. 145. Fabry-Pérot fibre optic sensor strain data for Gentilly 1 containment ring beam repair application [483].

Fibre Bragg grating sensors (see Fig. 146) consist of a region of germanium doped glass fibre core that has been exposed to ultraviolet radiation using a phase mask to fabricate a periodic grating of material with a modulated index of refraction. Precise spacing of the grating, called the pitch, reflects incident light with a narrow band centred about the Bragg wavelength. The sensor provides a linear response based on the measurement of wavelength shift due to straining of the gauge. After taking into account temperature effects, which also cause a wavelength shift, measuring phase shift provides a means of determining strain.

The sensed information of Fabry-Pérot sensors is the Fabry-Pérot cavity length, which is different from the sensed information of fibre Bragg grating sensors (optic wavelength), however, both are absolute parameters. Neither sensors' output depends directly on light intensity levels and losses in connecting fibres and couplers. Fabry-Pérot technology can be very precise, with a maximum resolution of ±0.01 με (microstrains), while fibre Bragg grating technology is less precise, obtaining a resolution around ±10 με with standard equipment. However, for an Fabry-Pérot sensor, calibration is needed every time readings are stopped. A fibre Bragg grating sensor requires no calibration [498].

7.3.2.10. Acoustic emission monitoring

Acoustic emission refers to the generation of transient elastic waves produced by a sudden redistribution of stress in a material. When a structure is subjected to an external stimulus (change in pressure, load or temperature),

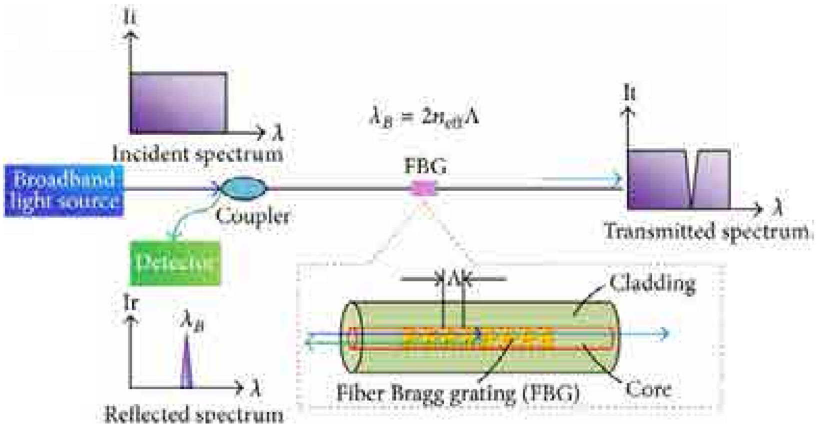


FIG. 146. Principle of fibre Bragg grating sensor [499].

localized sources trigger the release of energy in the form of stress waves which propagate to the surface and are recorded by sensors. With proper equipment and set-up, motions on the order of picometers (10^{-12} m) can be identified [500].

Acoustic emission monitoring is passive in that it listens for sound waves generated by stresses within a material. Acoustic emissions in concrete are due primarily to cracking processes, slip between concrete and steel reinforcement and fracture or debonding of fibres in fibre reinforced concrete. The elastic wave propagates through the solid to the surface where it can be recorded by one or more piezoelectric sensors and analysed (see Fig. 147).

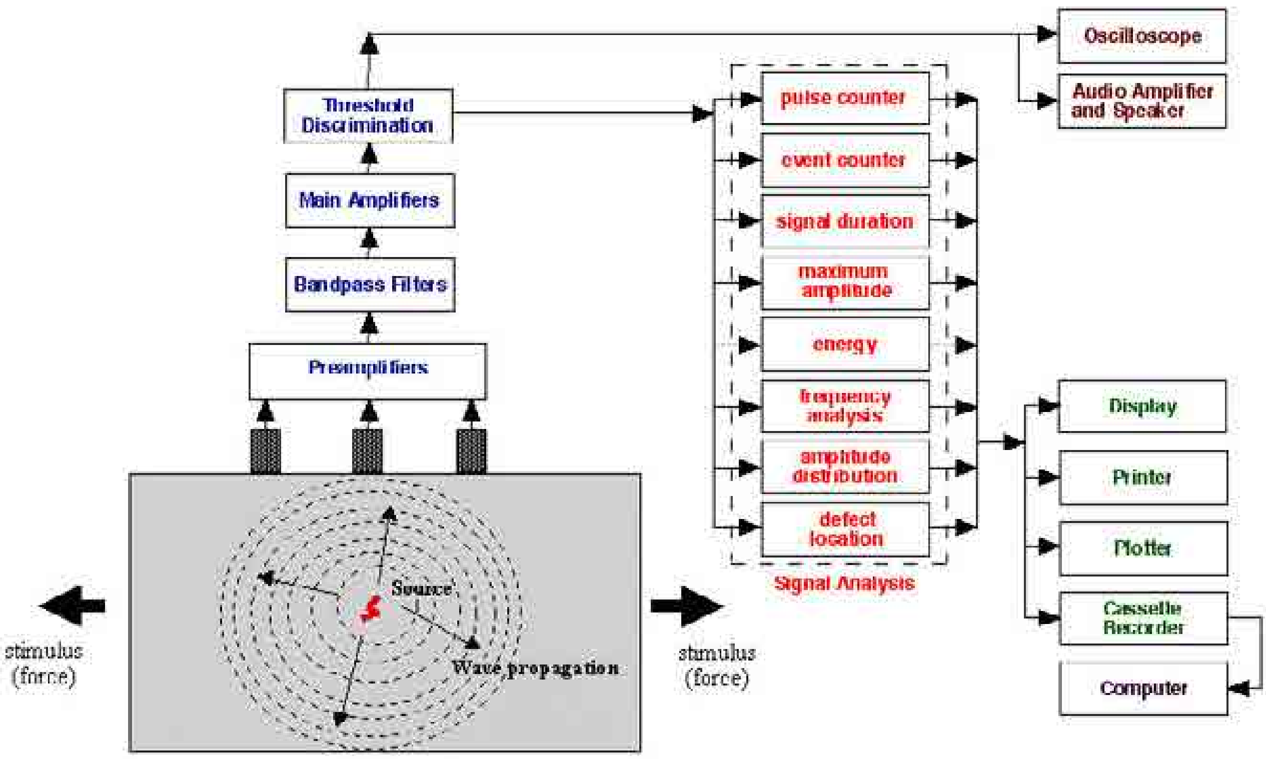


FIG. 147. Acoustic emission principle and schematic of data acquisition [449].

The sensor is a transducer that converts the mechanical wave into an electrical signal, enabling information about the existence of a source and its location to be obtained. Acoustic emission analysis is a useful method for investigation of local damage in materials and enables damage processes to be observed during the entire load history.

The typical applications of acoustic emission include:

— Detection and examination of material and structural flaws;
— Monitoring flaw growth or micro-damage progression;
— Leak detection;
— Assessment of differences caused by manufacturing variations;
— Studying fundamental deformation behaviour and failure of materials;
— Proof or requalification testing of structural components;
— Monitoring, in real time, certain manufacturing processes [501].

Acoustic emission has been successfully applied to concrete members to investigate hardening of concrete, deformation and fracture, condition of reinforced concrete beams, deformation and destruction mechanisms of concrete pipe elements, condition of foundation structures and corrosion of steel reinforcement [502]. A wireless acoustic emission sensor network based on micro-electro-mechanical systems has been developed for structural health monitoring of civil engineering structures [503]. The limitation of this method is that acoustic emission, unless a baseline exists from a series of prior tests, generally only estimates how much damage has occurred to a material and how long a component will last. Also, service environments can be noisy, making signal discrimination and noise reduction difficult.

Acoustic emission methods have found widespread use for the detection and location of wire failures in prestressed concrete structures [504, 505]. Continuous acoustic monitoring has been used since 1994 to monitor failures in bonded and unbonded tendon systems. For the system to be successful, it must be demonstrated that the signals generated by a wire break can be detected above general noise levels and distinguished from events that are not of interest. A typical system includes an array of broadband piezoelectric accelerometers fixed directly to the concrete member and connected to an acquisition system with a coaxial communication cable. Figure 148 presents a standard sensor used for monitoring buildings, bridges and parking structures and time domain and frequency spectrum plots of wire break detected by a sensor 10 m from the event. Figure 149 presents wire break results obtained from monitoring the superstructure of a prestressed concrete bridge consisting of a continuous beam with two spans of 30 m each [506]. By analysing the time taken by the energy wave caused by the break as it travels throughout the concrete to arrive at different sensors, the software can calculate the location of the wire break, usually within 300–600 mm of the actual break. Characteristics of the acoustic events including frequency spectrum are used to classify breaks and reject environmental noise. In most applications, the data are transmitted over the internet for processing and analysis.

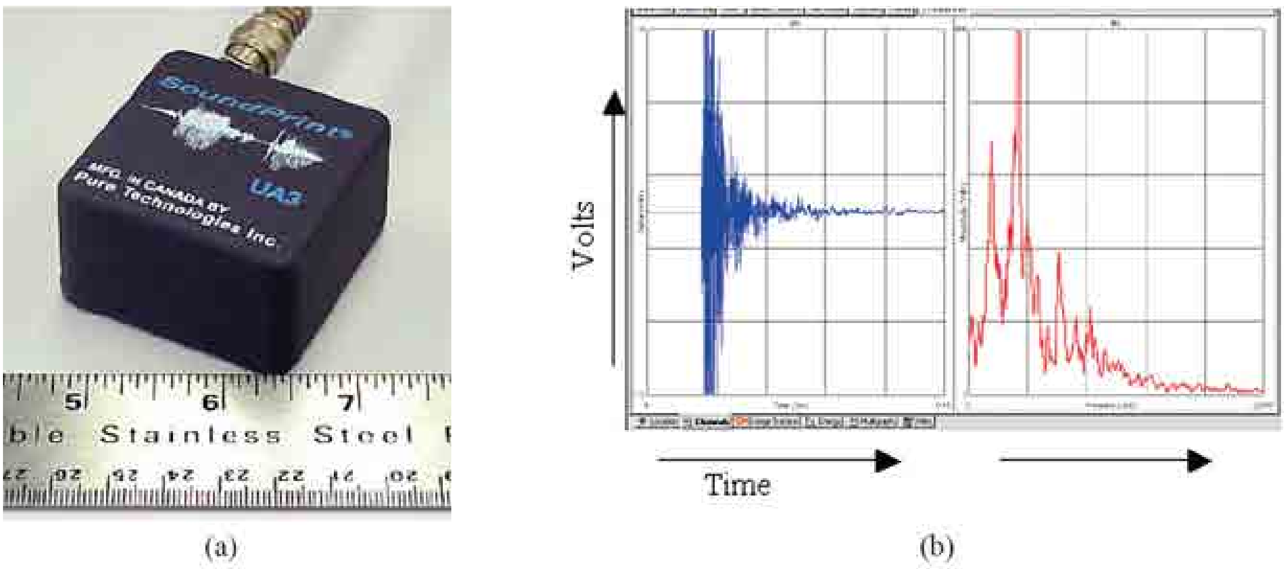


FIG. 148. Acoustic monitoring of tendons in post-tensioned structures: (a) building sensor; (b) time and frequency spectrum plots of wire break detected by a sensor 10 m from the event. [504].

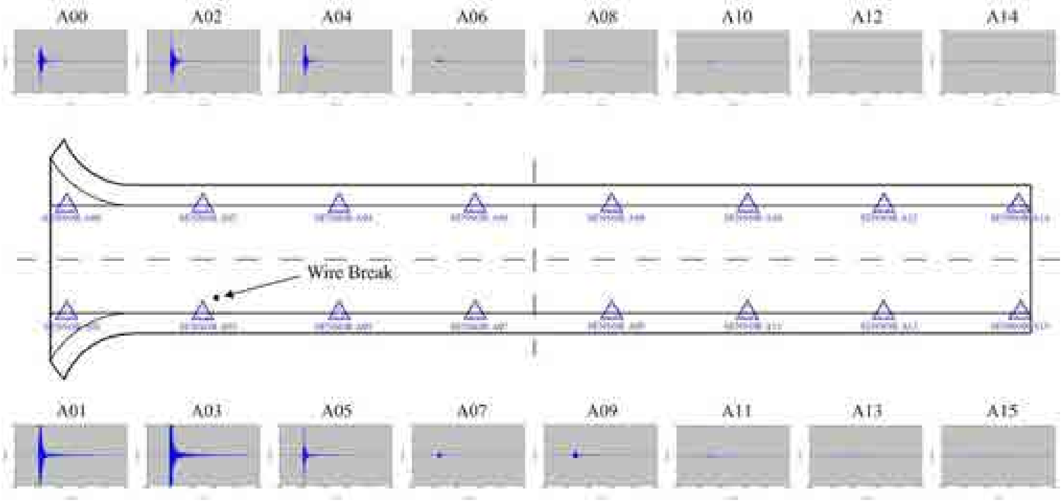


FIG. 149. Signals recorded by 16 sensors resulting from a prestressing tendon wire break [506].

7.3.3. Precise level surveying

Precise (usually first or second order) level surveying provides relative elevations between selected points in the structure and a reference location, frequently a benchmark (deemed to be an unmoving point). Surveying of this nature can provide frequent quantitative measures of structural performance, but requires very high quality equipment, experienced personnel, strict adherence to standard procedures, adequate lines of sight, minimum weather disturbance and absence of heat shimmer. Accuracies of less than 1 mm can be achieved.

Laser scanning is an enhancement of this technique and provides location data for multiple structure points that can be used in structural modelling.

7.3.4. Terrestrial laser structural scanning/metrology

Remote sensing technologies allow measurement of geometric characteristics and other features of facilities and objects without establishing a direct physical connection. One type of active remote sensing technology is laser scanning [507]. Terrestrial laser scanner surveying is similar to geodetic surveying performed by a conventional measuring station. Scanning can be performed at night and can be used to collect data about objects and/or areas where approach would be difficult or perhaps impossible [239, 508–510]. Results can be combined with finite element models to accurately predict structure performance, as described in Section 4.3.1.

The laser light spectrum used spans the 500–1500 nm wavelength range, which overlaps with the range of visible light. During analysis of reflected waves collected by the sensors, wavelengths of reflections (or their wavelength ranges) and their relative proportions can be determined and the measured object’s material and condition (e.g. moisture content), among other things, can be inferred.

Scanners are equipped with a high resolution camera and digital photographs of objects are made during the scan. Resulting colour information can be projected onto the point cloud, providing a photorealistic effect in addition to the exact spatial representation. This can assist analysis, or further processing of the model.

7.3.4.1. Operational principle of terrestrial laser scanners

Three types of scanning instruments exist depending on the method by which the scanner determines distances of points from the instrument, including [507]:

(a) Time of flight instruments: calculate a point’s position by using the elapsed time between emission and reflection (see Fig. 150). The advantage of time of flight instruments is their long range (hundreds of metres to multikilometres for some instruments).

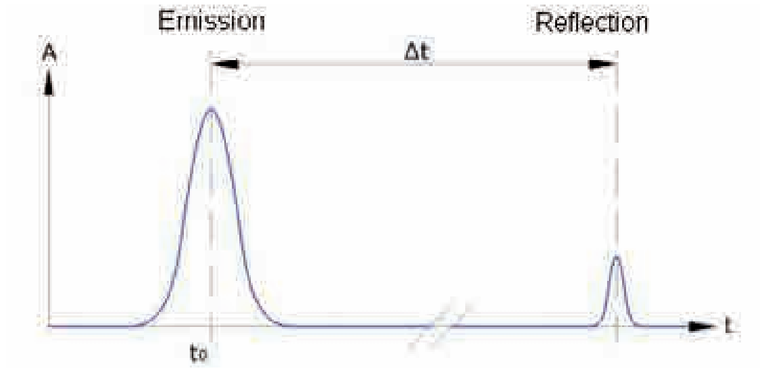


FIG. 150. Operation principle of time of flight instruments [507].

(b) Phase based instruments: calculate the distance via comparing phases of the emitted and reflected beam (see Fig. 151). Compared to the time of flight instruments, their advantage is greater accuracy.

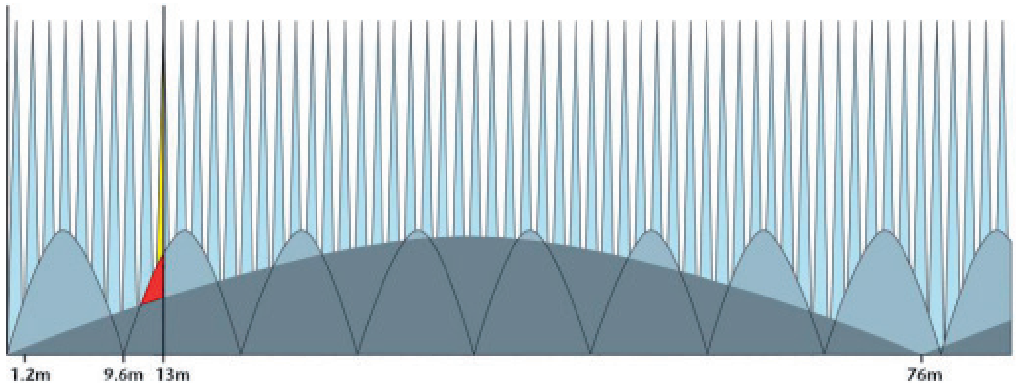


FIG. 151. Operation principle of phase based instruments [507].

(c) Full waveform instruments: enable recording of a high number of reflections to an emitted beam (see Fig. 152), which reduces processing time requirements (primarily for filtering).

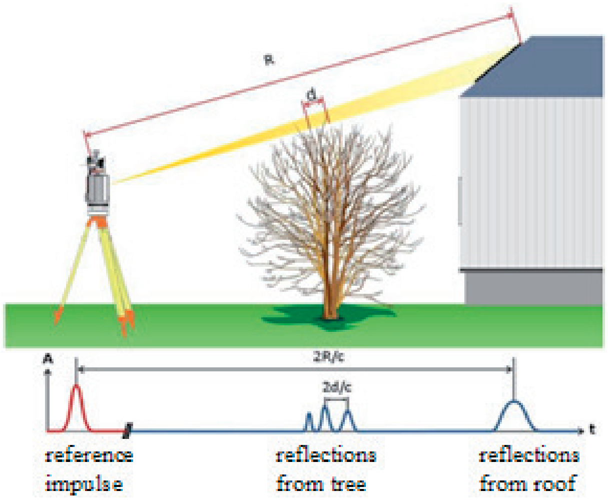


FIG. 152. Recording of full waveform: A is amplitude of signal received; R and d are distances the beam travels; c is the speed of light and; t is time [507].

7.3.4.2. Measurement and processing of laser scanning results

(a) Measurement results

Survey results from laser scanning are a multitude of points, called a point cloud, which are typically stored using the instrument manufacturer's proprietary file formats. However, uncompressed ASCII format is supported by most processing software for both reading and writing [507].

(b) Post-processing and production of results

For usable information, further processing of the point cloud is needed because data may include points that are not part of the object being surveyed. Cleaning and filtering of cloud data is carried out (compression, results oriented query, etc.) and specialized software is used for further evaluation. The following algorithms are normally included [507]:

— *Colour point cloud generation*. Using digital photos made during the survey, representations are made plastic by colouring.
— *Plane fitting*. 3-D shapes bounded by plane surfaces are produced.
— *Cylinder, sphere and cone fitting*. Similar to plane fitting, this produces 3-D objects.
— *Production of an irregular triangular model*. The most common method of calculating height (triangulated irregular network) is applied in terrain modelling.
— *3-D polyline fitting*. This function can be used in edge searching (edge and stairs) and digitalization (transmission line and cable) tasks.
— *Cross-section generation*. This function is important in the field of civil engineering.
— *Error map analysis*. Compares 3-D models of an object with actual measurement results and is mostly used for quality control of manufacturing processes (object scanning).

7.3.4.3. Areas of application for laser scanning

The main areas of application of laser scanning include the following [507]:

— Deformation tests that enable a comparison of results of measurements carried out at different times. Building subsidence or deformations are detectable.
— Survey and modelling of interior areas of buildings and structures, which can be used for representation of hard to reach areas of an NPP and for equipment placement.
— Building façade surveys.
— Modification and updating of construction documents for preparation of as-built and construction designs and condition recording.
— Facility monitoring for construction and operational control and condition documentation.
— Preparation of 3-D records.

Other applications can include land and topographic surveys, geodetic and condition surveys of linear facilities, archaeology, security and urban development.

7.3.5. Cathodic protective system monitoring

As discussed in Section 3.4.5, ICCP systems deliver precise levels of protective current to concrete and masonry embedded steel in concrete infrastructure, pipes and buildings. Monitoring systems are typically integrated with overall ICCP systems. Such systems allow for:

— Remote operation and adjustment in protection levels;
— Remote rapid assessment;

— Ongoing system maintenance;
— Collection and storage of historical operation and performance data;
— Graphical and/or numerical alarms when operating and protective parameters are not reached or exceeded.

Increases in protective current can be related to condition of the coatings or cover over the protected steel. Broken cables or short circuits can result in decreased protective current to the steel. A variety of standards exist for monitoring cathodic protection system effectiveness, including NACE International SP0169 [511], ISO 15589-1 [512], EN 12954 [513], AS 2832.2 [514], OCC-1 [515] and GOST 51164 [516].

7.4. ASSESSMENT OF STRUCTURES

An AMP applies acceptance criteria to results from periodic examinations to confirm that the current plant condition is acceptable and, through historical trending of results and available data, to estimate future performance.

7.4.1. Acceptance criteria

Performance is assessed against established acceptance criteria to confirm concrete's continued ability to meet original design specifications. This requires assessment against original design codes and standards plus consideration of more recent standards, particularly if changes to standards or practices have resulted from improved knowledge of deterioration processes or identification of new failure mechanisms.

Section 5.1 of this publication provides information regarding codes and standards applicable to concrete. Condition indicators, against which acceptance criteria are derived, are described in Section 4.12.

Although some acceptance criteria may be found in design codes, these criteria tend to be limited and wide ranging (for example, see the next section for tables on ranges of permissible crack widths). Determination of common acceptance criteria is difficult because of differing materials, functional requirements, behaviour characteristics, exposure conditions and other conditions.

A tiered approach to condition acceptance is often used. This approach uses evaluation findings (such as a condition survey) to trigger levels of response. In ascending order of severity, responses might be:

(a) Condition is acceptable and requires no further evaluation;
(b) Condition requires more frequent inspection or monitoring to assess whether active;
(c) Condition requires engineering evaluation, which may lead to mitigation or repair;
(d) Condition requires urgent action.

A refinement of this tiered approach is the development of damage charts for selected mechanisms or test methods. These charts provide a graphical representation of the relationship between pertinent factors (e.g. crack width and environmental exposure) and recommended actions. Figure 153 provides one form of a damage state chart. Reference [318] presents examples of such charts for some common degradation mechanisms (e.g. cracking, corrosion and loss of cover). Engineering judgment needs to be used when applying damage state charts.

Specific performance requirement values for plant components may be included in design documentation (e.g. minimum prestressing loads and maximum crack widths), and, if they exist, these values need to be clearly identified. Requirements may define both targets (which may be temporarily exceeded) and limits for developing a tiered approach to accepting an existing condition. Where no guidance is given in design documentation, criteria for the tiers need to be established, based on engineering judgment, for groups of structural components (groupings being defined on the basis of either structural or safety function, or environmental exposure).

For containment structures, IAEA Safety Guide NS-G-1.10 [29] on containment design (sections 4.62 to 4.68 in Ref. [30]) provides details on establishing design acceptance criteria in terms of structural integrity and leaktightness. Suggested leaktightness levels (leaktight, possible limited increase in leak rate, large or very large increase in leak rate) and structural integrity levels (elastic range, small permanent deformation and large permanent deformation) are defined. Design acceptance criteria (including design margins) are established for various load combinations based on the structural design code applied. Containment test pressure acceptance criteria and permissible leak rates will also be established. More detail on containment leak testing is in Section 7.3.1.

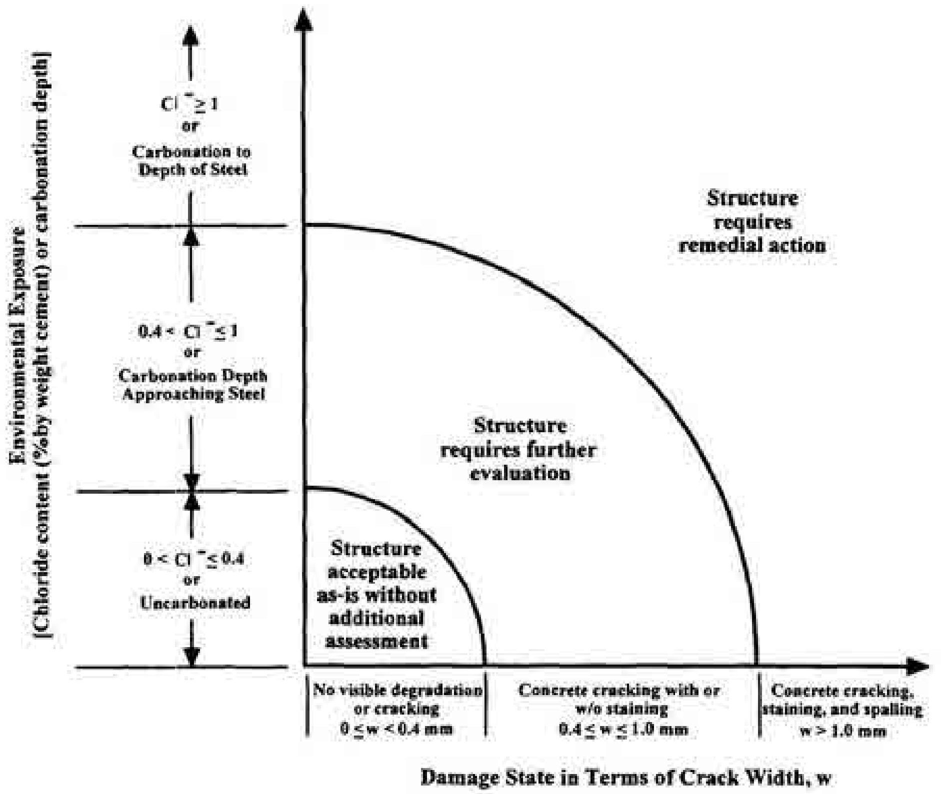


FIG. 153. Damage state chart relating environmental exposure, concrete crack width and necessity for additional evaluation and repair [318].

7.4.2. Evaluate current condition

Inspection and monitoring results are compared to design requirements and other predefined acceptance criteria to confirm structural fitness for service.

Degradation symptoms, if they exist, may require additional evaluation for which various tools and options are available. Key inputs to the assessment are an understanding of ageing and access to key plant documentation. Evaluation rigour must reflect potential defect significance. Key issues to address include:

— Whether reactor safety is threatened. Depending on fault severity, the reactor may either be shut down for repair or operated at reduced load while action is taken.
— Possible cause(s) of degradation (see Section 4 for an overview).
— Extent of damage. Section 7 highlights the range of NDT options currently available; these may be supported by selective destructive tests.
— Rate at which damage is progressing based on ageing knowledge (Section 4) and supported by more frequent inspection or monitoring, if appropriate.
— Structural or safety significance of damage (based on design requirements).

A documented assessment of a structure and significance of noted degradation for long term reactor operation needs to be prepared. This may show that reactor operation can safely be continued. The assessment would normally be backed by plant measurements, calculations (including a revisit of the design calculations) and specialist assessments of potential long term impacts. Where consequences are far reaching, or the process is not adequately understood, it may be necessary to commission material test programmes or simulations (e.g. mock ups or accelerated ageing studies). Outcomes from this evaluation may include recommendations for maintenance, discussed in Section 8.

7.4.3. Estimate future performance

A single inspection of a containment building gives a snapshot of its condition at a given point in time. In isolation, this inspection cannot provide sufficient data for assessing ageing behaviour (i.e. future performance). Comparisons with recorded or estimated data at the time of construction, or historical data, are needed to deduce deterioration rates and to establish trends.

The process of estimating future condition provides an important distinction between an AMP and a series of ad-hoc inspections. A comparison of estimated behaviour with performance limits is used to indicate that concrete condition will remain acceptable until the next planned inspection. If this is not the case, then estimates of time until performance reaches a lower limit will define the planning window for assessing and implementing any needed maintenance or repairs. Figure 154 presents an example of how performance trending can be used as part of an AMP. Also shown is an example of how maintenance is used to re-establish acceptable performance of an RB.

Estimates of future performance are generally based on extrapolation of results from earlier surveys. If degradation is present, an understanding of the processes involved is fundamental because degradation rates vary according to the mechanisms involved (e.g. diffusion or reaction controlled). More frequent inspections may then be planned or instrumentation used to monitor actual degradation rates.

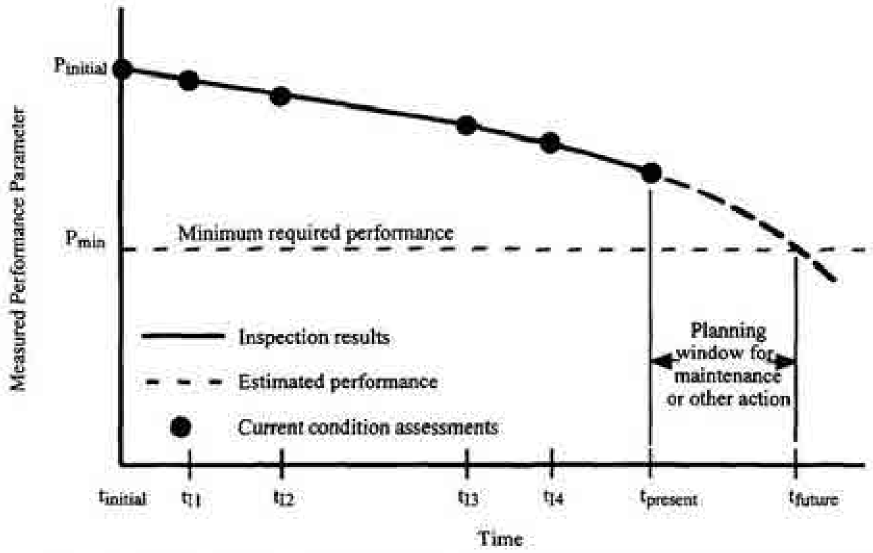
Where field results are worse than anticipated, a review of the structure's AMP, its design basis and related TLAAs (see Section 4.3.2) may be warranted.

7.5. RECORD-KEEPING

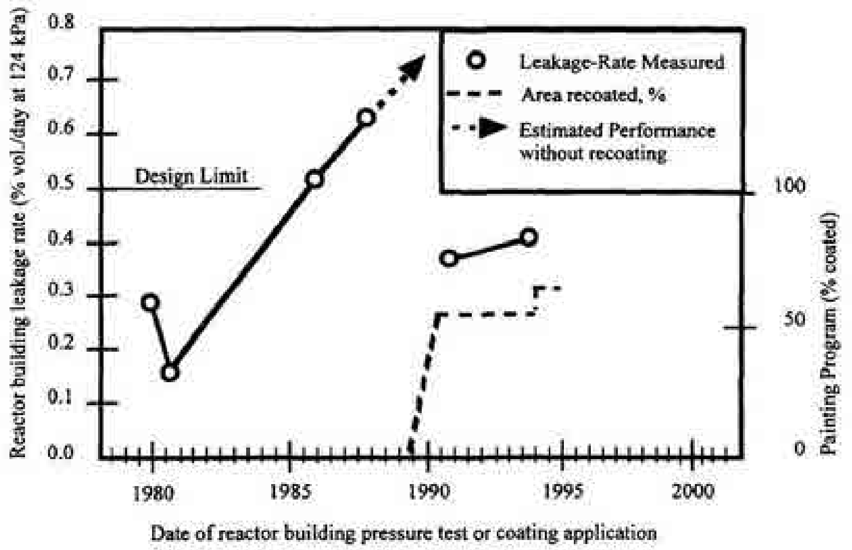
Systematic, effective record keeping is an important part of the ageing management process. This supports evaluation of current condition as well as estimates of future performance.

For visual inspections, permanent records are generally made of concrete condition at the time of the survey and may be used subsequently for trending behaviour (e.g. identifying active/inactive cracks and monitoring crack growth). Items most often identified include cracks, spalling, aggregate pop-out, honeycombing, exudation, distortion, unusual discolouration, erosion, cavitation, seepage, condition of joint and joint materials, corrosion of rebar and soundness of surface concrete. Records may consist of detailed drawings, photographs/videos or a combination of these techniques. To avoid subjectivity, photographs recording the degradation extent need to, where possible, be backed up by quantitative measurements.

Quantitative data provided by other testing and monitoring techniques also needs to be recorded appropriately. Practical guidance on the implementation of an effective system for ageing management data collection and record keeping is given in Ref. [314].



a. Example of how performance trending can be used as part of an ageing management programme.



b. Actual example of how maintenance was used to reestablish acceptable performance of a reactor building. (Adaptation of material presented at the Exhibit Desk of the *Third International Conference On Containment Design and Operation* held October 19-21, 1994 in Toronto, Canada.

FIG. 154. Use of performance trending as part of an AMP and maintenance to re-establish acceptable performance.

8. MAINTENANCE AND REPAIR OF AGEING EFFECTS

8.1. BACKGROUND

Maintenance and repair actions are designed to improve concrete performance so that it never drops below its required minimums. Figure 155 presents the relationship between concrete performance and time, including the impact of repair actions.

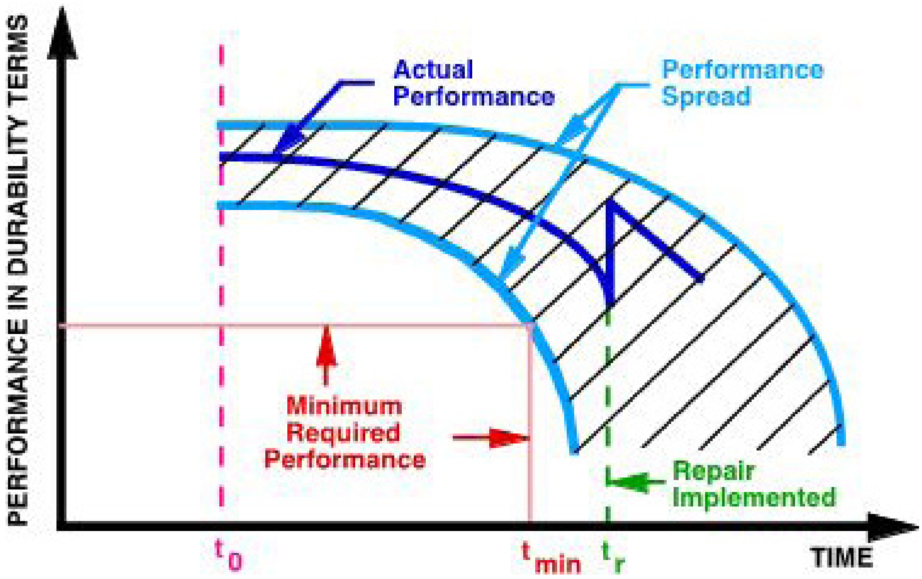


FIG. 155. Concrete performance versus time, including impact of repair action [2].

Improper or inadequate maintenance can contribute to concrete structure deterioration. Examples of poor maintenance include:

(a) Moisture exposure and penetration caused by unrepaired cracks;
(b) Improper application of coatings;
(c) Failure to clean drains and drain pathways.

Programme components include:

— *Diagnosis*: damage assessment, which was covered in the previous section;
— *Prognosis*: can repair be made and is it economical;
— *Scheduling*: priority assignments;
— *Method selection*: depends on nature of distress, adaptability of proposed method, environment and costs;
— *Preparation*: function of extent and type of distress;
— *Application* [517].

Figure 156 indicates typical repair strategy steps for three conditions that could result from a structure's condition assessment: existing damage, damage to be expected or no damage likely [518]. A detailed discussion of each step is provided in the reference.

Most concrete related maintenance and remedial work is implemented in response to an identified defect. Section 4.9.2 provided selected examples of plant degradation and repair experience. Depending on the degree of degradation and residual structure strength, plant response might be any one, or a combination, of structural, protective or cosmetic measures. Typical options considered in response to unacceptable plant degradation are:

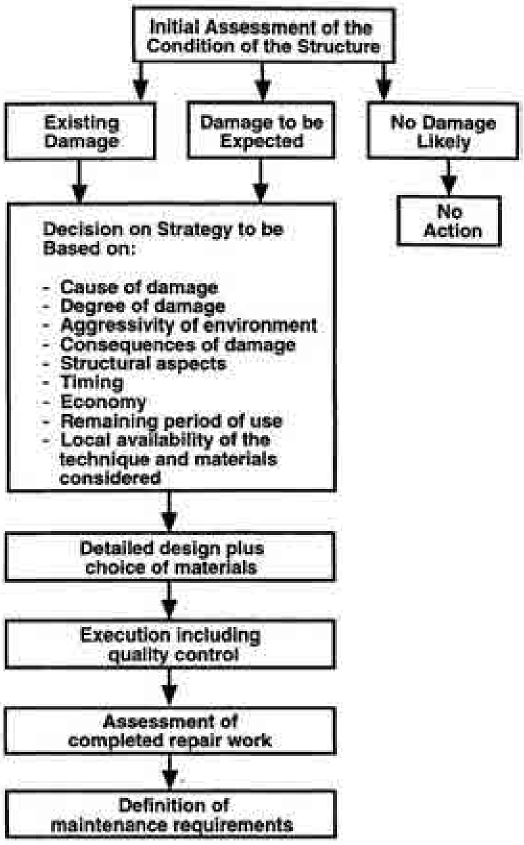


FIG. 156. Steps to be taken in a repair process [518].

- Enhanced surveillance to trend progress of deterioration. This is the initial approach adopted as part of the evaluation process during early stages of degradation.
- Maintenance/operational changes to prevent deterioration from getting worse (if safety margins are acceptable). This might include modified operating conditions (e.g. reducing reactor power, particularly while repairs are planned).
- Local repairs to restore parts of a structure to a satisfactory condition or mitigate further degradation (waterproofing and coating, among other things).
- Component replacement.

Rebar corrosion is by far the greatest threat to civil reinforced concrete structure durability. A high percentage of corrosion damage has been the result of design deficiencies, incorrect estimation of environmental actions and poor workmanship. Many of these structures have had to be repaired or, under extreme conditions, removed from service because of rebar corrosion. Fortunately, incidences of NPP concrete structure corrosion have been limited [81, 235], probably due to more detailed consideration of material selection and construction workmanship, exposure environments and effective use of quality assurance/quality control procedures. However, structure history is somewhat limited and as structures age, incidences of corrosion can be expected to increase.

8.2. PREVENTIVE MAINTENANCE

Preventive maintenance may be seen as proactive intervention where some form of activity is applied prior to damage becoming visible. Various strategies are available and they should ideally include periodic, predictive and planned maintenance activities [479].

Periodic maintenance is servicing, parts replacement, surveillance or testing at predetermined time intervals, operating cycles or operating times. For concrete structures of NPPs parts, replacement is often not an option.

Predictive maintenance is a form of preventive maintenance performed continuously or at regular intervals as governed by observed conditions (i.e. condition based maintenance). The results of such activities may generate required planned maintenance activities.

Planned maintenance is refurbishment or replacement of a system, structure or component that is scheduled prior to its unacceptable degradation.

Proactive condition control is well suited to nuclear facilities since it is important to keep performance above a certain safety level and also to minimize lifetime costs.

In NPPs, a condition based strategy for structures may involve monitoring the number and magnitude of applied load or action cycles, comparing measured values against design values, reviewing structure response and considering corrective actions, including structural changes or modifications in plant operation. In this sense, maintenance of concrete structures is similar to other components and systems in an NPP.

Some preventive maintenance methods related to concrete structures include cathodic protection, chloride extraction, re-alkalisation, electrochemical dehumidification, surface coating and overlays [519]. Possible time based preventive actions include periodic replacement of sealants used for joints, structural holes and gaps irrespective of condition. A condition based action could be the replacement of sacrificial layers due to wear, in which case the exceeding of an allowed wear value invokes the action of preventive maintenance. The line between preventive and corrective maintenance is not precise.

Some common preventive maintenance activities are described in the following.

8.2.1. Non-metallic liners, sealers and coatings

Liners and coatings can be used to protect concrete from environmental exposure that could lead to degradation (e.g. chlorides, sulphates and CO_2). Corrosion of rebar and other metallic embedments can be expected if cover concrete carbonation has occurred, or moisture and sufficient chlorides have reached the steel. Thus, coatings have to be applied to protect against hostile environmental exposure.

The most cost effective approach to treating corrosion is to provide preventive protection or, if necessary, early intervention. Application of barriers in the form of sealers, coatings or membranes to exposed surfaces is a commonly used measure of protection or intervention.

Various materials have been applied to concrete (e.g. boiled linseed oil, sodium or potassium silicates, stearates, silicones, asphaltic emulsion and cementitious formulations). Five categories of sealers are effective in bridge deck applications, including:

(a) Polyurethanes;
(b) Methyl methacrylates;
(c) Certain epoxy formulations;
(d) Relatively low molecular weight siloxane oligomers;
(e) Silanes [520].

Of these, silanes and oligomers have been commonly used. Newer formulations of these materials penetrate concrete surfaces to a degree, but still permit air or water vapour transmission.

Coating types include:

— Epoxy resins;
— Polyester resins;
— Acrylics;
— Vinyls;
— Polyurethanes;
— Cementitious materials.

Membrane types include:

— Liquid applied acrylics;
— Urethanes;

— Neoprenes;
— Vinyls;
— Rubberized asphalts;
— Silicones;
— Preformed membranes, such as:
  • Rubberized asphalts;
  • Neoprenes;
  • Butyl rubbers;
  • Hypalons;
  • Vinyls;
  • Ethylene propylene diene monomer.

Characteristics, advantages and disadvantages of these materials are provided in Ref. [235].

Selection of sealer, membrane or coating materials involves various factors (e.g. compatibility with new or old concrete, compatibility with joint sealant materials, crack-bridging ability, effective service life and weatherability). Typically, materials used as non-metallic liners, coatings and sealers have to be qualified for use inside containment. For example, CSA N287.2 [245] contains qualification test requirements.

Surface preparation (i.e. cleanliness and moisture condition) is extremely important when using sealers or coatings. Temperatures during application, as recommended by the manufacturer, must be observed. Test patches are recommended prior to application to confirm surface preparation, application conditions, thickness of the coating, number of layers and curing, among other things. Adhesion of film forming coatings can be evaluated by ASTM D3359 [451], direct tension (elcometer) or by direct peel. For membrane coatings, 25.4 mm wide strips can be cut in the membrane, clamped to a force gauge and pulled at 180° to the surface [235].

Preventive measures for liners, such as those for containment or SFPs, consist of inspections and maintenance of coating and waterstop integrity.

8.2.2. Electrical protection systems

Electrical/cathodic protection systems are often installed to prevent or mitigate corrosion of rebar and other embedments. Where such protection is installed, regular inspection and replacement of sacrificial anodes is needed. Cathodic protection is more fully described in Section 8.3.2.7.

8.2.3. Waterstops, joint sealants and gaskets

Maintaining (repairing or replacing) waterstops, joint sealants and gaskets helps prevent concrete and steel degradation by keeping aggressive substances away from their surfaces. Certain components are made of elastomers, which have limited lifetimes, especially at elevated temperatures or in high radiation fields. Such components require replacement at regular intervals.

8.3. CORRECTIVE MAINTENANCE

Corrective maintenance includes actions that restore (by repair, overhaul or replacement) a failed SSC's ability to function within acceptance criteria [479]. This may include:

— Coating/sealing repair or reapplication to prevent further degradation;
— Application of new grout or concrete;
— Removal of damaged concrete and rebuilding the structure (in severe cases).

Following any repair, follow up inspections need to be conducted periodically to confirm repair effectiveness.

8.3.1. Repair guidelines

Reviews of repair procedures for reinforced concrete structures have been done from both European and North American perspectives [327, 235]. Several documents are available in the form of guidelines or recommended practices for repair of degraded structures.

In the USA, ACI Committee Reports [521–523] discuss concrete repair, and a consolidated repair manual is available in Ref. [524], which contains recommendations from ACI and other sources. Some repair application procedures are available from the ACI web site. Both the US Army Corps of Engineers and US Bureau of Reclamation have produced concrete repair manuals [53, 525]. The Corps of Engineers manual provides repair techniques and includes chapters on the evaluation of concrete structures, causes of distress and deterioration, selection of materials and methods for repair or rehabilitation, maintenance of concrete, specialized repairs and case histories. Information on material applications and limitations is somewhat brief, however, and service life of repairs is not discussed. The Corps of Engineers has developed a notebook, in the form of a computer database, which provides material data sheets on specific products [526]. Under the joint initiative of the American National Standards Institute and the National Institute for Standards and Technology, a concrete repair task group has been established under the Nuclear Energy Standards Coordination Collaborative and a report was prepared [527].

As discussed in Section 5.1.5, ISO repair standards [282–285] have been issued.

In Europe, many countries have developed national guidelines or standards for concrete structure maintenance, which were usually published in the national language. The most widely known European regulations for concrete repair may be those of the German Committee for Reinforced Concrete [528]. The German guidelines address four areas:

(a) Rules and principles of planning;
(b) Construction and application;
(c) Requirements for establishments and monitoring performance;
(d) Test regulations.

International language guidelines or recommended practices have also been produced by the Concrete Society [529], the Construction Industry Research and Information Association [530], the United Kingdom Department of Transport [531, 532], Comité Euro International du Béton [533] and Fédération International de la Précontrainte [534]. Most European regulations addressed repair of corrosion damaged concrete, indicating the problem's magnitude. As discussed in Section 5.1.4, most European national standards are now superseded by the EN1504 repair standard with national addenda. For example, the former Austrian standard [535] has been superseded by a newer standard invoking EN1504 [536], and the UK Concrete Society has published a technical report [537] on concrete structure repair with reference to BS EN 1504 [538].

The guidelines recognize the importance of understanding the environment's critical role, at both the macro and micro level, and the importance of workmanship during installation. Repair strategies tend to look at the entire repair process as opposed to merely the selection and application of a repair material based on vendor information.

8.3.2. Techniques and materials for repairing damaged concrete

Remedial measures include repair of damaged concrete and mitigation of deterioration causes. Materials used need to be qualified for nuclear applications. Specifically, materials used inside containment structures need to be qualified for accident conditions so that they do not interfere with safety related systems. Qualification tests are given in standards such as N287.2 [245].

Reinforced concrete structure deterioration generally results in cracking, spalling or delamination of cover concrete. Of these, surveys of general civil engineering construction [539] and NPP structures [214, 215, 235] indicate that cracking is by far the most frequently occurring problem. Water seepage through construction joints or cracks was also reported in NPP surveys, as well as honeycombs and voids.

8.3.2.1. Cracks

As discussed in Section 4.5, cracking in concrete structures can occur for various reasons, including plastic and drying shrinkage, thermal effects, fatigue, reactive aggregates and excessive loads.

Table 23 indicates that these cracks may be active or dormant, which influences selection of a repair method. Crack size also influences repair method selection. Concrete structures have two categories of cracks: microcracks and macrocracks. Microcracks form within cement paste adjacent to aggregate particles and are discontinuous, very narrow, require magnification for identification and necessitate no repair action. Microcracks are important because under increased loading they become wider and propagate, and can eventually reach a size (i.e. macrocracks) sufficient to deteriorate concrete, accelerate embedded steel corrosion or produce leakage. Macrocracks are important to service life.

TABLE 23. CAUSES OF CRACKING (*adapted from Ref.* [235])

Cause	Type of Crack		Comment
	Active	Dormant	
Accidental loading		X	For extreme loads, a design assessment will be required that may lead to repair
Design error (inadequate reinforcement)	X		Limit loading to current capacity and repair, or redesign and repair
Temperature stresses (excessive expansion due to elevated temperature and inadequate expansion joints)	X		It may be desirable to redesign to include adequate expansion joints
Corrosion of reinforcing steel	X		Simple crack repair methods should not be used, as steel will continue to corrode and crack the concrete
Foundation settlement	X	X	Measurements must be made to determine if foundation is still settling
Alkali–aggregate reaction	X		Concrete will continue to deteriorate as long as moisture is present; crack repair methods will be ineffective
Poor construction (inadequate curing, formwork, cold joints, etc.)		X	
Design faults • Use of exposed, rigidly connected material to concrete that has a much different modulus of expansion • Stress concentrations • Faulty joint systems	X		

Note: This listing is to serve as a general guide only. It should be recognized that there will be exceptions to all items listed. Adaptation of Corps of Engineers information, Washington, DC [235].

(a) Crack width and durability

Two theories have been proposed regarding how cracks reduce reinforced concrete structure service life [540]. The first is they permit access of chloride ions, moisture and oxygen to the steel, not only accelerating the onset of corrosion, but also providing space for deposition of corrosion products. The second theory supports the idea that cracks may accelerate the onset of corrosion at localized regions on the steel, but states that after a few years of service, there is little difference between cracked and uncracked concrete. Factors influencing corrosion include crack arrangement, width, depth, shape, orientation with respect to steel, intensity, origin, type of structure and service environment [541].

Both crack geometry (i.e. width and depth) and environmental exposure conditions are important to reinforced concrete structure durability. A wide surface crack that quickly narrows with depth may not be as detrimental as a narrower surface crack that penetrates to the rebar. Clear experimental evidence is not available regarding the crack width beyond which corrosion danger exists [96]. For structures located in controlled environments, cracks are primarily an aesthetic concern. Researchers have provided relationships between maximum permissible crack width and exposure conditions. Table 24 presents a listing of permissible mean crack widths and environmental factors that have been proposed to prevent corrosion [234]. Crack width criteria, noted in several codes and standards pertaining to water retention structures, are presented in Table 25, with EN exposure classes shown in Table 26. Larger crack widths increase the probability of corrosion [542], however, as noted in Ref. [543], values of crack width are not always reliable indicators of corrosion or expected deterioration. Crack width criteria contained in ACI349.3R are being utilized by the US nuclear power industry. In general, there are no acceptance standards for crack widths in NPP CCBs, but Ref. [235] provides the following guidance for crack width acceptability:

— Severe exposure to deicing chemicals or watertightness, widths <0.1 mm;
— Normal exterior exposures or interior exposures subjected to high humidity, widths <0.2 mm;
— Internal protected structures, widths <0.3 mm;
— Structures containing chemicals or fluids that must remain leaktight, widths <0.05 mm.

(b) Selection of crack repair technique

After identifying that a crack is of sufficient size to require repair, it is important to determine if the crack is active or dormant. A procedure has been developed for identifying the cause of cracks [53]:

Step 1: Examine the appearance and depth of the crack to establish the nature of occurrence; that is, pattern of individual cracks, crack depth, open or closed cracks and the extent of cracking.
Step 2: Determine, if possible, when the cracking occurred. This requires examination of prior inspection and test reports and talking with persons who operate the structure and possibly those involved in its construction.
Step 3: Determine if cracks are active or dormant. This may require monitoring of cracks for a period of time to determine if crack movement is taking place. Also, attempt to determine if the crack movement detected is growth or simply cyclical opening and closing, such as that caused by thermal expansion. Cracks that are moving but not growing need to be treated as active cracks.
Step 4: Determine the degree of restraint. This requires examination of the structure and construction drawings, if available. Both internal restraint (e.g. rebar and embedded items) and external restraint (such as adjacent structures) need to be considered.

Using a checklist, eliminate as many potential causes as possible. If more than one potential cause remains, the final determination may require a laboratory analysis of concrete samples or detailed stress analysis. A checklist for determining the cause of cracking is presented below.

— Check for major errors in design.
— Check easily identifiable causes, including:
  • Corrosion of reinforcement;
  • Accidental or impact loading;
  • Poor design detailing;
  • Foundation movement.
— Check other possible causes, such as:
  • Incidents during construction;
  • Shrinkage induced stresses;
  • Volume changes;

- Chemical reactions;
- Moisture changes;
- Freezing and thawing.

Having established the cause of cracking, several questions need to be addressed, such as:

— Is repair necessary? Repair of cracking caused by expansion products of internal chemical reactions may not be necessary.
— Should repair be treated as spalling rather than cracking? If damage is such that loss of concrete mass is probable, treatment of cracks may not be adequate (e.g. cracking due to embedded metal corrosion or freezing and thawing would be better treated by removal and replacement of concrete than by crack repair).
— Is it necessary that the condition causing the crack be corrected? Is doing so economically feasible?
— What will be the future movement of the crack?
— Is strengthening across the crack required?
— What is the moisture environment of the crack?

With these questions answered, a repair technique can be selected. Figures 157 and 158 present repair methodologies for dormant and active cracks, respectively [53]. General guidance on crack repair options, including perceived durability, is presented in Table 27. Detailed descriptions of techniques available for repair of dormant or active cracks in reinforced concrete are available elsewhere [53, 81, 235, 342, 521, 523]. Final repair technique selection needs to take into account durability, life cycle costs and labour skill and equipment requirements.

TABLE 24. CRACK WIDTHS TO PREVENT CORROSION OF STEEL REINFORCEMENT (*adapted from Ref.* [235])

Author	Environmental factors	Permissible width (mm)
Rengers	Dangerous crack width	1.0 to 2.0
	Crack width allowing corrosion within ½ year saline environment	0.3
Abeled	Structures not exposed to chemical influences	0.3 to 0.4
Boscard	Structures exposed to a marine environment	0.4
Engel and Leeuwen	Unprotected structures (external)	0.2
	Protected structures (internal)	0.3
Voellmy	Safe crack width	up to 0.2
	Crack allowing slight corrosion	0.2 to 0.5
	Dangerous crack width	over 0.5
Bertero	Indoor structures	0.25 to 0.35
	Normal outdoor exposure	0.15 to 0.25
	Exposure to sea water	0.025 to 0.015
Haas	Protected structures (interior)	0.3
	Exposed structures (exterior)	0.2

TABLE 24. CRACK WIDTHS TO PREVENT CORROSION OF STEEL REINFORCEMENT (*adapted from Ref.* [235]) (cont.)

Author	Environmental factors	Permissible width (mm)
Brice	Fairly harmless crack width	0.1
	Harmful crack width	0.2
	Very harmful crack width	0.3
Salinger	For all structures under normal conditions	0.2
	Structures exposed to humidity or to harmful chemical influences	0.1
Wastlund	Structures subjected to dead load plus half the live load for which they are designed	0.4
	Structures subject to dead load only	0.3
Efsen	Exterior (outdoor) structures exposed to attack by sea water and fumes	0.05 to 0.25
	Exterior (outdoor) structures under normal conditions	0.15 to 0.25
	Interior (indoor) structures	0.25 to 0.35
Rüsch	Ordinary structures	0.3
	Structures subjected to the action of fumes and sea environment	0.2

TABLE 25. CRACK WIDTH CRITERIA NOTED IN SELECTED CODES AND STANDARDS[a]

Designation	Code/standard	Acceptable crack width (mm)	Commentary
ACI 224R-01	Control of Cracking in Concrete Structures, ACI, Detroit, Michigan (2001)[b]	0.41	Dry air or protective membrane
		0.30	Humidity, moist air, soil
		0.18	De-icing chemicals
		0.15	Sea water and sea water spray, wetting and drying
		0.10	Water retaining structures (excluding non-pressure pipes)
BS 8007 [c]	Design of Concrete Structures for Retaining Aqueous Liquids, British Standards Institution, London (1987)	0.20	Severe and very severe exposure
		0.10	Critical aesthetic exposure

TABLE 25. CRACK WIDTH CRITERIA NOTED IN SELECTED CODES AND STANDARDS[a] (cont.)

Designation	Code/standard	Acceptable crack width (mm)	Commentary
EN 1992-1-1	EN 2 Design of Concrete Structures (2004)[d]	0.40	X0, XC1 exposure classes, quasi-permanent load combination, for reinforced members and prestressed members with unbonded tendons
		0.30	XC2, XC3, XC4, XD1, XD2, XS1, XS2, XS3 exposure classes; quasi-permanent load combination, for reinforced members and prestressed members with unbonded tendons
		0.20	X0, XC1, XC2, XC3, XC4 exposure classes; frequent load combination, for prestressed members with bonded tendons
		Decompression[e]	For XD1, XD2, XS1, XS2, XS3 exposure classes; frequent load combination; for prestressed members with bonded tendons
JSCE	Standard Specification for Concrete Structures — 2007, Design, Japan Society of Civil Engineers, Tokyo, Japan (2007)	0.005 × C[f]	Outdoors, ordinary conditions, underground
		0.004 × C[f]	Severe alternating wetting and drying with water containing corrosive substance, mild marine environment
		0.0035 × C[f]	Steel reinforcement under corrosive conditions, structures subject to tidal action
JTG H10-2009	Technical Specification for Highway Maintenance, Ministry of Communications, Beijing, China	0.25	Reinforced concrete
		0.20–0.30	Prestressed concrete
		0.30–0.50	Arch ring
GB-50010-2010	Code for Design of Concrete Structures, Ministry of Construction, Beijing, China	0.3	Indoors, ordinary conditions
		0.2	Humidity, moist air, soil and seawater exposure
SNIP 52-01-2003	Construction Norms and Rules Concrete and Reinforced Concrete Structures, Russian Federation	0.3–0.4[g]	For short term crack opening
		0.2–0.3[g]	For long term crack opening
		0.5	Upper limit for massive hydraulic structures where not otherwise set by relevant regulatory documents

TABLE 25. CRACK WIDTH CRITERIA NOTED IN SELECTED CODES AND STANDARDS[a] (cont.)

Designation	Code/standard	Acceptable crack width (mm)	Commentary
STR 2.02.05:2005	Technical Construction Regulation, Design of the Concrete and Reinforced Concrete Structures, Lithuania	0.4	Reinforced concrete exposed to a controlled indoor environment
		0.3	Reinforced concrete structures exposed to soil or the external environment
		0.15–0.2	Reinforced concrete exposed to gas and alternating cold environment
		0.1–0.3	Prestressed concrete exposed to a controlled indoor environment
		0	Prestressed concrete exposed to an aggressive environment, i.e. external environment, soil, gas and alternating cold environment

[a] Limitation of stresses in reinforcing bars by several codes (e.g. ASME Section III, Subsection CC 3000 [544]) also has a consequence of limiting effective mean crack width.
[b] ACI indicates that a portion of cracks in a structure will exceed these values, and with time a significant portion can exceed the values. Values must be used with sound engineering judgement. In the absence of specific requirements, such as watertightness or specific exposure classes, a limiting value of 0.30 mm is satisfactory with respect to appearance and ductility.
[c] For reference only. Standard is now withdrawn and replaced by BS EN 1992-3:2006.
[d] EN exposure classes are shown in Table 26.
[e] The decompression limit requires that all parts of the bonded tendons or duct lie at least 25 mm within concrete in compression.
[f] Maximum concrete crack width expressed in terms of concrete cover (C) times a constant, with C limited to 100 mm or less.
[g] Based on permeability of the structure (lower number) and corrosion protection of reinforcement (larger number), respectively.
Note: Decompression essentially means that no crack may open (crack width is limited to 0 mm).

TABLE 26. EUROCODE 2 EXPOSURE CLASSES (EN 206-1) (*adapted from Ref.* [276])

Class designation	Description of the environment	Examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze-thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity; concrete permanently submerged
XC2	Wet, rarely dry	Concrete surfaces subject to long term water contact; many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity; external concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class

TABLE 26. EUROCODE 2 EXPOSURE CLASSES (EN 206-1) (*adapted from Ref.* [276]*)* (cont.)

Class designation	Description of the environment	Examples where exposure classes may occur
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools; concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements; car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5 Freeze-thaw attack		
XF1	Moderate water saturation, without deicing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with deicing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne deicing agents
XF3	High water saturation, without deicing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation, with deicing agents or sea water	Road and bridge decks exposed to deicing agents Concrete surfaces exposed to direct spray containing deicing agents and freezing splash zone of marine structures exposed to freezing
6 Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground
XA3	Highly aggressive chemical environment according to EN 206-1	Natural soils and ground

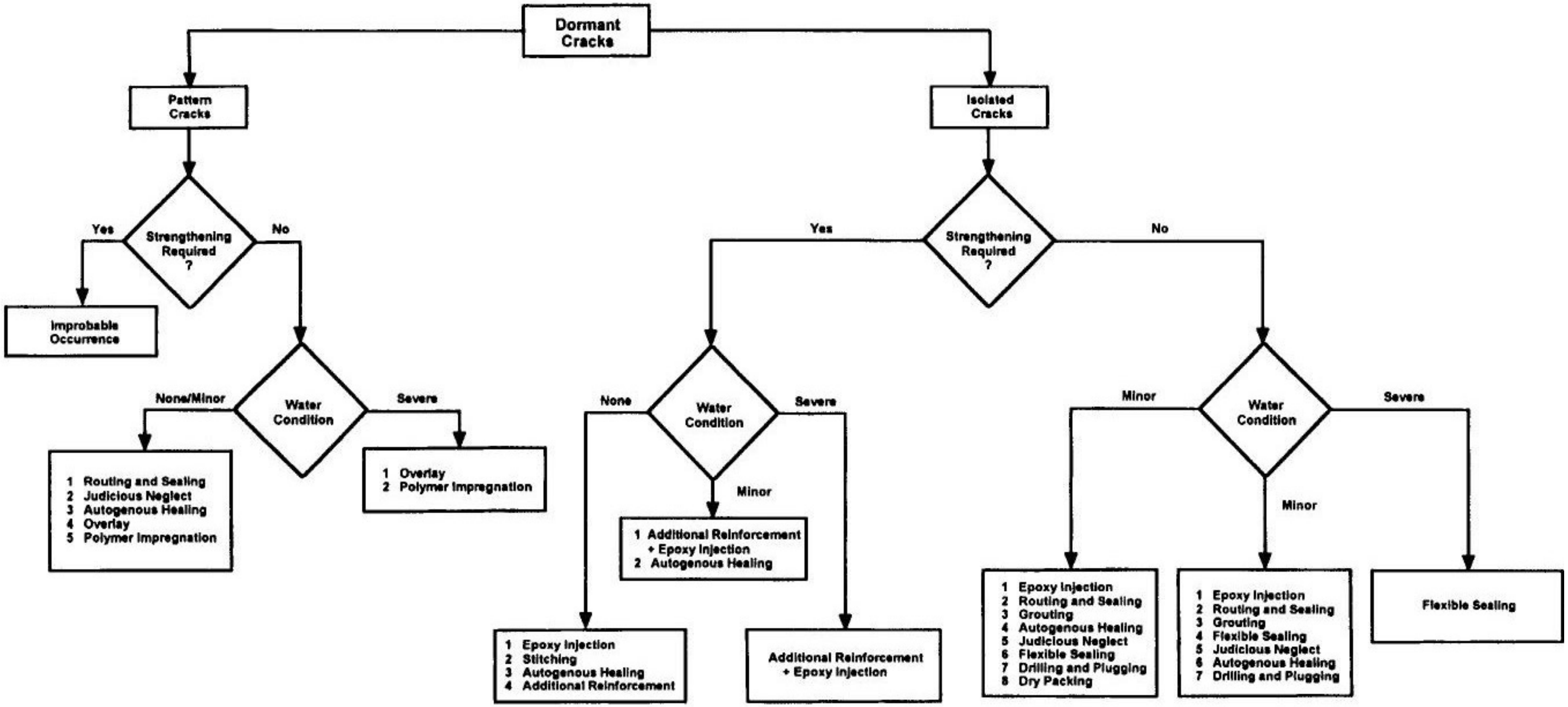


FIG. 157. Selection of repair technique for dormant cracks [53].

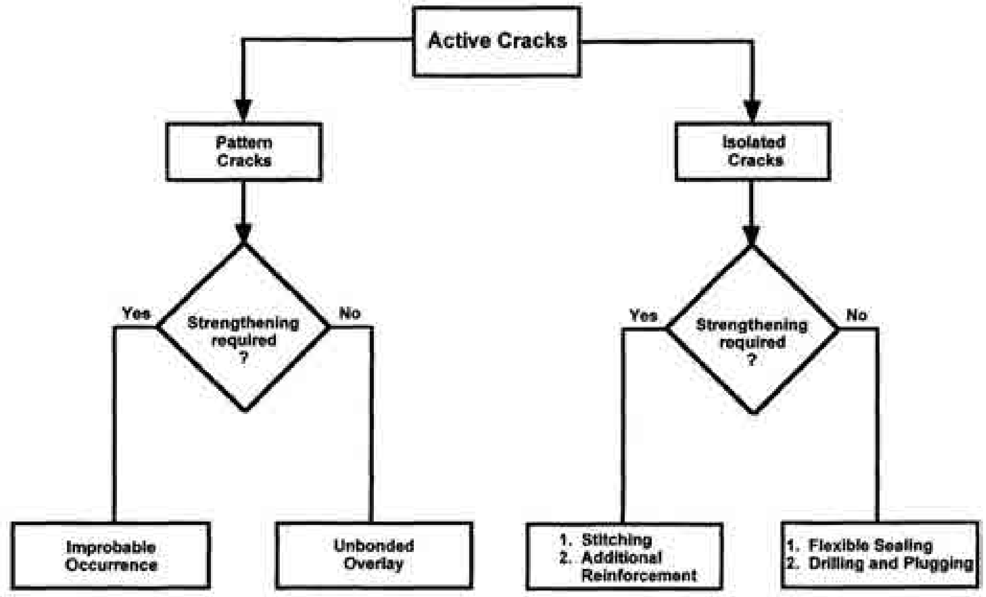


FIG. 158. Selection of repair techniques for active cracks [53].

TABLE 27. GENERAL GUIDE TO REPAIR OPTIONS FOR CONCRETE CRACKING (*adapted from Ref.* [235])

Description	Repair options	Perceived durability rating (1–5)*	Comments
Dormant pattern or fine cracking	Judicious neglect	4	Only for fine cracks
	Autogenous healing	3	Only on new concrete
	Penetrating sealers	2	Use penetrating sealer for H_20, Cl resistance
	Coatings	3	Use coating for abrasion and chemical resistance
	High molecular weight methacrylate or epoxy treatment	2	Topical application bonds cracks
	Overlay or membrane	2	For severely cracked areas
Dormant isolated large cracking	Epoxy injection	1	Needs experienced applicator
	Rout and seal	3	Requires maintenance
	Flexible sealing	4	Requires maintenance
	Drilling and plugging	3	
	Grout injection or dry packing	4	
	Stitching	5	
	Additional reinforcing	4	
	Strengthening	3	

TABLE 27. GENERAL GUIDE TO REPAIR OPTIONS FOR CONCRETE CRACKING (*adapted from Ref.* [235]) (cont.)

Description	Repair options	Perceived durability rating (1–5)*	Comments
Active cracks	Penetrating sealer	3	Cracks less than 0.5 mm
	Flexible sealing	3	Requires maintenance
	Rout and seal	3	Use for wide cracks
	Install expansion joint	2	Expensive
	Drilling and plugging	4	May cause new cracks
	Stitching	4	May cause new cracks
	Additional reinforcing	3	May cause new cracks
Seepage	Eliminate moisture source	1	Usually not possible
	Chemical grouting	2	Several applications may be necessary
	Coatings	4	May have continued seepage
	Hydraulic cement dry packing	4	May have continued seepage

* Scale from 1 to 5, with 1 being most durable.

8.3.2.2. Spalls

Spalls can occur due to impacts, corrosion of embedded metals, erosion or problems such as AAR, freeze-thaw and fire exposure. Surface preparation is critical to successful spall repair. The concrete substrate must be sound and exposed surfaces dry and free of grease, oil and loose particles. Suitable surface preparation techniques include using small chipping hammers (followed by abrasive blasting and removal of dust and chips by compressed air) and high pressure water blasting. If rebar is exposed during removal of degraded material, the excavation needs to be extended so that the steel will be enclosed in the patch material. If rebar is corroded, corrosion products need to be removed and the steel coated with a barrier material such as epoxy resin, or a high electrically resistant patch material [235]. Corrosion-inhibiting admixtures (e.g. calcium nitrate and organic based) can be included in the concrete patch material. Additional information specifically addressing remedial measures for corrosion damaged concrete is presented in Section 8.3.2.7. Shallow spalls (<20 mm) are generally repaired using Portland cement based mortar materials; however, polymer concretes containing epoxies or methyl methacrylates have also been successfully used. Deep spalls are treated in a similar manner to shallow spalls except that coarse aggregate is added to the repair material. To ensure repair durability, the repair material needs to have mechanical and physical properties similar to the in-place concrete and be properly consolidated and cured. For mass concrete requiring extensive material replacement, the repair patch may be built in two or more layers to prevent excessive heat buildup due to cement hydration. Fly ash can be used as a partial replacement for cement to reduce the maximum temperature buildup of the repair mass. If the spall to be repaired is on a vertical or overhead surface, special precautions are required because of the increased difficulty in application of material due to gravity effects. Generally, dry packing is used in which a very harsh mix (i.e. dry) is applied and compacted using a blunt instrument [545]. General guidance on spall repair options, including perceived durability, is provided in Table 28. Additional information on spall repair techniques is available in Refs [53, 81, 235, 545].

8.3.2.3. Delaminations

Delaminations are horizontal voids in concrete domes or slabs that commonly occur due to rebar corrosion or separation of concrete layers that did not develop adequate bonding. Spalls will occur if delaminations are not repaired. Delaminations can be repaired by removal and replacement of delaminated concrete using procedures similar to those for repair of spalls. In areas where concrete removal is not required, the delaminated area can be repaired by injecting an epoxy resin under low pressure or by gravity feed through holes made into the delamination by coring or by using a drill with a vacuum attachment to remove fine particles If water is used in either process, the concrete needs to be permitted to dry prior to epoxy injection. Dowell pins can be used to enhance shear transfer. Additional information on delamination repair techniques is available in Refs [94, 235, 545, 546].

TABLE 28. GENERAL GUIDE TO REPAIR OPTIONS FOR CONCRETE SPALLING (*adapted from Ref.* [235])

Description	Repair options	Perceived durability rating (1–5)*	Comments
Shallow spalling	Portland cement grouts	3	Not good for acid attack
	Polymer modified grout	2	Different thermal coefficient
	Coatings	4	Limited to shallow areas
	Membranes	3	Acids: epoxy, methacrylate, butyl, neoprene
	Polymer grouts	2	Acids use polyester grout
Deep spalling	Portland cement concrete	2	Inexpensive
	Expansive cements	3	Unreliable expansion
	Gypsum based concrete	5	Do not use in moist environments
	High alumina (modified)	3	Bonds best to dry concrete
	Magnesium phosphate	2	Base concrete must be dry
	Polymer modified	2	Thermal stress can be high
	Polymer patching materials	3	Less than 40 mm thickness
	Polymer overlays	2	25–50 mm thickness
	Latex modified concrete overlays	2	Greater than 30 mm thickness
	Portland cement concrete overlays	3	Use low water to cement ratio and high range water reducer
	Silica fume overlays	3	High strength
	Preplaced aggregate	2	Low shrinkage
	Shotcrete	3	Good for large areas

* Scale from 1 to 5, with 1 being most durable

8.3.2.4. Honeycomb and voids

Non-visible voids such as rock pockets, honeycomb or excessive porosity can be investigated by:

(a) Drilling small diameter holes that intercept the voids and determining the extent and configuration of the voids;
(b) Injection of compressed air or water into the void system;
(c) Visual inspection using a borescope.

Depending on void magnitude, repair can be made by injecting epoxy resin, expansive cement grout or mortar or epoxy ceramic foam. Injection of cement grouts requires pre-wetting of the substratum, with excess water removed prior to injection.

8.3.2.5. Water seepage

Water seepage through construction joints and cracks may result in concrete leaching, entry of aggressive environments into the concrete matrix, rebar corrosion or unacceptable flow of fluids either into or out of a facility. Long term reactions that require moisture, such as efflorescence, sulphate attack or AAR, may be initiated at the time that seepage into concrete occurs. Implementation of a repair, therefore, can prevent possible future deterioration and unacceptable fluid flow.

When moisture is present, chemical grouting using silicate, acrylamide, lignin or resin (i.e. epoxy, polyester and urethane) systems is the most effective repair technique. Chemical grouts consist of solutions of two or more chemicals that react to form a gel or solid precipitate, as opposed to cement or clay grouts that consist of suspensions of solid particles in a fluid. The reaction of chemical grout, which may be purely chemical or physiochemical, produces a decrease in fluidity and a tendency to solidify and form occlusions in channels or fill voids in the material [525]. Chemical grout reaction can be in the form of soft, flexible, semi-rigid or rigid gels. When seepage is intermittent and the path through the concrete periodically dries, it can be injected with epoxy resins, or water can be incorporated into a urethane injection system to promote expansion and curing to form a flexible foam material.

To protect the structure, coatings and sealers can be applied (refer to Section 8.2.1) and waterstops and joint sealants need to be maintained (refer to Section 8.2.3).

8.3.2.6. Alkali–aggregate reactions

Three requirements exist for concrete swelling and cracking due to AAR:

(a) Reactive silica or siliceous components in the aggregate;
(b) Sufficiently high hydroxyl ion concentration in the concrete pore solution;
(c) Sufficient moisture availability [547].

Mitigation procedures involve elimination of one (primarily elimination or minimization of exposure to moisture) or all of these requirements. In new construction, control of AAR is done by eliminating deleteriously reactive aggregate materials from consideration through petrographic examinations, laboratory evaluations and use of materials with proven service histories. One of the most reliable accelerated ASR tests is ASTM C1293 [548], and a method for evaluating potential reactivity of carbonate aggregates is available in Ref. [549]. Additional mitigation procedures for new construction include use of pozzolans, restricting the cement alkali content to less than 0.6% Na_2O equivalent, and applying barriers to restrict or eliminate moisture.

Organic (e.g. amines, alkyl-alkoxy-silanes and cryptands) and inorganic (e.g. phosphates, lithium compounds and sodium silicofluorides) chemical agents have been laboratory tested to reduce or alter the course of AAR in concrete [550]. The application of lithium hydroxide, lithium carbonate and lithium nitrite has been found to be somewhat effective [550]. Mitigating AAR in existing concrete structures can be done by interfering with the reaction mechanisms (e.g. drying and sealants), or treating symptoms (i.e. restraint and crack filling) [547]. The application of surface film coatings (e.g. acrylic/polyvinylacetate) to prevent moisture penetration has not always been found to be effective over the long term; however, solvent based silicone coatings have been shown to provide good protection against water penetration for over two years and reduce expansions caused by the ASR [551].

If AAR has stopped, the only repair generally required is filling surface cracks with cement grout or resin. Other methods that have been used, not to mitigate but to counteract the effects of AAR, include strengthening by adding external rebar [552] and cutting joints into the structure to accommodate expansion [553]. Mitigation of the ASR in NPP concrete structures could potentially be challenging owing to the concrete section thicknesses involved, and it may be difficult to reduce water sources.

8.3.2.7. Remedial measures for active corrosion

In situations where preventative measures may not be possible, or corrosion has already begun, remedial measures are required. Basic remedial measures to strengthen or repair reinforced concrete structures damaged by corrosion include:

— Taking no action;
— Damaged component replacement;
— Stopping corrosion;
— Reducing corrosion rate [518].

An example where no action might be taken would be when the structure is near the end of its desired service life, and an assessment indicates that it can continue to meet its functional and performance requirements. Local repairs and monitoring may be part of this strategy. When corrosion damage is localized on exposed surfaces, replacement or partial reconstruction may be the most feasible solution. Often, however, some form of intervention is required to inhibit corrosion (i.e. reduce corrosion rate to negligible values or repassivate corroding areas).

Because anodic, cathodic and electrolytic processes are necessary for corrosion occurrence, repair activities are directed at halting one or more of these processes. Halting cathodic processes requires total blockage of oxygen access to the steel reinforcement. Since this cannot generally be accomplished, this process will not be addressed [518]. Basic principles for halting the anodic and electrolytic processes are presented in Fig. 159.

Brief descriptions of each principle are provided below. Reduction of moisture content will not be discussed as it is essentially the same as described in the previous section covering sealers, coatings and membranes as preventive measures. More detailed information, including proper application, effectiveness, advantages and disadvantages and limitations is provided in Refs [327, 518, 530, 554].

(a) Repassivation

Three methods for rebar repassivation are available:

(a) Using alkaline cement or mortar;
(b) Electrochemical realkalization;
(c) Chloride extraction.

Using alkaline cementitious materials involves placing cement based mortar or concrete in the form of a patch (local), or layer, over the entire concrete surface (general) and relying on the migration of alkalis into the old concrete. If cracks or spalls exist, loose material needs to be removed and loose rust cleaned from steel. New material depth needs to be greater than the estimated depth of carbonation during the remaining desired service life. This method is not effective if carbonation has penetrated greater than 20 mm below the depth of rebar or if steel depassivation has been caused by chlorides [518].

Electrochemical realkalization restores high pH to concrete by generating hydroxide ions at the steel and transporting alkaline material from the electrolyte (e.g. one molar sodium carbonate) into the concrete by capillary absorption, diffusion and possibly by electro-osmosis. This method has been shown to arrest corrosion, but its long term effectiveness is questionable. The method has been shown to be suitable for preventing corrosion in structures where cover concrete has been carbonated, but it is unknown if it can stop ongoing corrosion. Due to hydrogen generation during the process, this method is not recommended where prestressing steel is present and it may affect the steel to concrete bond. This method increases potential for AAR due to the high concentration of sodium ions [327].

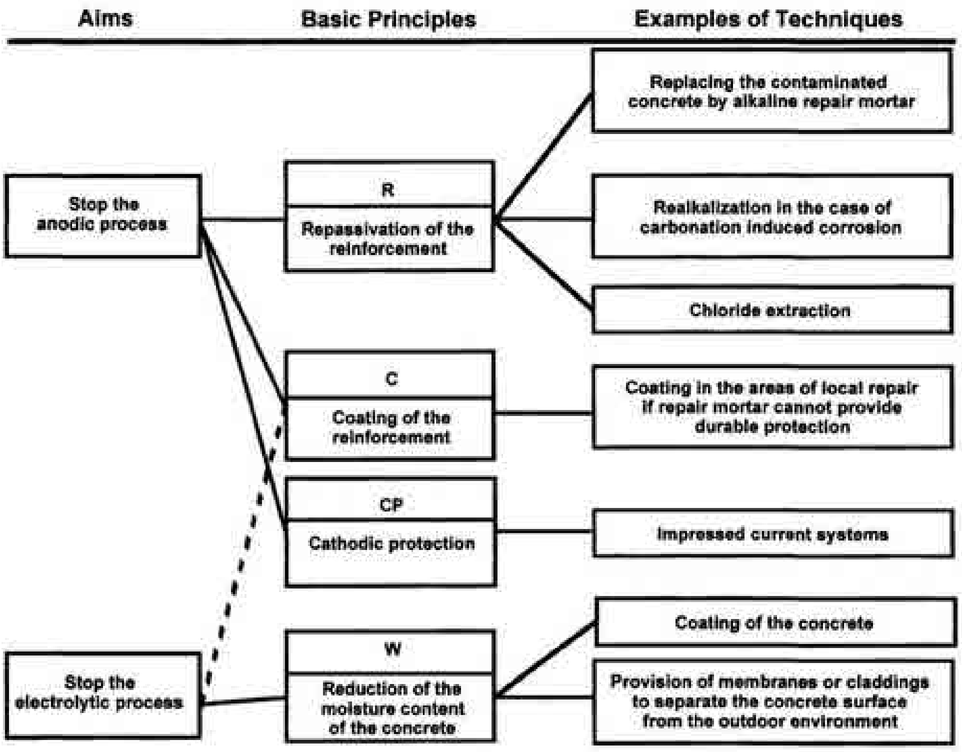


FIG. 159. Basic principles for halting the anodic and electrolytic process [518].

Chloride extraction removes chloride ions from concrete to achieve a residual chloride ion concentration low enough to stop the corrosion process. A direct current field is applied through the concrete by means of an external anode on the surface causing chloride ions to migrate to the surface where they are captured in the electrolyte and removed when the process is completed. This method has often been combined with electrochemical realkalization and has the same basic precautions [327, 518].

(b) Steel reinforcement (rebar) coating

Some repair systems require or include applying a physical barrier (e.g. epoxy) to protect the steel and provide electrical resistance. The procedure requires concrete removal to a depth below the rebar, a clean steel surface free of rust, and sometimes, if badly corroded, replacement of portions of the rebar. If the remainder of the steel is uncoated, and the coating in the repaired area is damaged and becomes corrosively active, a high rate of corrosion can occur. In chloride environments, chlorides may penetrate the coating ends and cause crevice corrosion where steel has not been repassivated. Use of low water to cement ratio cementitious coatings may be preferred in chloride contaminated concrete [518].

(c) Cathodic protection

Cathodic protection of reinforced steel structures is usually limited to high value structures in corrosive environments, such as bridges in marine atmospheres. The approach is similar to those used on structures such as pipelines and offshore marine platforms. Cathodic protection does not eliminate corrosion; it merely transfers it to a less expensive, consumable, non-dangerous, known location, specifically the anode.

Corrosion can be effectively halted, or its rate decreased, by applying a small direct current to rebar, making it slightly cathodic relative to an externally applied anode at or near the concrete surface.

Two types of cathodic protection systems are available: impressed current, and sacrificial anode.

Impressed current systems use a direct current power supply (rectifier) to force current flow from a relatively inert electrode (anode) through the concrete to the steel being protected. The current density necessary to maintain a passive layer on reinforcing steel, before the reinforced concrete is contaminated with chlorides, is relatively low.

Typical operating current densities range between 0.2 and 2.0 mA/m^2 for new reinforced concrete structures. For existing salt contaminated structures, operating current densities range from 2–20 mA/m^2 [162]. Anode materials for impressed current systems include:

- Graphite rods;
- Silicon–iron alloys;
- Lead–silver alloys;
- Titanium rods;
- Niobium rods;
- Conductive graphite impregnated polymer wires;
- Conductive paints or grouts;
- Mixed metal coatings on titanium substrates [555].

Sacrificial systems use a metal (typically aluminium, magnesium and zinc) that is more anodic (higher tendency to corrode) than the embedded steel. When any of these anode materials is coupled to steel, it discharges current, which is picked up by the steel, arresting the corrosion process.

Cathodic protection has been successfully used for several decades and is best suited for applications where the concrete is extensively contaminated and high chloride levels exist at the level of the steel. Removal of contaminated concrete is not required in this situation. Special precautions are required when the following are encountered:

- AAR;
- Lack of rebar continuity;
- Highly electrically resistant concrete;
- Epoxy coated rebar or galvanized steel.

Hydrogen generation at the cathode may embrittle prestressing wires or strands and, since cathodic protection systems are direct current, there is the potential for stray current corrosion in attached structures [518, 530]. Applications of cathodic protection systems to NPP concrete structures are somewhat limited and have generally involved safety related concrete structures other than containments.

Guides such as ACI 222.1R-01 [162] are available and contain information on cathodic protection systems.

8.3.3. Remedial measures related to post-tensioning systems

Tendons and anchor heads of ungrouted systems can be retensioned or replaced. Corrosion preventive medium (grease) can be replaced. This has been done at a few stations where larger than expected loss of prestressing force occurred (e.g. Crystal River 3 in the USA).

Grouted tendons cannot be retensioned or replaced. If prestressing levels fall below minimum design requirements, supplemental prestressing may need to be considered. Such mitigation methods have been used for general civil structures (e.g. bridges). Further information can be found in Ref. [556].

8.3.4. Remedial measures for liners and coatings

8.3.4.1. Metallic liners

Metallic liners are typically exposed to concrete on one or both sides. Protective coatings (e.g. inorganic zinc and polyamide epoxy) are generally applied to exposed interior surfaces of containment metallic liners. The primary location where corrosion occurs is along the liner circumference adjacent to upper portions of the concrete basemat [557, 558]. The cause is attributed to waterstop breakdown allowing fluids to accumulate in this region. Remedial measures involve inspecting the region, cleaning, recoating and reapplication of the waterstop. In some cases, material loss is sufficient to require weld overlays or application of replacement material patches.

In one instance [557] where corrosion had progressed to the state that 1 cm diameter holes had penetrated the liner, repairs involved removing concrete adjacent to the region, sandblasting the liner, inspecting, welding plates over areas where holes were present and painting. In addition, corrosion inhibitor was injected into the space between the liner and floor slab and a redesigned, more durable, waterstop was installed.

Another liner repair method involved replacement of a liner steel plate section. Steps for this repair included (see Figs 160 and 161):

(a) Cutting out faulty liner;
(b) Placement of flat steel spacer and rebar anchor, repairing concrete structure;
(c) Cleaning steel sheet surfaces along the joint of existing and new liner;
(d) Welding in a new piece of steel sheet;
(e) Weld inspection;
(f) Welding up of the examining steel sheet;
(g) Restoration of coatings.

During a liner failure repair, possible reasons for, and effects of, corrosion have to be investigated (e.g. foreign material, lack of concrete, etc.) and appropriate measures carried out for effective corrosion prevention.

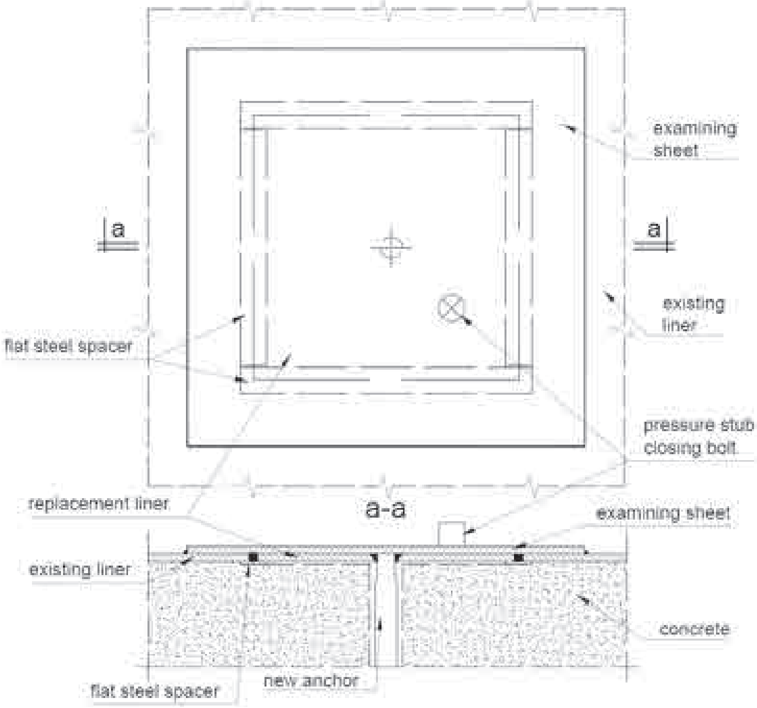


FIG. 160. Repairing liner with steel sheet.

FIG. 161. Repaired liner.

8.3.4.2. Non-metallic liners and coatings

Areas of non-metallic liner can be visually inspected for degradation, and leaktightness can be re-established by applying an additional coating such as polyurethane. Repairs are typically performed by installing new liners on top of old ones after the surface has been properly prepared and compatibility of materials evaluated. Alternatively, the old liner can be completely removed, the substrate properly prepared and new liner applied. In either case, the repaired liner system needs to be qualified for application by tests that include the substrate, surface preparation and combination of old and new material, including the thickness and all layers of each material.

Liner systems include originally applied materials (e.g. epoxy, fibre glass reinforcement, and/or urethane) and repair materials, as applicable. The thickness and number of layers of each material need to be defined and constitute a part of the system as far as qualification testing is concerned. As a minimum, liners and coatings applied inside containment need to be tested for design basis accident conditions and for adhesion (to avoid interference with safety related systems if the material detaches from the substrate).

To ensure long term integrity of liner repairs, installation needs to be performed by qualified personnel in accordance with approved procedures.

Coatings are generally inspected visually, and areas of deterioration are repaired by cleaning and recoating with a compatible material.

8.3.5. Remedial measures for spent fuel pools

For SFP with metallic liners provided with a leak chase system behind seam welds, it is important to ensure that drainage system channels are free from obstructions in order to allow for leakage collection and prevention from exposure of concrete and carbon steel components to water.

Underwater welding techniques are available for application if the leakage location can be detected. Accessibility may also be an issue due to obstructions present in SFPs. Additional information related to underwater welding can be found in Ref. [559].

For SFPs with non-metallic liners, underwater repairs can be considered by recoating. However, this approach requires accessibility, careful surface preparation, material selection and application to ensure a long lasting solution. For rigid liners, concrete cracks or joint movements may cause cracks in the repair material. EPRI and EDF have documented a robotized process for polymer repairs of spent fuel pools [560].

8.3.6. Remedial measures for cooling towers

Repairing corrosion induced deterioration in cooling towers typically involves removing deteriorated concrete, undercutting, cleaning and protecting reinforcing steel and re-establishing the original concrete section.

Corrosion reoccurrence can be reduced by installing a sacrificial cathodic protection system for the affected concrete components, typically by encapsulating mesh anodes within a stay-in-place fibreglass form filled with cementitious grout. The mesh is connected to existing reinforcing steel by means of insulated copper wires. Figure 162 shows a typical installation. System performance can be periodically monitored to detect when anodes need replacement.

In cases of severe degradation, repair involves replacing selected beams and columns (usually in mechanical towers). Epoxy coated rebar is generally used for this purpose despite its higher cost.

8.3.7. Remedial measures for water intake structures

Repair of intake structures typically involves sealing of cracks and spalls using specialty epoxies, grouts and patch materials. Underwater specialty grouts are used for concrete surfaces under water, and underwater repair guides are available (see Section 8.3.10).

Cathodic protection can be used to prevent corrosion damage; however, its use is not common in these types of structures.

Concrete deterioration by biofouling (microorganisms attached to surfaces) is to be regularly addressed to avoid further multiplication of organisms that can be detrimental to mechanical equipment. Additional measures, such a chemical treatment systems, are often necessary.

FIG. 162. Cathodic protection system being installed as part of cooling tower concrete repair [555].

8.3.8. Remedial measures for concrete pipe

Concrete pipe repairs are either applied externally or internally. External options include applying additional post-tensioning, pipe replacement with a closure piece, replacement of a portion of the pipe, concrete encasement, external steel cylinder reinforcement or application of an external repair, such as a clamp or wrap material such as fibre reinforced plastic. Internal repair methods include carbon fibre reinforced plastic hand lay-up repair, installing a steel or fibre reinforced plastic internal liner or conducting a slip-lining repair with high-density polyethylene or other material.

An example of internal repair is the cured in place pipe process, whereby a resin saturated felt or fibre tube is inverted, using water or air pressure, or pulled into the damaged pipe. The resin is cured using hot water, ultraviolet light, ambient light or steam and forms a tight-fitting, jointless and corrosion resistant replacement pipe. Service laterals (i.e. adjoining pipe connections) are cut in the repair material using robotic cutting machines. Cured in place pipe repairs are typically designed for a 50 year life and can be made on pipe that is partially deteriorated (unrepaired pipe able to withstand soil and highway loads) or fully deteriorated (unrepaired pipe unable to withstand any loads) [561]. Figure 163 shows a process schematic using ultraviolet light as the curing method. Some applicable standards include ASTM F2019 [562], F1216 [563] and F1743 [564]. Liner repairs incorporating rebar have also been developed [565].

Failure risk reduction, life extension and increases in reliability of PCCP may also be accomplished by applying cathodic protection to the prestressing wires.

8.3.9. Remedial measures for foundations

Repairs to NPP basemats are similar to those in general civil structures, though access to affect repairs can be a problem. Typical remedial measures can include additional piles, groundwater level control (e.g. pumping),

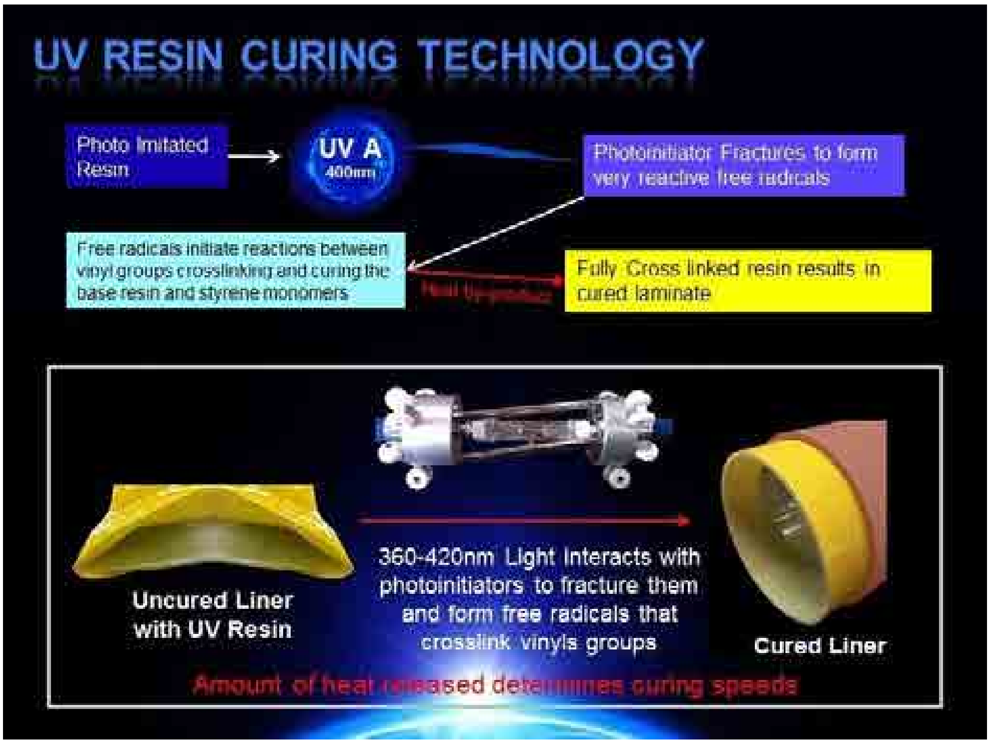


FIG. 163. Cured-in-place-pipe repair process [561].

crack or anchorage repairs, waterproofing, machine foundation modification or soil strengthening by cementitious grout injection.

Soil strengthening by cementitious grout injection is used for water control and/or structural improvement [566]. Its advantages over removal and replacement of poor soils are that it is an in situ treatment that is less disruptive to surrounding areas. Polyurethane grouts can be used for some applications such as concrete levelling and void filling.

Structural improvements include:

— *Densification*. Whereas deep dynamic compaction and vibro-compaction are pre-construction improvements. Compaction grouting and soil fracturing are used as successful techniques for densifying loose granular soils beneath existing structures.
— *Settlement mitigation and restoration*. Slab jacking and soil fracturing have been used successfully to restore post-construction settlements. Pre-construction grouting fills voids to prevent future settlement.
— *Ground strengthening*. Grouting has been used under existing structures to prevent or reduce settlement due to adjacent excavations, dewatering and poor soil condition, among other things. Strengthening can also provide lateral support for excavations, tunneling and foundation underpinning.
— *Liquefaction mitigation*. Densification and strengthening of loose granular deposits to mitigate liquefaction potential.
— *Slope stabilization*. Creating improved soil support.

Figure 164 provides a visual representation of different types of grouting such as:

— Rock fissure grouting;
— Rock void (slurry, intrusion and splitting) grouting;
— Soil permeation (penetration or chemical) grouting;
— Soil compaction (displacement) grouting;
— Soil jet (replacement) grouting;
— Soil fracture grouting [567].

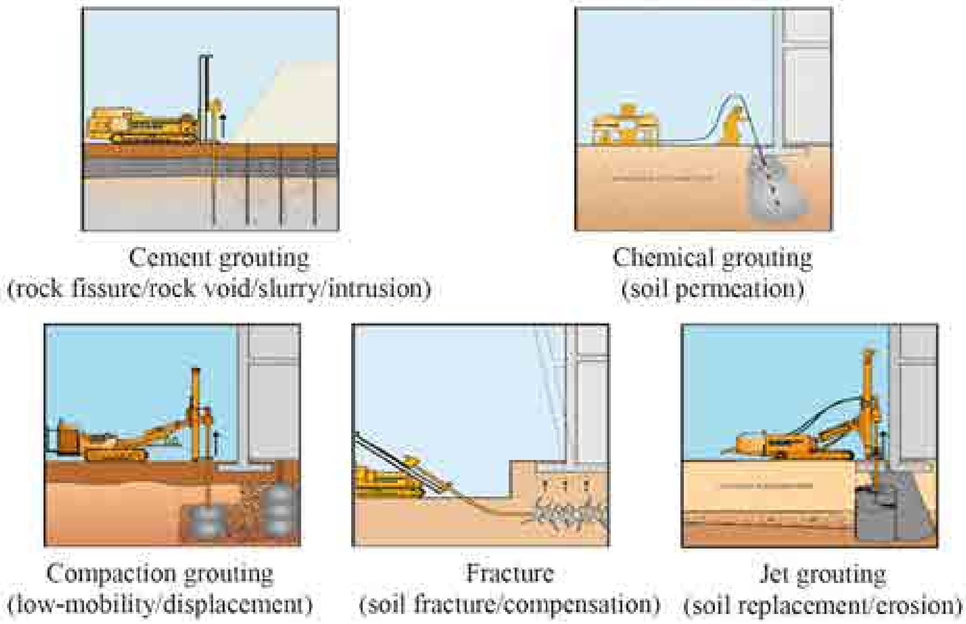


FIG. 164. Types of grouting (courtesy of Hayward Baker/Keller GmbH).

Figure 165 shows a computer generated image of soil permeation/chemical grouting done at the Pickering Unit 8 NPP to reduce settlement of its main output transformer.

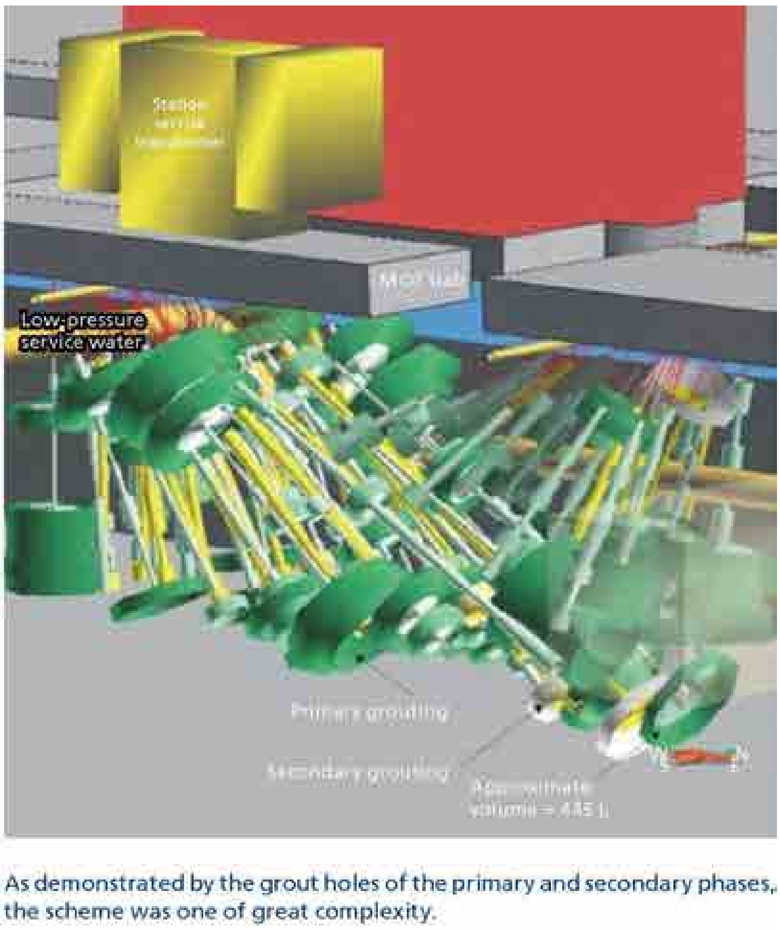


FIG. 165. Model of grouting completed under Pickering Unit 8 main output transformer [568].

8.3.10. Underwater repair methods

Approaches to repairing normally wet or submerged concrete typically involve: applying techniques and materials qualified for underwater use, structure draining or constructing a cofferdam around the structure and removing water from inside the cofferdam. When structure drainage or a cofferdam is used, repairs can be made as in a dry environment.

Underwater work is generally classified into two broad categories for accessing the work site: diving or using a remotely operated vehicle. Divers have traditionally been used for balance of plant/conventional areas. However, in recent years, work in more hazardous areas, such as SFPs, has been documented [569, 570]. Remotely operated vehicles are preferred in these cases, where practical.

Specialty coatings, concrete mixtures and/or specialty grouts are available for underwater applications and repair guides specific to underwater use are available, such as ACI 546.2 [523] or Section 14 of IS456 [571]. Repairing intake structures and canals generally involves sealing cracks and spalls through the use of epoxies, grouts and concrete patch materials such as dry pack. The type of repair depends on the location and size of the crack or spall.

8.4. PARTS MANAGEMENT

For concrete structures, parts management (including spare parts and obsolescence) is typically only applicable to non-metallic liners, coatings and other replaceable components, such as joint sealants, waterstops, airlock seals and gaskets, among other things. This is also applicable to materials for concrete repair.

Materials or components may become obsolete when a product is discontinued, a manufacturer or supplier goes out of business, or applicable standards change requiring a new product/material to be qualified to more stringent requirements. Refer to table 1 of IAEA Safety Guide NS-G-2.12 [1] for different types of obsolescence.

To avoid interference with operations, it is important to ensure qualified materials and/or components exist and are readily available when needed for repair or replacement.

Although, it may be prudent to keep spare parts and materials available at all times, it may not be economical to do so because some parts and materials have a relatively short shelf life (e.g. elastomers, coatings and some repair materials). Facility purchasing and stocking procedures must account for spare part shelf life.

Condition assessment (see Section 5.2.2) enables identifying obsolescence, determining needs and estimating the timing for availability of spare parts and materials.

8.5. REPLACEMENT

Concrete structure replacement is normally only used when repairs would be ineffective. With the exception of components, such as ungrouted tendons, joint seals, liners and coatings and waterstops, concrete structures are not usually fully replaceable. Occasionally, some concrete structural components can be replaced.

8.6. MAINTENANCE HISTORY

Similar to inspection and monitoring records (see Section 7.5), keeping good maintenance records is an important element of an AMP. They help document where repairs have been made, identify areas of concern, provide OPEX for others and help diagnose problem areas in the future. Repaired areas are candidates for additional surveillance as part of the formal AMP to ensure the repair’s long term effectiveness.

9. SUMMARY AND CONCLUSIONS

This publication provides a guide to the set-up and implementation of an NPP concrete structure AMP. Concrete structures play a large role in both safety related systems (such as containment) and other balance of plant, economically significant, NPP processes. Both generation II and generation III/III+ designs are covered.

AMPs and methodologies have been developed for NPPs and are generally well established. Programme structures for concrete and civil structures are similar to other plant component programmes, such as electrical or mechanical components. Common IAEA Safety Guide information is available in Ref. [1].

Degradation mechanisms and results are described in detail and characteristics of a wide range of concrete inspection, monitoring and repair techniques are presented. However, no single set can cover all requirements or circumstances and techniques continue to be refined and developed.

Cracking has been the most commonly observed form of degradation. Degradation factors of primary concern are rebar corrosion due to concrete carbonation or presence of chloride ions, excessive loss of prestressing force, excessive containment leakage and leaching due to percolation of fluids through concrete.

Some specific issues that have been raised in this publication are:

(a) Increasing requirements on concrete structures beyond what was originally envisaged owing to plant life extensions, security and evolving safety requirements;
(b) Addressing environmental stressors that increase the potential for impact on functionality and durability of structures as they age;
(c) The need for routine re-evaluation of OPEX and research results related to concrete ageing and adjustment of AMPs, as necessary;
(d) The need for re-evaluation of actual environmental operating conditions (e.g. radiation, temperature and humidity) and structure behaviour against design assumptions;
(e) The need for strict quality control during the construction phase of new NPPs to ensure concrete quality for long term reliability;
(f) The need for new construction to consider provisions for monitoring (e.g. using embedded instruments) and inspecting (i.e. accessibility considerations) concrete structures, incorporating long term durability requirements in the design, as well as other lessons learned.

Concrete ageing has been identified as one of the most important issues in relation to plant life management and life extension. As such, this publication will serve to enlighten manufacturers, utilities and new reactor designers as to how concrete structures may be managed for long life operation and what must be considered to ensure that they are reliable throughout their service life.

Appendix I

IAEA PROGRAMME ON SAFETY ASPECTS OF NPP AGEING

To assist Member States in understanding ageing of SSCs important to safety and in effective ageing management of these SSCs, the IAEA in 1989 initiated a project related to safety aspects of NPP ageing. This project integrated information on evaluation and management of safety aspects of NPP ageing that had been generated by Member States into a common knowledge base, and derived guidance and assisted Member States in the application of this guidance. Results of this work are documented in Refs [10, 214, 314, 572–574], and a focus on ageing remains to this day.

Some important activities resulted from these efforts and ongoing IAEA work, as it relates to concrete and containment structures, is described in the following paragraphs.

In 1989, pilot studies on management of ageing NPP components were initiated to assist Member States in the application of ageing management methodology. Four safety significant components were selected for the studies [10]. These four components represented different safety functions and materials susceptible to different types of ageing degradation, including:

(a) Primary nozzles of reactor pressure vessels;
(b) Motor operated valves;
(c) Concrete containment buildings (CCB);
(d) Instrumentation and control (I&C) cables.

Phase 1 studies were completed via technical committee meetings held in 1990 and 1991 and consisted of paper assessments of the current state of knowledge on age related degradation, its detection and mitigation and recommendations for Phase 2 studies. Separate coordinated research projects were set-up for each of the above four components to implement the Phase 2 pilot studies. The objective of each pilot study was to identify dominant ageing mechanisms and to develop effective strategies for managing ageing effects caused by these mechanisms.

In 1991, a safety practices publication, Data Collection and Record Keeping for the Management of Nuclear Power Plant Ageing [314] was issued. This publication provides information on data requirements and a system for data collection and record keeping. General data needs are grouped into three categories (baseline, operating history and maintenance history data) and are illustrated by several examples of component specific data requirements. Actual record keeping systems, including an advanced system, are also described.

In 1992, methodology for the management of ageing of NPP components important to safety was documented [573]. Managing physical ageing of NPP components important to safety requires predicting and/or detecting when a plant component will have degraded to the point that the required safety margins are threatened (taking appropriate corrective or mitigative actions).

The methodology for management of ageing of NPP components important to safety is documented in Ref. [573] and consists of three basic steps:

(1) Selecting, from a safety perspective, plant components in which ageing needs to be evaluated by assessing effects of ageing degradation on the ability of components to perform design functions, and crediting existing programmes and activities that manage ageing effectively.
(2) Performing ageing management studies for selected components to determine appropriate ageing management actions. The two phased method reviews current understanding, monitoring and mitigation of component ageing, and identifies or develops effective and practical technology and practices for monitoring and mitigation.
(3) Using results of ageing management studies to take appropriate management actions (i.e. improving existing operations and maintenance practices and design) and to improve relevant codes, standards and regulatory requirements.

In 1998, IAEA-TECDOC-1025 [2] (the precursor to this publication) was issued to document results of the Phase 2 study related to ageing management of concrete containment buildings.

The objectives for the coordinated research project on CCBs were to:

— Produce a summary of current ageing management practices and experiences for CCSs;
— Compile a state of the art report on concrete repair techniques and materials specifically applicable to nuclear containment structures;
— Develop crack mapping and acceptance/repair guidelines applicable to nuclear containment structures;
— Develop a set of practical condition indicators and associated guidelines for monitoring concrete containment ageing.

The coordinated research projects investigated CCBs with the underlying objective of making use of research and engineering expertise of project participants in order to develop a technical basis for a practical AMP for CCBs. The project work, therefore, focused on compiling and evaluating information on age related degradation of CCBs. In addition, the project evaluated current ageing management methods and practices aimed at documenting CCB ageing and identifying significant ageing mechanisms and effective methods for timely detection and mitigation of age related degradation. More detail on the coordinated research project and its results is included in Appendix VI of this publication.

In 1999, the IAEA publication, AMAT Guidelines, Reference Document for the IAEA Ageing Management Assessment Teams (AMATs) [575] was issued. This publication was prepared to document guidelines for IAEA missions that provide assessments of AMPs at NPPs. This service remains available from the IAEA.

In 2000, IAEA-TECDOC-1181 [576], Assessment and Management of Ageing of Nuclear Power Plant Components Important to Safety: Metal Components of BWR Containment Systems, was issued. This publication covered ageing management of the steel components of BWR pressure-suppression type containments, and thus provided information that was not within the scope of the previously issued IAEA-TECDOC-1025 [1], that had focussed on concrete only.

In 2002, the IAEA-TRS-17 Guidebook on Non-destructive Testing of Concrete Structures [257] was issued. This publication was prepared separately from the coordinated research project effort described above. It evolved from a desire to provide a guide to experts delivering NDT training under regional cooperation agreements, and to have a standard curriculum for training. It contains more detail regarding certain NDT techniques than is covered in this publication's Section 7.

In 2002, the publication, IAEA Guidance on Ageing Management for Nuclear Power Plants [577] was also issued. This CD-ROM/non-serial publication was prepared to consolidate all IAEA publications related to ageing management at the time in one easy to access location. It remains available in a web accessible form.

In 2005, the publication, Plant Life Management for Long Term Operation of Light Water Reactors, IAEA-TRS-448 [578] was issued. This documented guidelines for plant life management (PLiM) activities associated with long term operation of light water reactors.

In 2007 and 2008, IAEA publications were issued, including, Safety Aspects of Long Term Operation of Water Moderated Reactors [579], Safe Long Term Operation of Nuclear Power Plants [70], and a first edition of Safe Long Term Operation (SALTO) Guidelines [580]. These dealt with ageing management aspects associated with plant life extensions and long term operation of NPPs. Concrete ageing management was identified as an important issue. The SALTO mission service remains available from the IAEA (an updated edition of SALTO peer review guidelines [581] was issued in 2014).

In 2006, the publication Nuclear Power Plant Life Management Processes: Guidelines and Practices for Heavy Water Reactors [582] was issued. This publication includes references to CSA standards related to CCSs and other PLiM related activities at PHWRs.

In 2009, the NS-G-2.12 [1] was issued, along with the associated Safety Report Series No. 62, Proactive Management of Ageing for Nuclear Power Plants [583].

In 2011, 2012 and 2013, the IGALL project undertook to document proven ageing management plans for various reactor types. IGALL working group No. 3 dealt with concrete structures. IGALL AMPS and TLAAS have been published on the IAEA web site [584], a summary final report IAEA-TECDOC-1736 [585] was issued in 2014.

Appendix II

CONTAINMENT DETAILED CONCRETE DESCRIPTIONS BY REACTOR TYPE

II.1. GENERATION II CONTAINMENT DESIGNS

II.1.1. Pressurized water reactors

PWR concrete containments may be fabricated from reinforced concrete, prestressed concrete or a combination of the two. PWR reinforced CCSs generally consist of a vertical cylindrical wall, dome and a concrete basemat. In prestressed containments, the vertical cylinder and dome are prestressed. The dome can be hemispherical, torispherical or ellipsoidal. The basemat may consist of a simple mat foundation on fill, natural cut or bedrock, or may be a pile/pile cap arrangement. Most of the plants have used the simple mat on fill or bedrock design.

Containment encloses the entire primary circuit including the reactor pressure vessel, steam generators and piping. Single or double wall construction may be used and the inside surface may or may not be lined to provide a leaktight barrier.

Three general categories of PWR containments exist (see Fig. 166):

(a) Large dry containments designed to have sufficient capacity to contain the entire volume and energy of primary coolant fluid, which would be released as steam and water in the event of a LOCA;
(b) Ice condenser containments, which channel steam resulting from a LOCA through ice beds to reduce pressure build up below containment volume and pressure requirements;
(c) Subatmospheric containments fabricated from reinforced concrete and designed such that a slightly negative pressure is maintained within containment to reduce volume requirements.

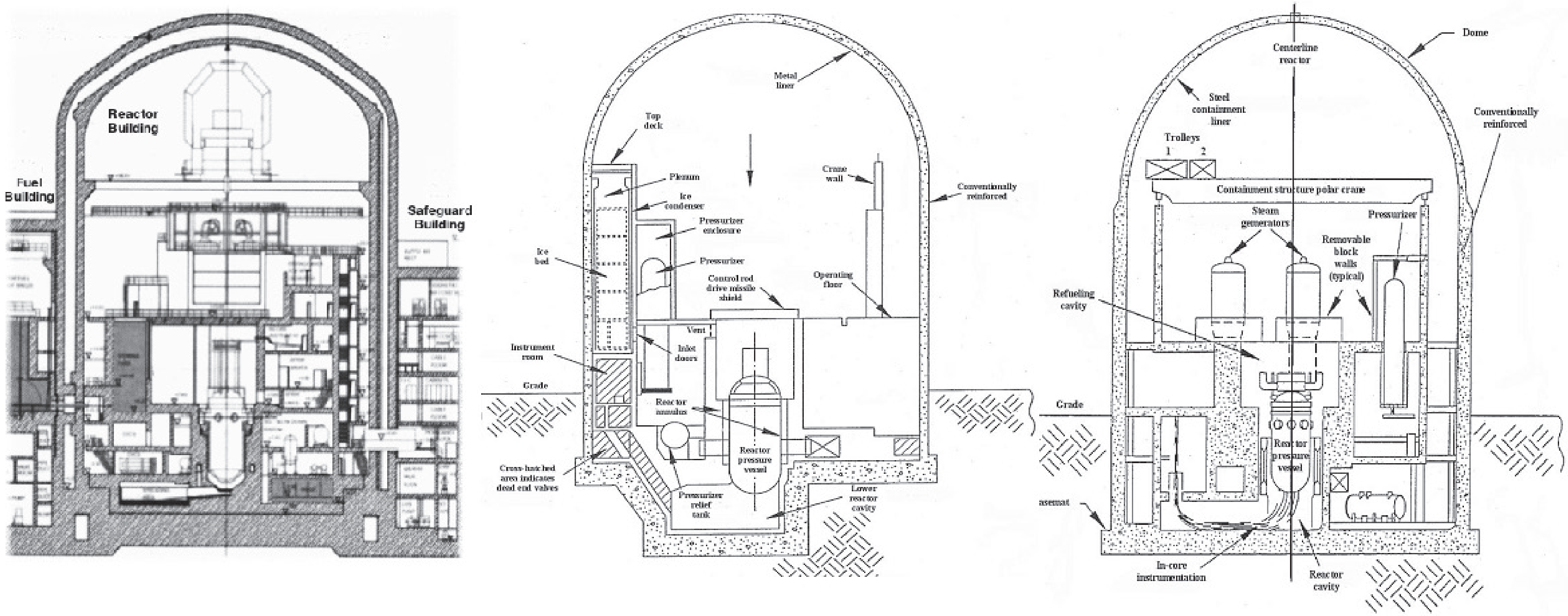
Differences between these containment designs relate to volume requirements, provisions for accident loadings/pressures and containment internal structure layout. A secondary concrete containment is used as a shield when a steel primary containment is used. Table 29 presents typical design parameters of PWR concrete containments.

PWR containment internal structures are constructed of conventionally reinforced concrete and tend to be more massive in nature than internal structures in BWRs because they typically support the reactor pressure vessel, steam generators and other large equipment and tanks. Additionally, these structures provide shielding of radiation emitted by the nuclear steam supply system. Internal structures for dry containments include the primary shield wall, secondary shield wall and operating and intermediate floors. Internal concrete structures are supported on the containment basemat.

The primary shield wall is constructed of reinforced concrete and completely surrounds the reactor vessel, providing biological shielding and reactor vessel support (in some plant designs). It is designed to withstand the thermal gradient across the wall that is generated by gamma and neutron radiation from the reactor core during normal operation.

Secondary shield walls are reinforced concrete structures surrounding the steam generator components, protecting containment and reactor coolant systems from postulated effects of a pipe rupture (e.g. jet impingement loads). These walls can also provide anchorage for major piping and may support intermediate and operating floors and the polar crane, and provide lateral restraint to the steam generator, coolant pumps, pressurizer and associated piping.

Operating and containment internal floors are constructed of reinforced concrete or steel grating supported by structural steel framing. Other internal structures include the refuelling canal (fabricated of reinforced concrete with a stainless steel liner to prevent leakage) and removable concrete shield slabs. Internal structures for ice condenser plants are similar in function and structural design to internal structures for dry containments (e.g. primary shield wall, operating floor and refuelling canal) except for a reinforced concrete divider barrier that isolates all primary pressure system pressure retaining equipment from upper containment and ice condenser features.



(a) (b) (c)

FIG. 166. Typical PWR concrete containments: (a) PWR large dry double barrier; (b) PWR ice condenser; (c) PWR subatmospheric [2].

TABLE 29. TYPICAL DESIGN PARAMETERS OF PWR CONCRETE CONTAINMENTS (*adapted from Ref.* [83])

Type	External environment	Internal environment					
		Normal operating conditions			Postulated accident conditions		
		Pressure (kPa)	Temp. (°C)	Humidity[a] (% RH)	Pressure (kPa)	Temp. (°C)	Humidity[a] (% RH)
PWR (dry)	Dome and upper walls – natural environment Lower walls – subterranean	Atmos.	<50	20–60	410–520	125–138	100
PWR (sub-atmospheric)	Dome and upper walls – natural environment Lower walls – natural environment with dewatering system (dewatered); basemat below grade	32	15.6–40.6	20–60	310	66	100
PWR (ice condenser)	Dome and upper walls – natural environment Lower walls and basemat below grade	Atmos. or slight vacuum	38[b] 50[c]	20–85	170–210	104[b] 124[c]	100

[a]Typical ranges for relative humidity listed. Some plants have different ranges.
[b]Upper compartment.
[c]Lower compartment.

PWR plants using a metallic primary containment (large dry and ice condenser designs) are contained in a reinforced concrete secondary containment or shield building. This is typically a reinforced concrete structure consisting of a vertical cylinder wall with shallow dome roof (see Fig. 167) and is often supported by the containment basemat. In addition to withstanding environmental effects, secondary containment provides radiation shielding and particulate collection and ensures that the freestanding metallic primary containment is protected from the natural environment. The ability to control radioactive releases is provided by maintaining the annular region between primary and secondary containment structures at a slightly negative pressure relative to ambient.

II.1.2. Boiling water reactors

Although the majority of BWR plants use a steel containment vessel, some units use either prestressed or reinforced concrete containment. Figure 168 shows typical General Electric (GE) type BWR containments; other designs such as the Siemens-KWU Bauline ‘69’ and ‘72’ and the ABB Atom Types I and II are shown and described more fully in IAEA-TECDOC-1181 [576].

Where a concrete containment is used, leaktightness is provided by a liner (typically metallic) attached to the concrete surface by studs or structural steel members. Exposed surfaces of the liner are usually painted for corrosion protection and to facilitate decontamination. A portion of the liner toward the bottom of the containment structure and over the basemat is typically embedded in concrete to protect it from damage and abrasion due to corrosive fluids or impact. A seal (waterstop) is typically provided at the interface around the containment circumference where the vertical portion of the liner becomes embedded in concrete. In addition, the containment vessel can provide structural support for the nuclear steam supply system and other internal equipment.

The containment foundation, typically a basemat, provides primary support and load transfer to the ground below.

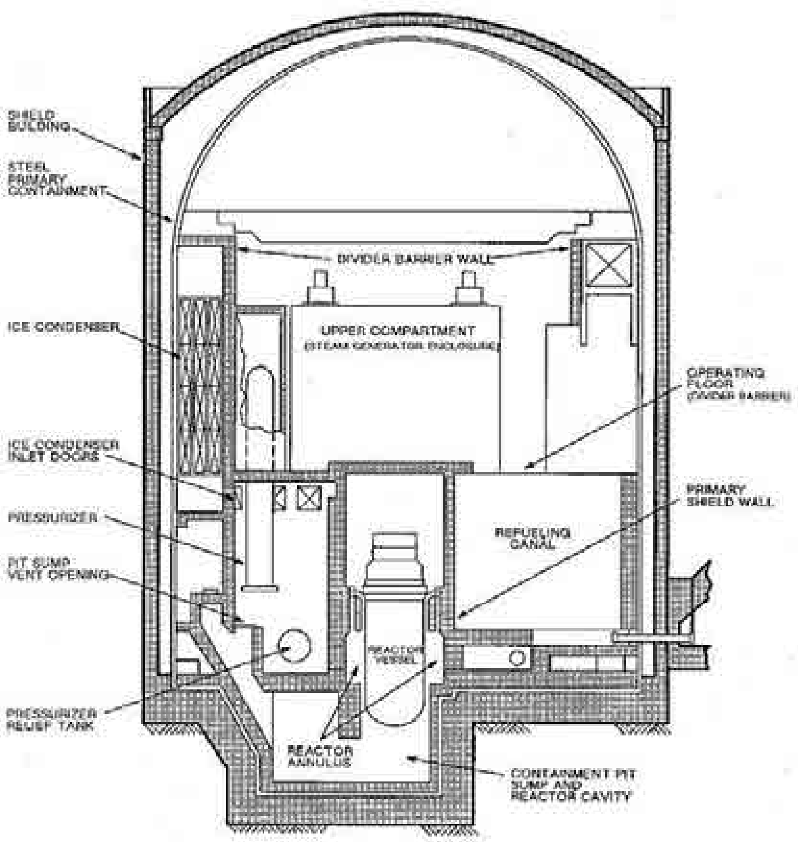


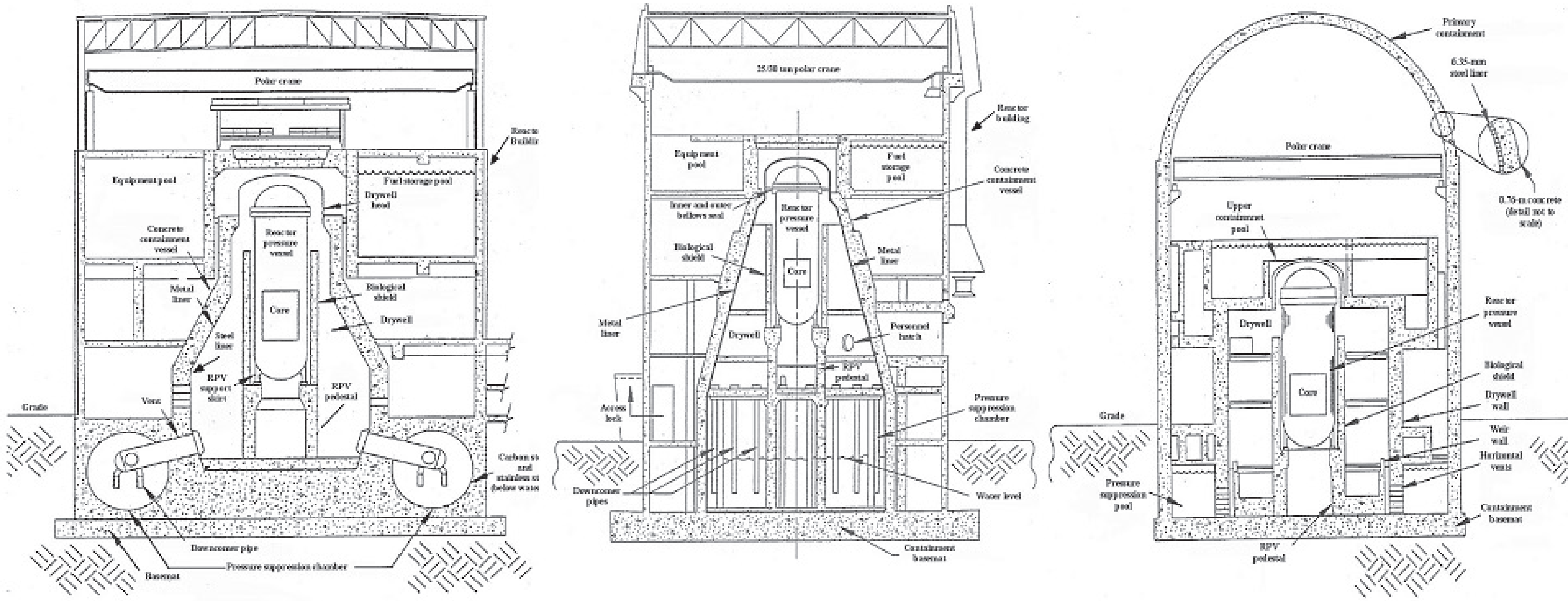
FIG. 167. PWR ice condenser steel containment with concrete shield building [22].

BWR containments are all designed with a pressure suppression system and are divided into two main compartments: wet-well and dry-well. After a LOCA, air and steam in the dry-well is forced through a number of downcomers to a pool in the wet-well where the steam condenses. Water spray systems are provided to reduce containment volume requirements. Auxiliary systems are usually housed in a building surrounding the containment structure, which is kept at a slight subatmospheric pressure, thereby serving as a secondary containment.

Table 30 presents an example of typical design parameters of BWR concrete containments.

Each of the three GE BWR primary plant types (Mark I, Mark II and Mark III) incorporates a number of reinforced concrete containment internal structures. The primary shield wall is the only containment internal structure for Mark I plants and consists of a cylindrical wall surrounding the reactor vessel. It is supported on the reactor pedestal, functions to attenuate radiation from the reactor core and is subjected to a thermal gradient. The wall is typically a composite structure, usually fabricated from concrete that may have a steel liner on both sides to act as the primary structural support. A Mark II containment's primary shield wall also encloses the suppression pool, which is covered by a diaphragm floor and several intermediate floors. The diaphragm floor is constructed of reinforced concrete and is supported by wet-well walls and other supports within the suppression area. Intermediate floors are constructed of reinforced concrete, structural steel or a combination of the two. Mark III plant internal structures, in addition to the primary shield wall and intermediate floors, which are similar to those of the Mark II plants, include:

— Drywell;
— Weir wall;
— Refuelling pool;
— Operating floor;
— Floors located inside the drywell;
— Floors in the annulus between drywell and containment.



(a) (b) (c)

FIG. 168. Typical GE BWR concrete containments: (a) Mark I; (b) Mark II; (c) Mark III [2].

TABLE 30. TYPICAL DESIGN PARAMETERS OF GE BWR CONCRETE CONTAINMENTS (*adapted from Ref.* [83])

Type	External environment	Internal environment						
		Normal operating conditions			Postulated accident conditions			
		Pressure (kPa)	Temp (˚C)	Humidity[a] (% RH)	Pressure (kPa)	Temp (˚C)	Humidity[a] (% RH)	Comments
BWR MARK I	Enclosed in RB Basemat below grade	atm	<50	40–85	427	149	100	Suppression chamber design temp. = 105˚C
BWR MARK II	Enclosed in RB Basemat below grade	atm	<57	40–85	380–480	137–171	100	Suppression chamber design temp. = 105˚C
BWR MARK III	Dome and upper walls – natural environment Lower walls and mat – subterranean	atm	<57	40–85	140–210	60–85	100	Drywell accident temp. ~ 166˚C

[a] Typical ranges for relative humidity listed. Some plants have different ranges.

The drywell encloses the reactor vessel and reactor coolant system and functions as a pressure boundary and radiation shield. The drywell is a cylindrical reinforced concrete structure with flat roof slab. The structure's venting portion is a composite consisting of two concentric steel cylinders with a concrete filled annulus between the cylinders, which forms the inner and outer surfaces of the drywell. The upper portion of drywell is reinforced concrete. The reinforced concrete weir wall forms the inner boundary of the suppression pool and completely surrounds the lower portion of the reactor coolant system. The inner surface of the weir wall is lined with stainless steel plate to provide leaktightness. Upper containment refuelling pool walls are reinforced concrete with a stainless steel inside surface liner to ensure leaktightness. Operating floors usually consist of a combination of structural steel framing and reinforced concrete and may be supported by containment walls and the refuelling pool. Other internal structures include floors, constructed of reinforced concrete and/or structural steel, located in the drywell and in the annulus between the drywell and containment.

Of BWR plants that use steel primary containment, except for pre-Mark plant types, a reinforced concrete structure is generally used as a RB or as secondary containment and provides primary containment support and shielding functions. Mark I and Mark II RBs completely surround containment and house most safety related equipment. RBs are safety related because they:

— Provide radiation shielding;
— Provide resistance to environmental/operational loads;
— House safety related mechanical equipment, spent fuel and primary metal containment.

The RB's upper elevation contains the refuelling and fuel storage area and is typically a multilevel structure composed of reinforced concrete that completely encloses the drywell and pressure suppression pool structures. It is supported on the containment's reinforced concrete basemat. Some RBs are reinforced concrete structures up to and including the roof, while others are reinforced concrete construction only up to the refuelling floor elevation. The remaining upper portion is a steel framed structure with metal siding and roof panels. Exterior load-bearing walls are reinforced concrete shear walls designed to resist lateral loads. Interior walls are either reinforced concrete or concrete masonry block. Floors are constructed of reinforced concrete supported by a steel beam and column framing system. Walls and floors provide radiation shielding for compartments containing radioactive components.

RBs are maintained at a slight negative pressure, relative to ambient, to facilitate collecting and treating any airborne radioactive material that might escape during accidents.

Mark III RBs consist of a reinforced concrete structure that completely encloses a freestanding steel primary containment vessel. The design provides for:

- Biological shielding;
- Controlled release of the annulus atmosphere under accident conditions (secondary containment function);
- Environmental protection of the steel primary containment.

In some Mark III containment designs, the secondary containment function is provided in part by a steel frame with metal siding and roof decking that is sealed because it forms part of the secondary containment ventilation barrier. The ability to control radioactive releases is provided by maintaining the annular region between the secondary containment and the primary containment structure at a slightly negative pressure, relative to ambient.

Other BWR structures include:

- The auxiliary building;
- The control room/control building;
- The diesel generator building;
- The spent and new fuel storage facility;
- Tanks;
- The intake structure;
- The turbine building;
- The radwaste processing facility;
- Tunnels;
- The switchgear room;
- Unit vent stacks.

The auxiliary building is generally a multilevel structure composed of reinforced concrete, masonry block walls and structural steel construction, and is supported on a reinforced concrete basemat. Reinforced concrete exterior walls are designed both as shear walls to resist lateral loads and as load-bearing walls. Floors are constructed of reinforced concrete, either self-supporting or supported on steel beams and column framing systems using steel or composite interior columns and reinforced concrete bearing walls. Interior walls are made of both cast-in-place reinforced concrete and concrete masonry block and provide the required radiation shielding and equipment separation. Floors are designed for vertical loads and as diaphragms to transmit lateral loads to the shear walls. The roof is constructed of reinforced concrete and is designed to withstand postulated missile and tornado wind loads. The control room/control building is typically constructed of structural steel framing with reinforced concrete bearing and shear walls. Floor slabs are supported by steel beams and serve as diaphragms to transmit lateral loads to shear walls. Interior walls are concrete masonry with light partitions. Either reinforced concrete or reinforced masonry block is used for exterior walls of the control building.

The diesel generator building is a reinforced concrete structure. Bearing walls are reinforced concrete and are also designed as shear walls to resist lateral loads. Compartmentalization is by reinforced concrete walls. Roof and floors are of reinforced concrete supported by steel beams. The fuel storage facility is a multistory reinforced concrete structure and is an integral part of the RB for most Mark I and II plants, while for Mark III plants and PWRs, the fuel storage facility is a separate structure. The fuel storage facility and pools for reactor internals storage typically have a four wall with bottom slab configuration. Fuel storage facility bearing walls are reinforced concrete and are designed as shear walls. The floors and roof are reinforced concrete supported by steel beams. Some plants have exterior walls of structural steel framing with concrete panels, concrete block or metal siding. Roofs of these buildings are of composite design.

II.1.3. Pressurized heavy water reactors

Of heavy water reactor designs, CANDU reactors are the most prevalent. These plants are constructed in single or multiunit configurations. CANDU multiunit stations employ a common vacuum building and pressure relief duct connected to up to eight RBs that form the main components of the containment system. During normal operation, the vacuum building and pressure relief duct are isolated from the RBs, however, with sufficient increase in pressure, due to an accident in any of the RBs, panels will rupture and the hot gases and steam causing the rising pressure will fill and pressurize the pressure relief duct. Pressure relief valves then open to release the hot gases and steam into the vacuum building, where a pressure actuated water dousing system condenses the steam. RBs for CANDU multiunit stations come in two basic designs:

(a) Domed cylindrical structures with an unlined single exterior wall, independent internal frame and calandria vault;
(b) Thick walled cube shaped structures with steel liner and post-tensioned roof beams.

Single unit RBs are similar to the domed cylindrical structure design listed in (a) above, except they are fully post-tensioned (basemat, wall and upper dome) and have a non-metallic liner. A reinforced concrete lower dome contains the dousing water reservoir (contained in the vacuum building in the multiunit configuration).

Multiunit station vacuum buildings and pressure relief ducts are reinforced concrete structures with a post-tensioned ring girder to stiffen the top of the vacuum building cylinder perimeter wall. Vacuum buildings are of approximately a 50 m diameter base area and of similar height. The perimeter wall also provides shielding. In later vacuum buildings, design changes were implemented to satisfy requirements to contain higher positive pressures. Most notable are the roof dome’s monolithic connection to the peripheral ring girder, circumferential and vertical prestressing of the wall and ring girder, prestressing of the roof dome and lining of the floor slab and lower six metres of the perimeter wall with carbon steel.

Figure 169 provides examples of CANDU vacuum and RB cross-sections.

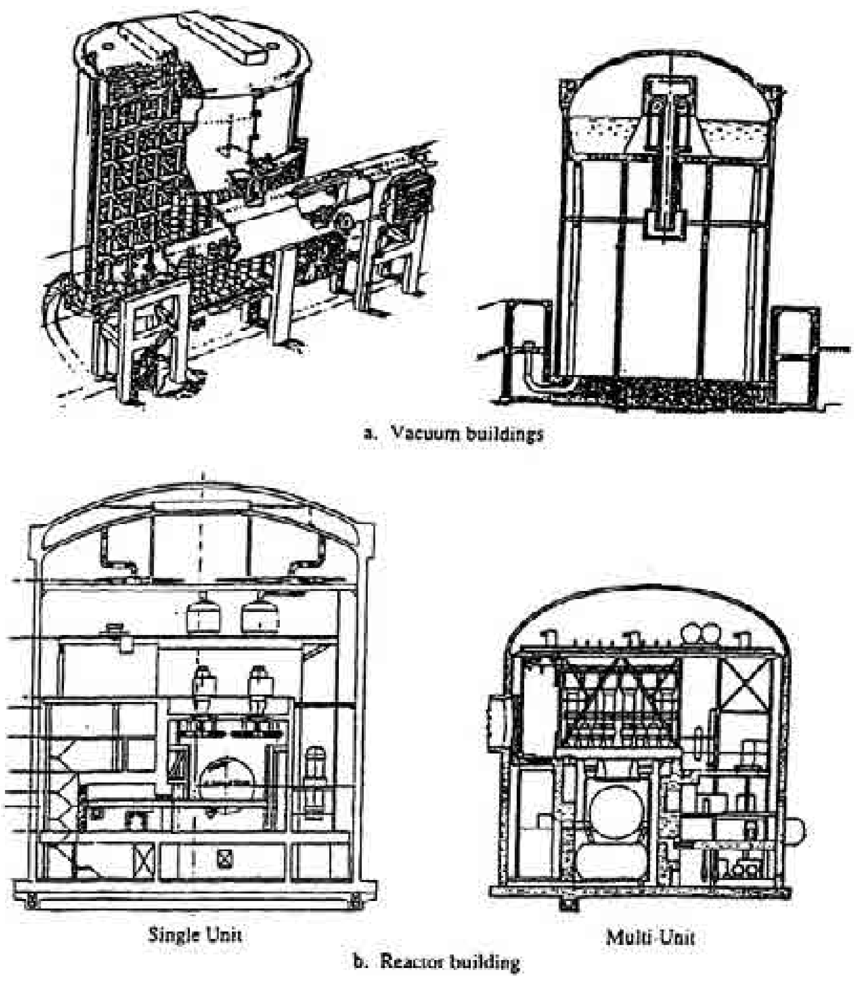


FIG. 169. Typical CANDU concrete containment [2].

Containment systems of Indian PHWRs have undergone progressive improvement. The primary containment of current Indian PHWR designs consists of a prestressed concrete perimeter wall topped by a prestressed concrete dome. The containment building is made of concrete with epoxy/vinyl coating for leaktightness and has a passive suppression system. Double containment is extended to cover the entire RB (except the basemat) with the outer secondary containment envelope consisting of a reinforced concrete cylindrical wall topped by a reinforced concrete dome [586].

II.1.4. Gas cooled reactors

Primary containment for many gas cooled reactors (e.g. Magnox stations, advanced gas cooled reactors (AGR) and high temperature gas cooled reactors) is provided by a prestressed concrete pressure vessel (see Fig. 170). In most cases, an integral design has been adopted, with the pressure circuit, reactor core and boiler all contained within the prestressed concrete pressure vessel. The principal structural functions of these vessels are to support the reactor and boiler and to provide support to the steel liner that creates the pressure enclosure for the gas circuit, which transfers heat from the reactor to the boiler. The liner is provided with insulation and is cooled by recirculated water, thus, it also provides a thermal barrier between the hot gas and concrete. Mass concrete temperatures are generally kept below 50°C, with some isolated hotter areas. Although the prestressed concrete pressure vessel also provides biological shielding, the thicknesses of the vessel wall and end caps (typically 4–5 m) are determined by the pressure boundary support function.

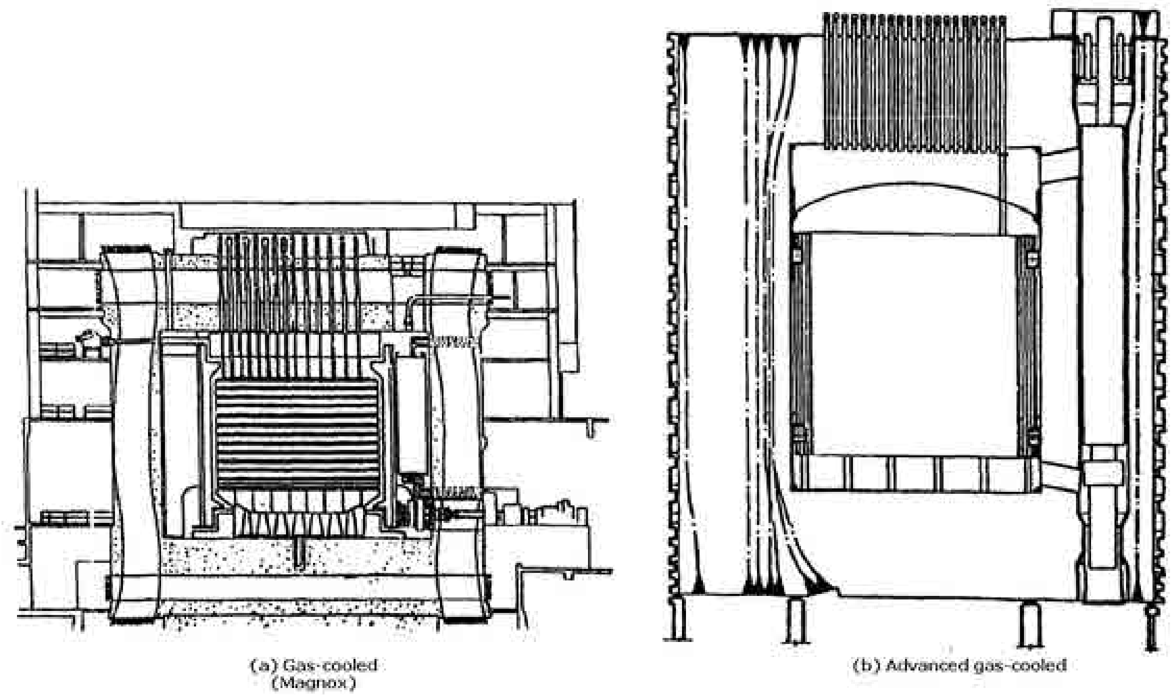


FIG. 170. Typical gas cooled reactor prestressed concrete pressure vessel (adapted from Ref. [1]).

II.1.5. Russian nuclear power plant designs

Russian designed NPPs were built in a series of four generations (I, II, III, III+) from the 1960s through to present day (refer to Table 31).

TABLE 31. RUSSIAN FEDERATION NPP PLANT GENERATIONS

Generation	Plants	Normative documents
I	Bilibino NPP (4 units) Kola NPP (2 units) Kurskaja NPP (2 units) Leningradskaja NPP (2 units) Novovoronezskaja NPP (2 units)	None
II	Balakovo NPP (4 units) Belojarskaja NPP (1 unit) Kalininskaja NPP (4 units) Kola NPP (2 units) Kurskaja NPP (2 units) Leningradskaja NPP (2 units) Novovoronezskaja NPP (1 unit) Smolenskaja NPP (2 units)	OPB-73, OPB-82, PBJA-09-74
III	Smolenskaja NPP (1 unit)	
III+	Novovoronezskaja NPP (2 units under construction) Leningradskaja NPP (4 units) Belojarskaja 2 NPP (1 unit)	

II.1.5.1. High-power channel-type reactor

High-power channel-type reactors (RBMK) localize safety systems using a dense solid box system. Primary circuit equipment and pipelines are partially located within these boxes. The RBMK-1000 main building is shown in Fig. 171.

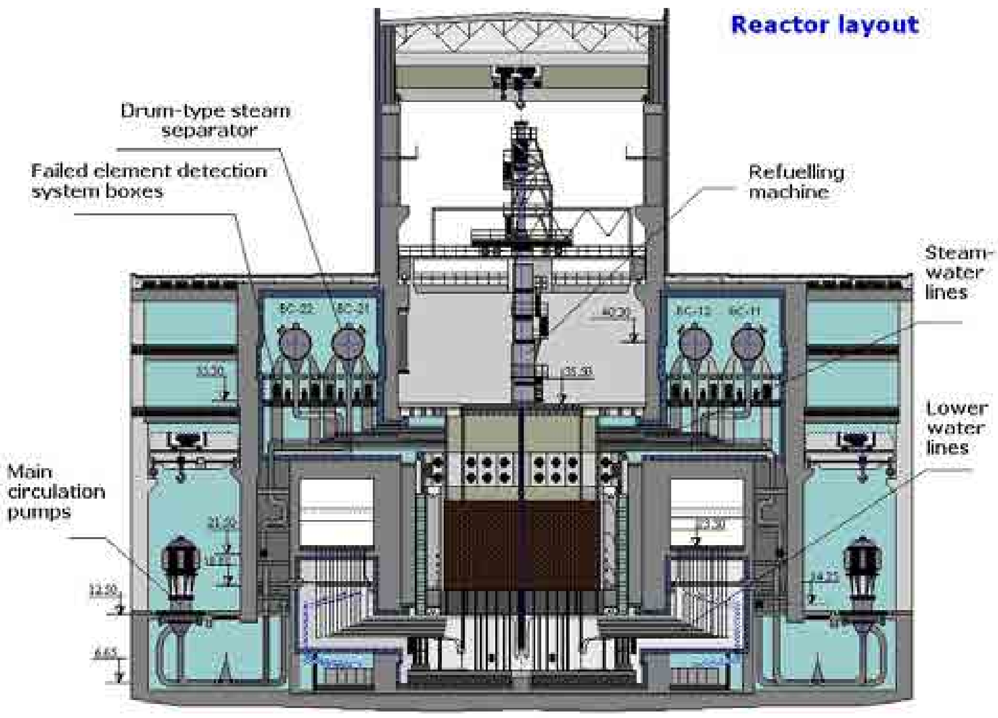


FIG. 171. RBMK-1000 arrangement (adapted from Ref. [587]).

II.1.5.2. Water cooled, water moderated power reactors

Water cooled, water moderated power reactors (WWERs) are PWRs of Russian design (see Figs 172–174). WWERs all feature some level of containment, but the degree of provision varies widely between older and newer designs. Currently, WWER-440 and WWER-1000 NPPs are operated in the Russian Federation. A description of each follows.

Older WWER-440 type reactors (WWER-213 and WWER-230) generally have a confinement structure that surrounds the pressure vessel and consists of confinement concrete over a reinforced basemat, which forms a sealed set of linked compartments designed to confine radioactive material in the event of an accident. Cold water spray systems are provided to minimize overpressure in the event of a LOCA.

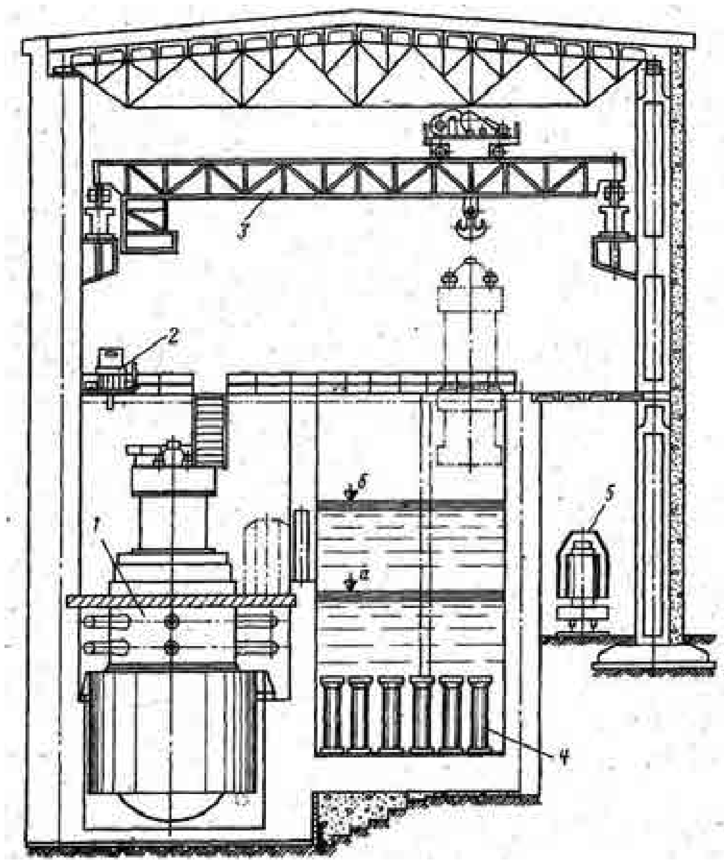


FIG. 172. WWER-440 arrangement of power unit: 1-reactor, 2-fuel handling machine, 3-reactor hall traveling crane, 4-cover for worked out TBC, 5-transportation container [588].

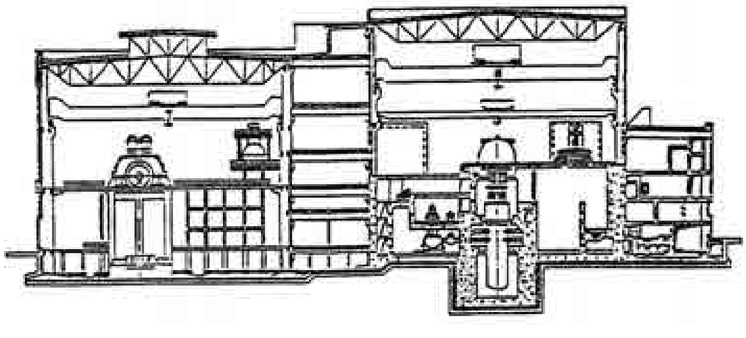


FIG. 173. WWER-440, version V-230 [1].

Newer WWER-1000 type reactors use a single barrier large dry prestressed concrete containment of similar design to that used by western PWR plants. Non-grouted tendons in polyethylene ducts are arranged helically in the barrel and orthogonally in the dome. Leaktightness is provided by a coated metal liner that also serves as formwork during concrete placement. A view of a power unit is shown in Fig. 174, and containment in Fig. 175.

The containment structure is a prestressed reinforced concrete cylinder with an inner diameter of 45 m and a height of 54 m and is covered by a low pitched spherical dome with inner radius of 35 m. Wall thicknesses are: cylinder 1.2 m and dome 1.1 m.

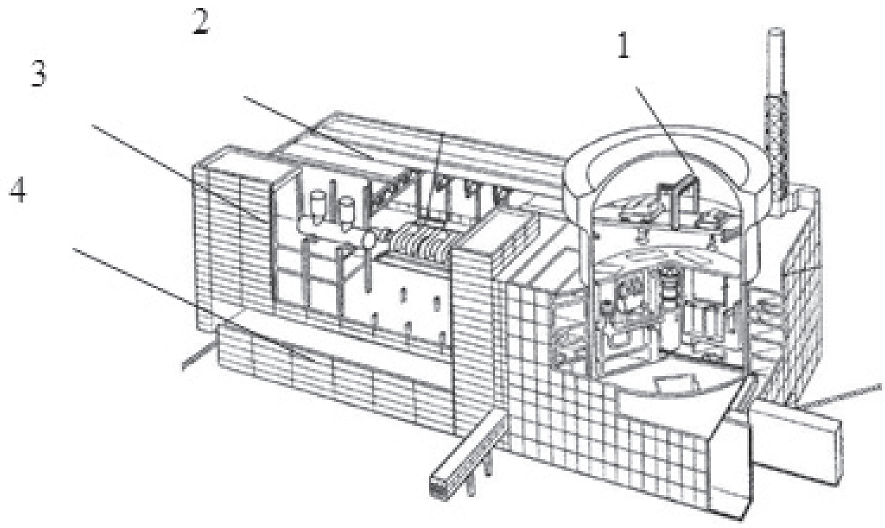


FIG. 174. WWER-1000: 1-containment, 2-machine hall, 3-deaerator room, 4-rack for electric devices [19].

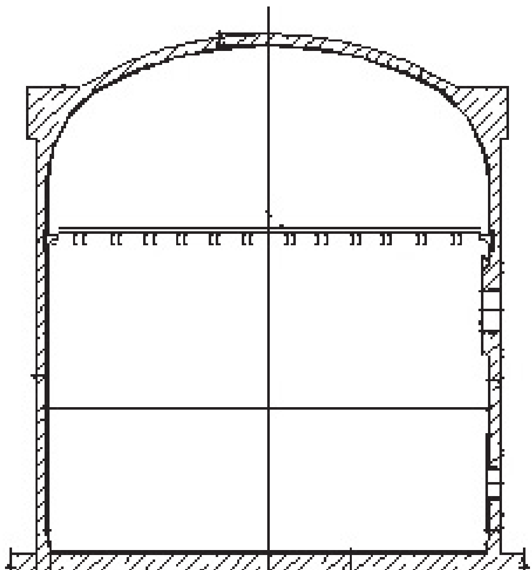


FIG. 175. WWER-1000 containment [19].

The prestressing system SPN-1000 is used, consisting of double looping high strength bundled bars produced by continuous coiling of 5 mm diameter wires. The design stretching force value of the strands in the cylinder and dome is 1000 ton-force. Containment leaktightness is provided by 8 mm steel cladding on the inner surface.

II.2. GENERATION III AND III+ CONTAINMENT DESIGNS

Examples of Generation III and III+ plants include:

- — European Pressurized Reactor/Evolutionary Power Reactor (EPR);
- — Pebble Bed Modular Reactor (PBMR);
- — Advanced Boiling Water Reactor (ABWR);
- — AP-600/AP-1000;
- — System 80^{+};
- — Economic Simplified BWR (ESBWR);
- — CANDU EC6;
- — ACR-700/ACR-1000;
- — AFCR, Gas Turbine Modular Helium Reactor (GT-MHR);
- — US-APWR;
- — International Reactor Innovative and Secure (IRIS);
- — Siedenwasser Reaktor (SWR)-1000;
- — Super Safe, Small and Simple (4S);
- — mPower;
- — WWER-1200.

Table 32 provides information on design parameters for numerous proposed plants. Provided in the following is a brief description of several of these plants, including pertinent concrete structures.

TABLE 32. SELECTED DESIGN PARAMETERS FOR GENERATION III AND III+ PLANTS *(adapted from Ref.* [589])

Plant type	Nominal power		Coolant temp. (˚C)		Containment design conditions		Containment type[b]
	MW(e)	MW(t)	Inlet	Outlet	Pressure (kPa)	Temp. (˚C)	
EPR	1600	4500	295.6	328.9	427	154	PC
PBMR	165	400	500	900	—	—	RC confinement
ABWR	1385	3926	278	288	310.3	171	RC
AP-600	619	1940	279.5	315.6	508.1	148.9	Steel cylinder
AP-1000	1117	3415	280.7	321.1	406.7	148.9	Steel cylinder
SYSTEM 80+	1389	3914	291	323.9	365.4	143	Steel sphere
ESBWR	1535	4500	276.2	287.7	310	171	RC
ACR-700	731	1982	278.5[a]	325[a]	250	—	PC
ACR-1000	1165	3200	275	319	450	149	PC
GT-MHR	286	600	491	850	—	—	RC silo
US-APWR	1700	4451	289	325	469	148.9	PC
IRIS	335	1002	292	328.4	1310	200	Steel sphere

TABLE 32. SELECTED DESIGN PARAMETERS FOR GENERATION III AND III+ PLANTS *(adapted from Ref.* [589]) (cont.)

Plant type	Nominal power		Coolant temp. (˚C)		Containment design conditions		Containment type[b]
	MW(e)	MW(t)	Inlet	Outlet	Pressure (kPa)	Temp. (˚C)	
mPower	125	400	297.8	320.6	—	—	RC
4S	10	30	355	510	—	—	RC silo below grade
WWER-1200	1200	3200	298.6	329.7	500	150	PC/RC (double wall)

[a] Reactor inlet and outlet header temperatures.
[b] PC: prestressed concrete and RC: reinforced concrete.

II.2.1. Pressurized water reactors (AP1000, EPR and System 80+)

The AP-1000 nuclear reactor design is a two loop 3415 MW(t) PWR. Figure 176 presents a view of an AP-1000 plant and passive containment cooling system. The containment vessel exterior provides a surface for evaporative film cooling and works in conjunction with the natural draft airflow created by the shield building baffle and chimney arrangement to reduce containment internal pressure and temperature following a design basis accident. The AP-1000 primary containment consists of a cylindrical steel shell with ellipsoidal upper and lower heads. Primary containment encloses the nuclear steam supply system (i.e. reactor vessel, steam generators, reactor coolant pumps, pressurizer and associated connecting piping), in-containment refuelling water storage tank, core make up tanks, accumulator tanks and the refuelling canal.

The containment bottom head is embedded in concrete for corrosion protection. The containment vessel is coated with an inorganic zinc coating, except for those portions fully embedded in concrete. The inside of the vessel, below the operating floor and slightly above the operating floor, also has a phenolic topcoat. Seals are provided at concrete surfaces inside and outside the vessel so moisture is not trapped next to the vessel below the top of the concrete.

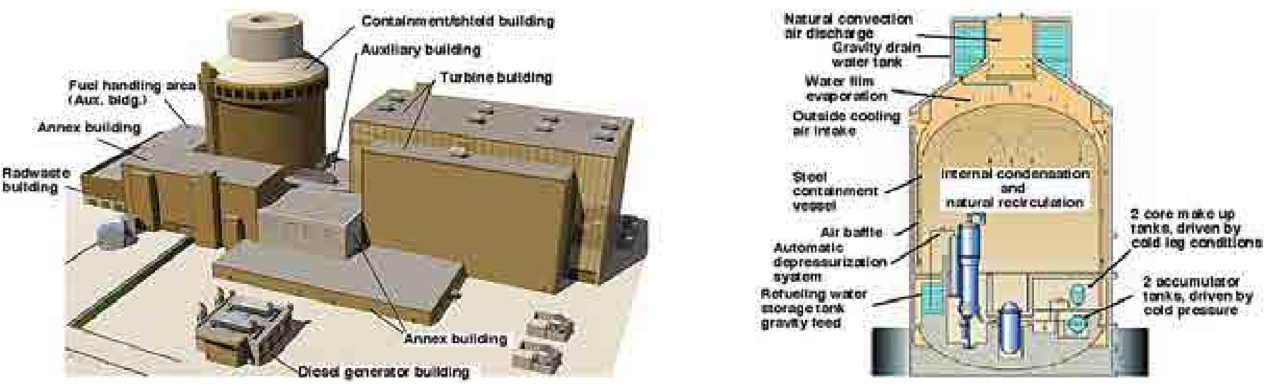


FIG. 176. General view of AP-1000 plant and passive containment cooling system [590].

The shield building:

— Is the structure and annulus area surrounding the containment vessel;
— In conjunction with containment building internal structures, provides shielding for the reactor coolant system and other radioactive systems and components within containment, during normal operation;

— Provides shielding for radioactive airborne materials that may be dispersed in containment as well as radioactive particles in water distributed throughout containment during accidents;
— Protects the containment vessel and reactor coolant systems from tornadoes and tornado produced missiles;
— Is a cylindrical, reinforced concrete structure with a conical roof that supports the water storage tank and air diffuser (chimney) of the passive cooling system;
— Shares a common basemat with the primary containment and auxiliary building and is designed as a seismic structure.

The auxiliary building:

— Provides protection and separation for safety related seismic mechanical and electrical equipment located outside containment (e.g. the main control room, safety related electrical and I&C systems, fuel handling and spent fuel handling equipment).
— Provides shielding for radioactive equipment and piping.
— Is a reinforced concrete and structural steel structure.
— Has floor slabs and structural walls that are structurally connected to the cylindrical section of the shield building.
— Can have sides that include structural modules built with welded steel structural shapes and plates. Where required, concrete, normally without reinforcing steel, is used for shielding.
— Modules include the SFP, fuel transfer canal and cask loading and wash down pits.

The main control room ceiling and instrumentation and control rooms are designed as finned floor modules. A finned floor consists of a concrete slab poured over a steel plate stiffened ceiling. Fins are rectangular plates welded perpendicular to the faceplate of the floor module. The fins are exposed to the environment of the room and enhance the heat absorbing capacity of the ceiling.

Containment internal structures consist of:

— Primary shield wall;
— Reactor cavity;
— Secondary shield walls;
— In-containment refuelling water storage tank;
— Refuelling cavity walls;
— Operating floor;
— Intermediate floors;
— Various platforms.

Containment internal structures are designed using reinforced concrete and structural steel. Walls and floors are concrete filled steel plate structural modules. Walls are supported on the massive concrete containment internal structure basemat, with a steel surface plate extending down to the concrete floor on each side of the wall.

The reinforced concrete shield building, including the passive cooling water storage tank, steel containment vessel, modular constructed containment internal structures and reinforced concrete auxiliary building are supported on a common cast-in-place, reinforced concrete basemat and form the nuclear island. The containment internal structure's basemat is the reinforced concrete structure filling the containment vessel bottom head.

Reference [590] provides more detail on the AP1000.

Evolutionary PWR plant configuration is shown in Fig. 177. The nuclear island consists of the RB, safeguard buildings and the fuel building, all of which have a common basemat. The RB consists of a cylindrical reinforced concrete outer shield building, a cylindrical post-tensioned concrete inner containment building with steel liner and an annular space between the shield building and inner containment building. The shield building protects the inner containment building from external hazards. The nuclear island common basemat is a heavily reinforced concrete slab that supports the seismically qualified structures. The specific basemat configuration may differ according to site soil type.

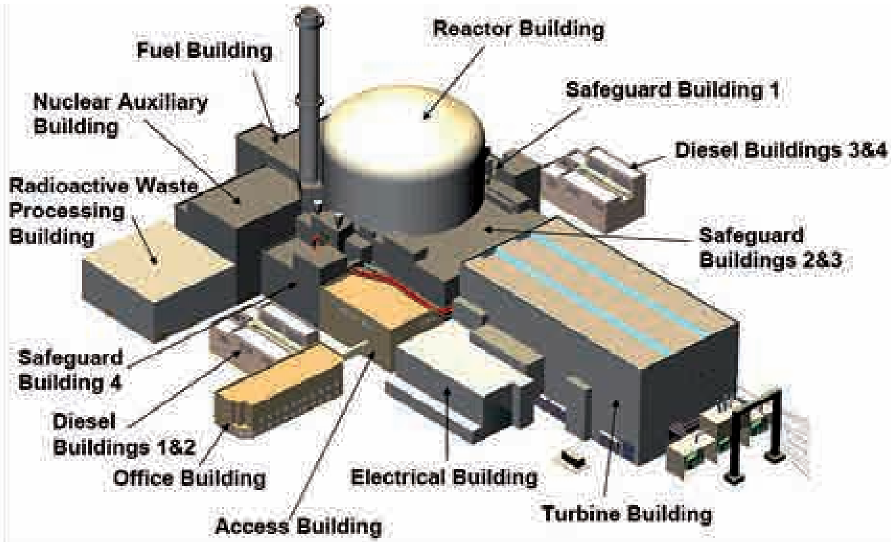


FIG. 177. EPR plant configuration [591].

The reactor containment building is a post-tensioned concrete pressure vessel with a steel liner covering the entire inner surface of the building, including the basemat. An RB cross-section showing the reactor containment and shield buildings is shown in Fig. 178. Wall and dome containment building shells are post-tensioned with hoop, vertical and dome tendons, with each tendon consisting of 54 seven-wire strands that are grouted after tensioning (bonded tendons). Reinforcing steel bars are provided in the containment walls and dome for crack control and strength to accommodate seismic and other loads. Three buttresses run vertically, at azimuths of 0°, 112° and 230°, and project from the outside surface of the cylindrical containment wall. These buttresses serve as anchorage locations for terminating horizontal hoop tendons. A ring girder is provided around the top perimeter of the containment wall where it transitions into the spherical dome. A tendon access gallery is provided under the circumference of the cylindrical containment wall below the nuclear island common basemat.

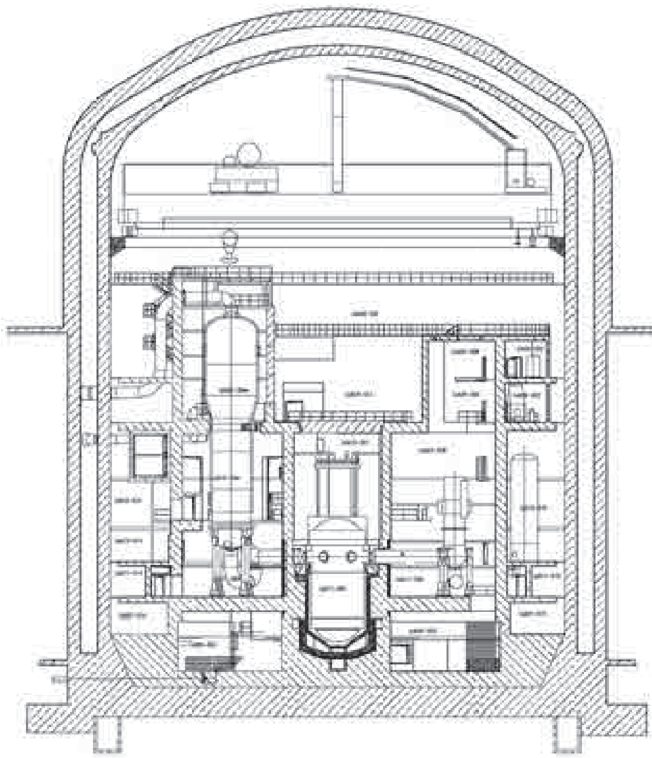


FIG. 178. EPR reactor building [591].

RB internal structures consist of reinforced concrete walls and floors, steel framing members and other concrete and steel structural elements. They provide support for components and radiation shielding for the reactor cooling system and refuelling operations. The RB internal structure’s basemat consists of reinforced concrete in the lower part of the containment above the liner plate. The reactor vessel support structure consists of a circular reinforced concrete wall extending from the top of the RB internal structure’s basemat to the reactor vessel piping steel supports. The secondary shield wall provides radiation shielding and support for components. This wall is anchored to the RB internal structure’s basemat and extends from the basemat up to the operating floor.

The reactor shield building is a heavily reinforced concrete structure consisting of a cylindrical wall and dome roof designed to withstand an aircraft impact. The shield building completely encloses the containment building to protect it from missiles resulting from external event loading (e.g. hurricanes, tornados, aircraft hazards and explosion pressure waves). The shield building serves as an additional preventive barrier to radiation or contamination release in the event of an accident. The nuclear island common basemat supports the reactor shield building.

The RB annulus is the space between the reactor shield building and containment building. The RB annulus is an area providing access for personnel to inspect the outside of the containment building and to route piping, ventilation ducts, electrical cables and other items.

EPR safety systems address the hypothetical case of a severe accident that includes core melting, with a target of restricting consequences to the immediate plant vicinity. As shown in Fig. 179, an EPR safeguard system is a core catcher (i.e. molten core spreading area) inside containment above the internal structure’s basemat. The core catcher is designed to contain any molten material that penetrates the bottom of the reactor pressure vessel, preventing it from attacking the basemat or reaching outside of the building.

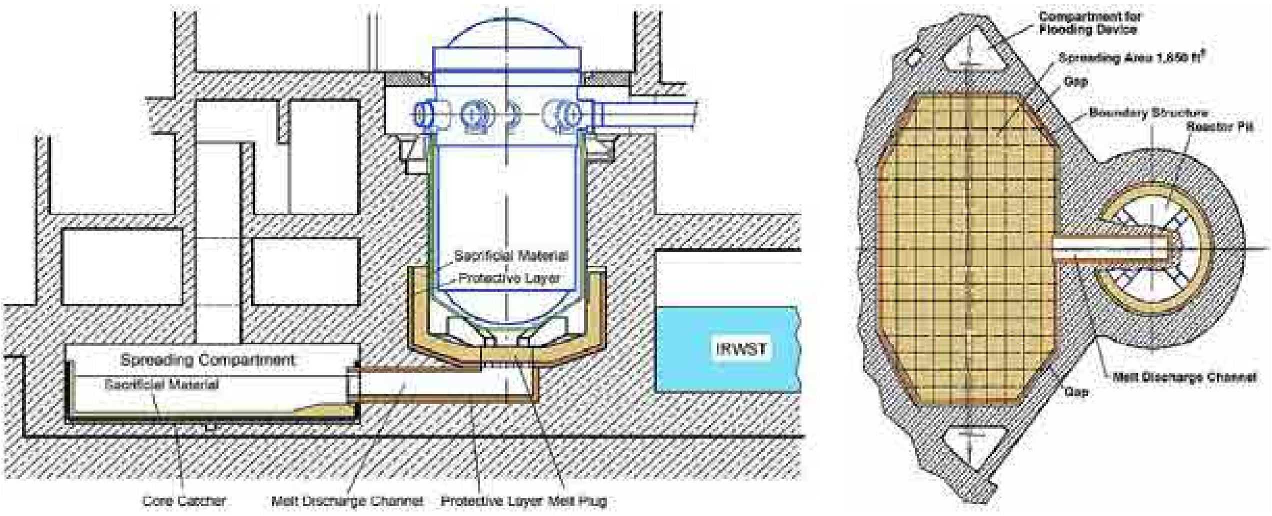


FIG. 179. Plan view of core melt spreading system and detail of core melt spreading compartment [591].

Other seismically important concrete structures include:

— The fuel building, a reinforced concrete structure.
— Four safeguard buildings of reinforced concrete located approximately three quarters of the periphery of the reactor shield building.
— Two emergency power generation buildings located adjacent to the nuclear island common basemat structure and near the essential service water buildings. Each emergency power generation building is primarily constructed of reinforced concrete and is supported by its own independent reinforced concrete basemat.
— Essential service water buildings that house equipment and cooling water associated with the essential service water system.

See Ref. [591] for further details regarding the EPR design.

The System 80+ primary containment design is a spherical welded steel shell. Bottom containment portions are embedded in concrete and connected to the shield building and the internal structure's basemat by radially extending shear transfer plates. Above the embedment region, containment is an independent freestanding steel structure. There is no structural connection between the freestanding portion of containment and adjacent structures, except for penetrations and supports. Figure 180 provides a cross-section showing the spherical steel containment inside the reinforced concrete secondary containment.

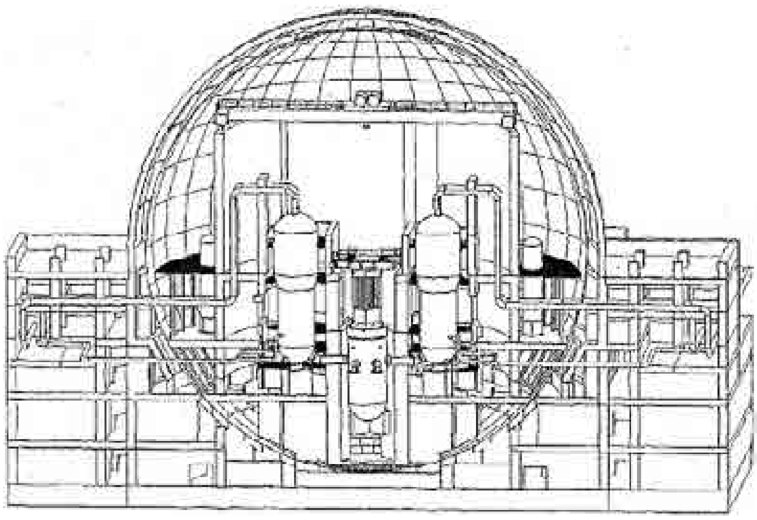


FIG. 180. System 80+ cross-section showing the spherical steel containment inside the reinforced concrete secondary containment [592].

The RB comprises the containment shield building and subsphere, in addition to the steel containment vessel and internal structures. The containment shield building is a reinforced concrete right circular structure with a hemispherical dome that shares a common basemat with nuclear annex structures. It serves as a biological shield and as protection from external missiles for the containment vessel and safety related equipment enclosed within the containment building. Above the concrete base there is an annular space between the containment vessel and the shield building for structural separation.

Internal structures consist of reinforced concrete structures that enclose and support the systems and components inside containment and also serve as biological shields and missile protection. The internal structure's basemat rests inside the lower portion of the containment vessel and is connected to it with radially extending steel bars.

The primary shield encloses the reactor vessel, protects it from internal missiles and serves as a biological shield. The secondary shield wall (crane wall) supports the polar crane, protects the containment vessel from internal missiles and serves as a biological shield for coolant loop and equipment.

The refuelling cavity also includes storage areas for the upper guide structure and crane support barrel. When filled with borated water, the refuelling cavity forms a pool above the reactor vessel.

The nuclear island structure's basemat is reinforced concrete, with a flat bottom resting on soil or rock. Separate reinforced concrete basemats are used for seismic non-nuclear island structures.

The cavity flooding system provides a means of flooding the reactor cavity to cool debris and to scrub fission product releases in the event of an accident [593]. The cavity is sized and configured to spread ejected core debris over the limestone concrete floor surface during a postulated accident. Within the cavity, the containment shell is protected from corium debris by a concrete basemat layer designed to protect the containment shell for at least 30 hours at its thinnest point. Additionally, concrete below the containment pressure vessel boundary acts to further prevent basemat melt through for at least eight days [594].

See Ref. [595] for further details.

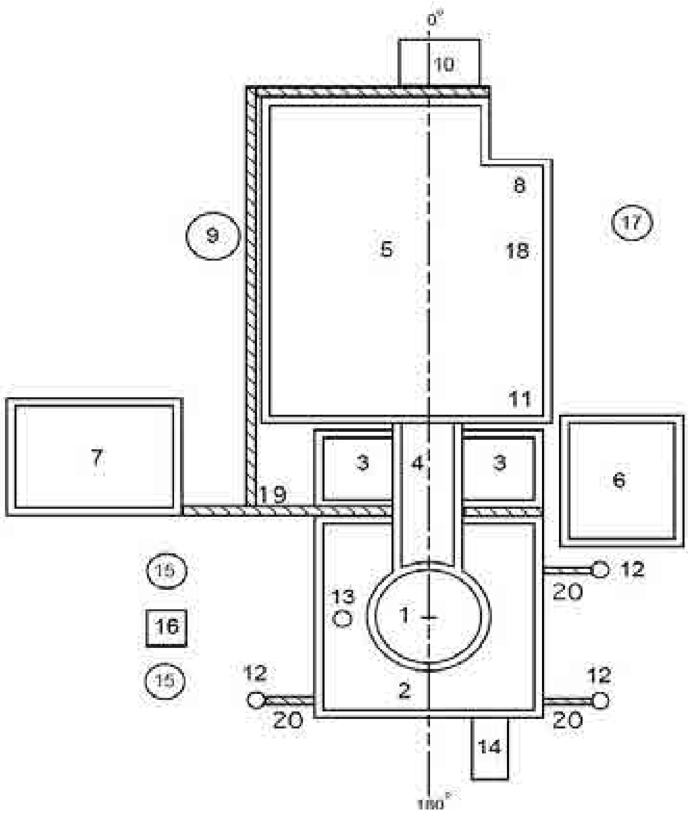
II.2.2. Boiling water reactors

The advanced boiling water reactor is a direct cycle BWR that incorporates modular construction methods to minimize construction time. Its containment is surrounded by an RB, with the two buildings integrated to improve overall seismic response by using larger load-bearing walls (see Fig. 181).

The ABWR pressure suppression and primary containment system comprises a drywell, wetwell and supporting systems. A 'safety envelope' encloses containment, with the exception of areas above the containment top slab and drywell head. The steel reactor pressure vessel pedestal and containment are integrated with and are fully contained within the RB. The RB surrounds the containment/safety envelope and serves as a secondary containment.

The reinforced concrete containment vessel (RCCV), located in the centre of the RB, encloses the reactor pressure vessel. This containment vessel supports the upper pool and is integrated with the RB structure from the basemat up through the RCCV top slab. It is a low leakage reinforced concrete structure with an internal steel liner serving as a leaktight membrane in the drywell and suppression chamber. It is divided by the diaphragm floor and reactor pedestal into an upper drywell chamber, a lower floor and drywell, and a suppression chamber. The containment wall, top slab, RB floor slabs and basemat are constructed of cast-in-place, conventionally reinforced concrete.

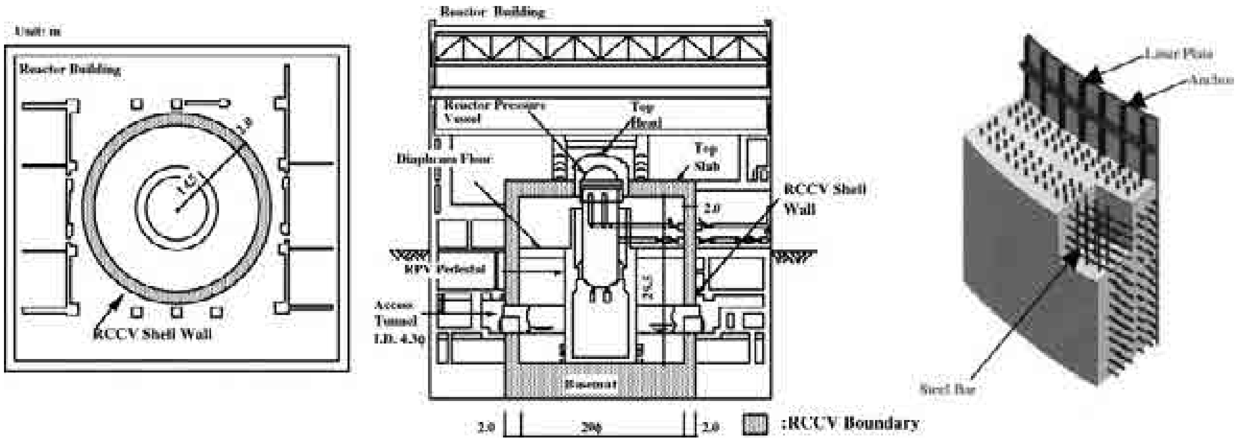
The containment top slab main reinforcement consists of top and bottom layers of rebar, with both layers arranged in a rectangular grid. The bottom layer is bent near the containment wall into a radial pattern to avoid interference with wall vertical reinforcement. Hoop reinforcement is provided in the area of the drywell head opening.



No	FACILITY
1	REACTOR CONTAINMENT
2	REACTOR BUILDING
3	CONTROL BUILDING
4	MAIN STEAM/FEEDWATER TUNNEL
5	TURBINE BUILDING
6	SERVICE BUILDING
7	RADWASTE BUILDING
8	HOUSE BOILER
9	CONDENSATE STORAGE TANK
10	UNIT AUXILIARY TRANSFORMERS
11	NORMAL SWITCHGEAR
12	DIESEL OIL STORAGE TANK (3)
13	STACK
14	EQUIPMENT ENTRY LOCK
15	FIRE PROTECTION WATER STORAGE TANK (2)
16	FIRE PROTECTION PUMPHOUSE
17	BUNKER FUEL TANK
18	COMBUSTION TURBINE GENERATOR
19	RADWASTE TUNNELS RB, CB, TB
20	DG OIL TRANSFER TUNNEL (3)

FIG. 181. ABWR site plan: CB is control building; TB is turbine building and DG is diesel generator [596].

The containment's internal surface is lined with welded carbon steel plate (stainless steel plate or clad is used on the suppression chamber's wetted surfaces) to form a leaktight barrier. The liner is stiffened using structural sections and plates to carry design loads and to anchor the liner to the concrete. The containment wall liner and top slab liner serve as a form for concrete placement. A cross-section of the RB and RCCV are shown in Fig. 182, along with a section of the RCCV wall showing concrete, rebar and the liner plate. Containment is rendered inert with nitrogen during operation.



(a) (b) (c)

FIG. 182. (a) ABWR reinforced concrete containment vessel top view[3]*; (b) side view*[3]*; (c) section of RCCV wall [597].*

Containment internal structures are made of reinforced concrete and structural steel and include:

— *A diaphragm floor*. A reinforced concrete circular slab serves as a barrier between the drywell and suppression chamber. A carbon steel liner plate on the bottom of, and anchored to, the diaphragm floor serves as a form during construction and prevents bypass steam from flowing from the upper drywell to the suppression chamber air space during a LOCA.
— *A reactor pedestal*. A composite steel and concrete structure providing support for the reactor pressure vessel, reactor shield wall, diaphragm floor, access tunnels, horizontal vents and lower drywell access platforms. It consists of two concentric steel shells tied together by vertical steel diaphragms with the voids between them filled with concrete, except the vents and vent channels. Wetted portions of exterior surfaces of the reactor pedestal steel shell in the suppression chamber are clad with stainless steel to provide corrosion protection.
— *A reactor shield wall*. Supported by the pedestal, surrounds the reactor pressure vessel. The wall is shaped as a right cylinder and consists of two concentric steel cylindrical shells joined together by horizontal and vertical steel plate diaphragms and filled with concrete.
— *Drywell equipment and pipe support structures*. Consisting of structural components such as beams and columns. Built up box shapes are used for beams and columns that must resist torsion and biaxial bending. The beams span between the shield wall and vertical support columns that are anchored to the diaphragm floor.
— *Miscellaneous platforms of steel beams and grating*. Designed to allow access and to provide support for equipment and piping.

The turbine building is a reinforced concrete building with a steel superstructure housing power conversion system components.

RB and containment are integrated to improve seismic response. The RB provides biological shielding for plant personnel and the public outside of the site boundary and doubles as a secondary containment to protect the reinforced concrete containment from environmental hazards, such as tornado and site proximity generated missiles. The RB is a reinforced concrete shear wall structure surrounding the primary containment and its walls are designed to accommodate all seismic loads. Frame members, such as beams or columns, are designed to accommodate wall deformations during earthquake conditions.

[3] INTERNATIONAL ATOMIC ENERGY AGENCY, Incident Report 772, 1987-09-07, Gravelines-I, France (1999), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=1073

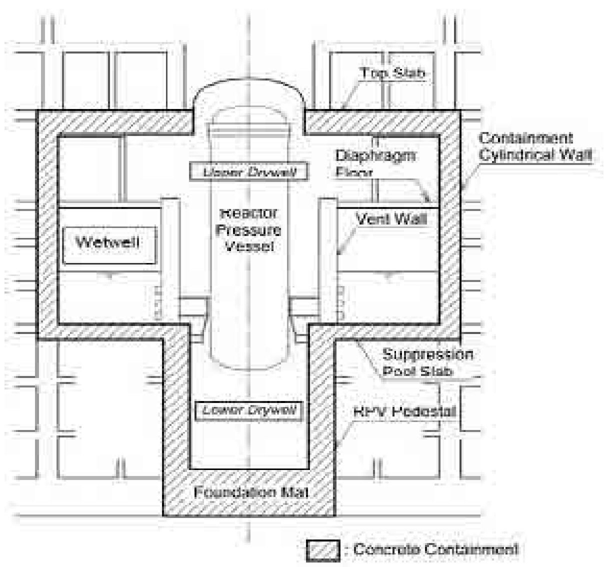
RB and control building foundations are reinforced concrete basemats separated by a gap to minimize structural interaction between the buildings. The RB basemat and the radwaste building basemat are constructed of cast-in-place conventionally reinforced concrete.

See Ref. [598] for further details on the ABWR.

The economic simplified boiling water reactor (ESBWR) is a 4000 MW(t) (1390 MW(e)) reactor that uses natural circulation for normal operation and has passive safety features. The ESBWR design has a pressure suppression type containment completely enclosing the reactor system, drywell, suppression chamber and certain other associated volumes. This containment, in conjunction with the RB and other safety related features, limits radiological effects from design basis accidents to less than prescribed regulatory limits.

Containment is a low leakage reinforced concrete structure with an internal steel liner in the drywell and wetwell to serve as a leaktight membrane. Containment configuration is shown in Fig. 183 along with key dimensions. The containment wall and reactor pressure vessel pedestal are right circular cylinders with the cylindrical outer wall extending below the suppression pool floor slab to a common basemat. The containment top slab and suppression pool slab are circular plates of uniform thickness. As noted in Fig. 183, containment is divided into drywell and wetwell regions with an interconnecting vent system.

The drywell region is a leaktight gas space, surrounding the reactor pressure vessel and reactor coolant pressure boundary, which provides containment for radioactive fission products, steam and water released by a LOCA prior to directing them to the suppression pool via the drywell/wetwell vent system. A relatively small quantity of drywell steam is also directed to the passive containment cooling system during LOCA blow down. The drywell is comprised of upper and lower chambers. The upper drywell is a cylindrical, reinforced concrete structure with a removable steel head and a diaphragm floor constructed of steel girders with concrete fill.



Structure	Dimension
Foundation mat	Thickness = 5.1 m
Containment wall	Thickness = 2.0 m
	Inside radius = 18.0 m
	Height = 19.95 m (From top suppression pool slab to bottom top slab)
Reactor pressure vessel (part of lower containment)	Thickness = 2.5 m
	Inside radius – 18.0 m
	Height = 15.05 m (From top foundation mat to top suppression pool slab)
Top slab	Thickness = 2.4 m
Suppression pool slab	Thickness – 2.0 m

FIG. 183. Configuration and key dimensions of concrete containment [599].

The wetwell region consists of a suppression pool with a gas space above it. The suppression pool is a large body of water designed to absorb the energy of condensing steam from safety relief valve discharges and pipe break accidents. The wetwell boundary is an annular region between the vent and containment walls and is bounded above by the drywell diaphragm floor.

The containment liner plate is fabricated from carbon steel, except that stainless steel plate or clad is used on wetted surfaces and at the water to air interface area of the wetwell and gravity driven cooling system pools as protection against pitting and corrosion.

The RB encloses the concrete containment, its internal SSCs and is integrated with an RCCV, both located on a common basemat. The RB and roof are constructed of reinforced concrete. Roof trusses and supporting columns are made of structural steel. The RB outer walls are reinforced concrete shear walls and the building is partially embedded. Figure 184 presents a cross-section showing the RB arrangement.

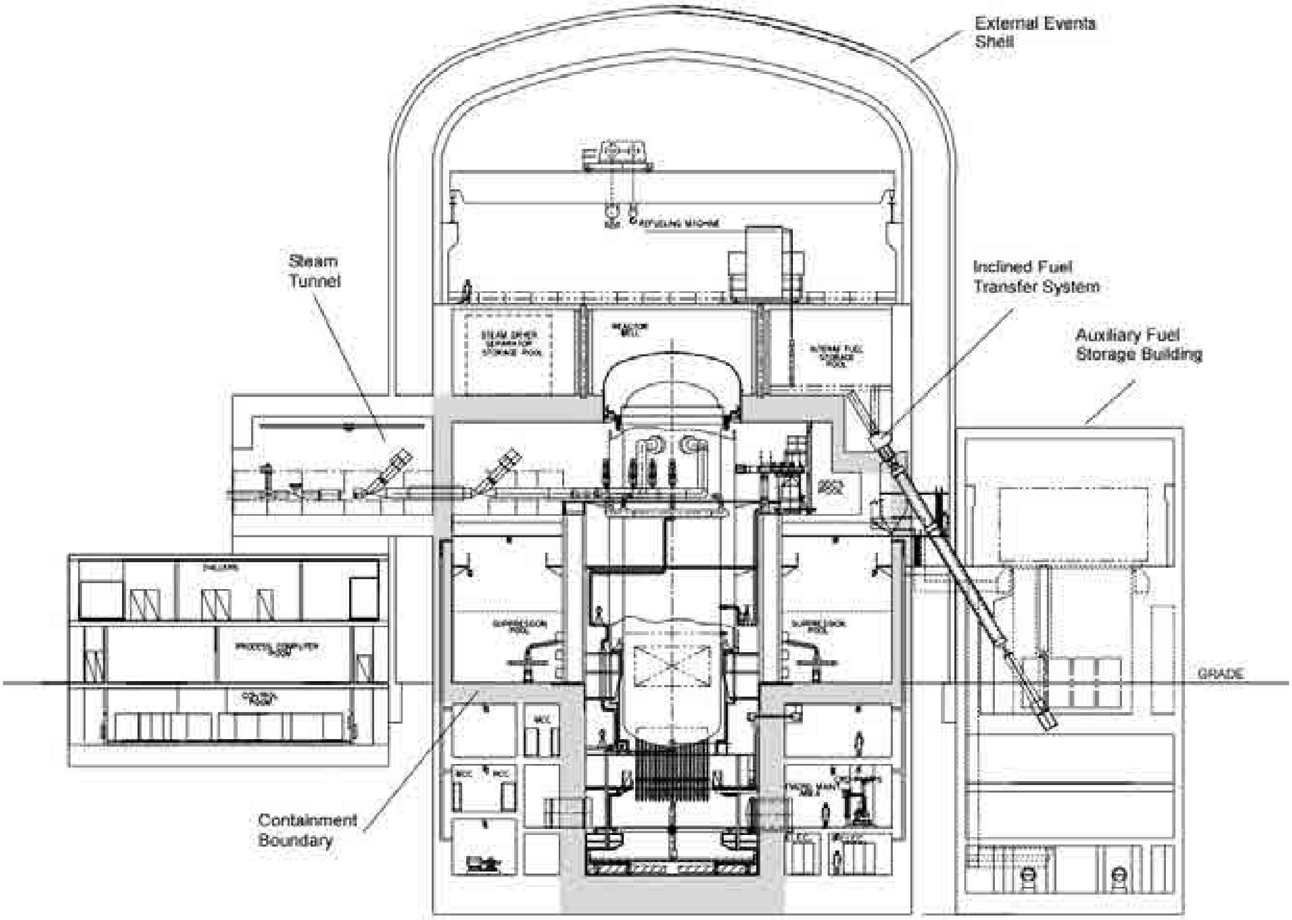


FIG. 184. General arrangement of an ESBWR reactor building [600].

The RB, containment and fuel building are built on a common rectangular reinforced concrete basemat constructed of cast-in-place conventionally reinforced concrete. The basemat is a flat plate.

See Ref. [599] for further details on the ESBWR.

II.2.3. Pressurized heavy water reactors (ACR1000, EC6 and AFCR)

New CANDU designs are based on the single unit design described in II.1.3. RB walls and dome are post-tensioned, while the basemat is of reinforced concrete construction. Basemat construction is a continuous one concrete pour in order to minimize the number of construction joints, to improve durability and to keep on the construction schedule.

Containment wall thickness and amount of reinforcing steel have been increased to achieve the higher design pressure and to provide increased protection from external threats. Containment is designed to handle the pressures calculated for all design basis accidents, including main steam line breaks. A steel liner is used on the internal surface of the RB to enhance leaktightness.

The irradiated fuel discharge chamber and its access passage are of reinforced concrete construction with a steel liner.

II.2.4. Water-water power reactors (WWER-1200 (NPP-2006))

WWER-1200 NPPs (NPP-2006) have double containment shells consisting of an inner leak proof shell of prestressed reinforced concrete and an outer non-prestressed reinforced concrete shell, which, together with the floor panel at baseline ground elevation, restrains an annulus that is kept under vacuum for collecting possible steam air leaks from accidents. A general view of an NPP-2006 containment shell is shown in Fig. 185.

The outer reinforced concrete shell is intended to protect systems from impacts (e.g. aircraft, external air waves, extreme winds and climate) and the external environment. It also provides for a ventilated gap and leak collection. The heavy concrete shell is cylindrical, with a monolithic reinforced concrete hemispherical dome. Dimensions are listed in Table 33.

TABLE 33. CONTAINMENT SHELL GEOMETRICAL DIMENSIONS OF AN NPP-2006

Item	Dimension	Notes
Cylinder inner diameter	50.8 m	
Reference mark of dome top	+64.4 m	
Thickness of walls and dome	0.5 m	Chosen to accommodate aircraft impact load (for cylinder areas protected from aircraft impact, thickness is 0.4 m)
Gap between outer and inner shells	2.20 m	Chosen to allow for inner containment shell prestressing system maintenance, shell surfaces inspection and to allow for access to and maintenance of equipment located in gap

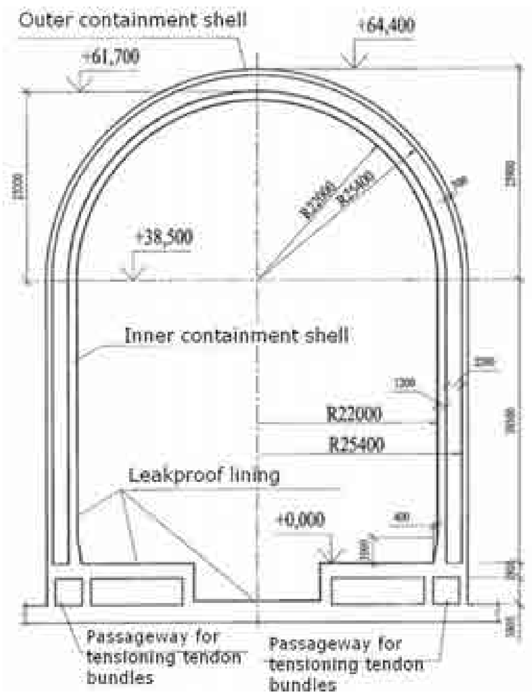


FIG. 185. NPP-2006 containment shell and geometrical dimensions (courtesy of Rosenergoatom).

The inner containment shell is produced from prestressed reinforced concrete with insulated steel cladding and is designed to restrict radioactive material leakage under design accident parameters combined with the maximum design earthquake.

The NPP-2006 type reactors have an inner containment shell that consists of a cylindrical and hemispherical dome. The cylinder inner radius is 22.0 m, and the design cylinder and dome wall thickness is 1.2 m. The shell height is 61.7 m. The shell inner surface has 6 mm thick insulated steel cladding.

An orthogonally-looping scheme of prestressed concrete strands is used for creating prestress. In this design, two vertical pilasters in the lower ring passageway anchor horizontal prestressed concrete strands of the cylinder and dome. A schematic view of the prestressed concrete strand arrangement is shown in Fig. 186.

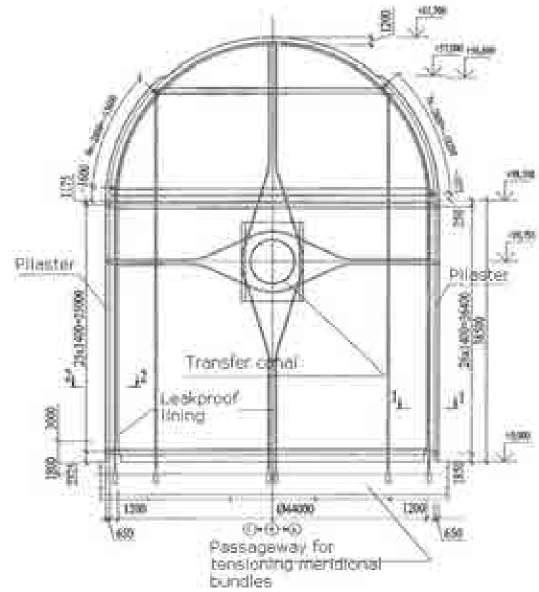


FIG. 186. NPP-2006 inner containment shell and prestressed concrete strand arrangement in cylinder and dome (courtesy of Rosenergoatom).

II.2.5. Modular reactors (mPower and 4S)

Modular reactors are currently under development for potential future use. Being smaller, the containment volumes are typically also smaller than those in conventional NPPs. The Westinghouse designed small modular reactor (SMR), for example, has containment dimensions of 9.8 m outer diameter and 27.1 m height compared to the 39.6 m outer diameter and 65.6 m height of their AP-1000 design.

The mPower reactor is a simplified, passive, modular, light water cooled PWR in which the reactor core, steam generator and pressurizer are combined into a common pressure vessel that also encloses the control rod drive mechanisms and reactor coolant pumps. The reactor, with a rated power output of approximately 125 MW(e), can operate for up to four years between refuellings.

The nuclear island is also small compared to conventional PWRs, and has the containment building and other critical structures located below grade level. Figure 187 is a cutaway view of the reactor service building and fuel handling building for an arrangement that houses two reactor modules.

FIG. 187. Reactor service building and fuel handling building for a two unit mPower plant [601].

The containment building (see Fig. 188) is a low leakage, reinforced concrete, steel lined, seismic structure. Located below grade level are the reinforced concrete control and fuel handling buildings. Located partially below grade level are the reinforced concrete reactor service building, which surrounds containment, and the reinforced concrete and steel frame radioactive waste building.

FIG. 188. Babcock and Wilcox mPower small modular reactor containment (courtesy of Babcock & Wilcox).

Refer to Ref. [601] for more detail on the mPower design.

The super safe, small and simple reactor (4S) is offered by a partnership, including Toshiba and the Central Research Institute of Electric Power Industry [602]. The reactor presently being considered is a 10 MW(e) (30 MW(th)) plant intended for use in remote locations and will operate without refuelling during its planned 30 year life. It is a small, sodium cooled fast reactor with a core heat source about 0.7 m in diameter by about 2 m in height. Liquid sodium cooling allows the 4S to produce steam temperatures of around 500°C.

The 4S is a totally enclosed unit with core and primary coolant loops sealed in a cylindrical structure. Enclosing the reactor assembly and core are the reactor vessel, guard vessel, guard vessel top dome and the heavily reinforced concrete cap on top of the reactor module [603]. The guard vessel surrounds the reactor vessel and ensures that the core will not be uncovered, and that core cooling can be accomplished even if the reactor vessel leaks. The reactor vessel contains and supports the core, the primary radioactive sodium coolant and the primary sodium intermediate heat exchangers. The reactor vessel is installed up to 30 m below grade in a structure with a large, heavily reinforced concrete cap. The RB houses the reactor module, steam generator and other vital safety equipment. The lower part of the reactor module (containing the core) is in its own below grade reinforced concrete silo. The heavy reinforced concrete cap structure on top of the reactor module encloses the reactor assembly and core and provides the only access to the nuclear fuel, which is contained within the reactor module. Heavy lift equipment is required to remove the cap and gain access to the nuclear fuel. Figure 189 presents a schematic view of 4S installation.

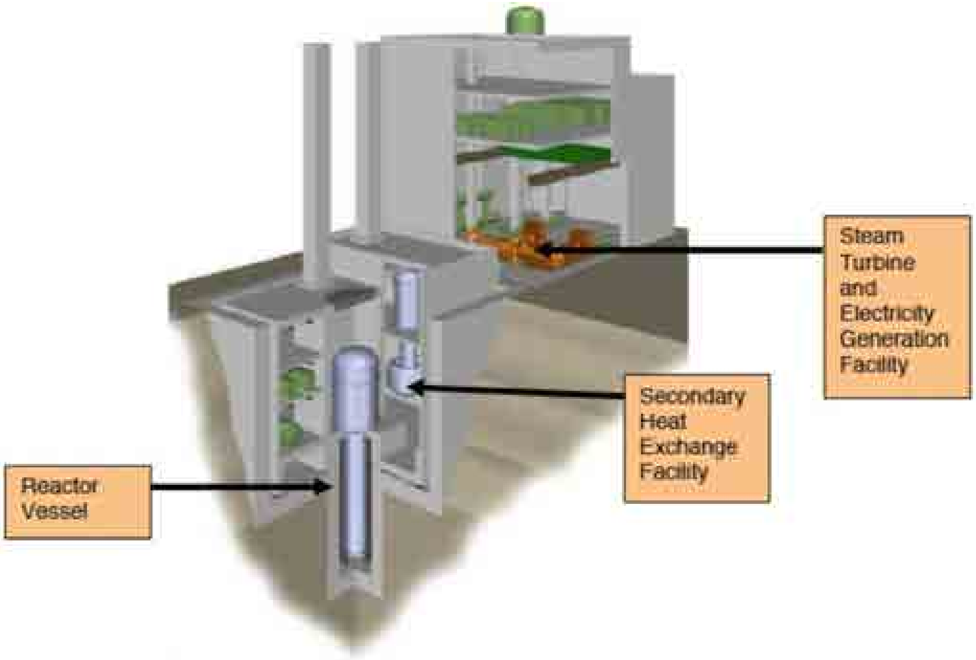


FIG. 189. Schematic diagram of 4S installation showing a reactor vessel installed about 30 m below grade [604].

Appendix III

AGEING MANAGEMENT PRACTICES AND OPERATING EXPERIENCE OF SEVERAL MEMBER STATES

Inspection and monitoring programmes used by Member States to assess concrete structures vary significantly. Examples of several of these programmes, and associated ageing management experience, are summarized below.

III.1. BELGIUM

Belgium's seven PWR units, built to US regulatory (e.g. ASME Boiler and Pressure Vessel Code) requirements and operated by Electrabel SA, entered commercial operation between 1974 and 1985. Each unit has its own in-service inspection programme that includes visual inspection of the prestressed concrete and liner, lift-off tests of the tendons and monitoring strain evolution in the prestressed concrete. These programmes have been updated to take into account in-service inspection requirements contained in subsections IWE and IWL of the ASME Code.

Belgian NPPs have no time limited authorization of operation and therefore no predetermined licensed life. Safety authorities continually assess safety behaviour and, as required in each NPP's licence, an overall safety review must be performed every ten years.

Containment has been instrumented at both the Tihange and Doel sites. Due to a defect in several of Doel 3's containment strain gauges, new vibrating wire strain gauges were installed on the face of the concrete prestressed concrete shell [605].

III.2. CANADA

The Canadian vendor of pressurized heavy water (CANDU) reactors until 2011 was Atomic Energy of Canada Ltd (AECL) — a crown corporation (i.e. wholly owned by the Canadian Government). In 2011 SNC-Lavalin Inc. acquired AECL's CANDU Reactor Division and Candu Energy Inc. was formed to specialize in the design and supply of nuclear reactors, as well as nuclear reactor products and services. The Canadian authority that regulates and licenses NPPs is the Canadian Nuclear Safety Commission (CNSC). CANDU stations are owned and operated by a group of Canadian utilities — Ontario Power Generation, Bruce Power, New Brunswick Power Corporation and Hydro Quebec. Currently, there are 22 nuclear reactors in Canada and the Canadian nuclear industry is working together to address ageing management of concrete structures.

III.2.1. Ontario Power Generation

Ontario Power Generation (OPG) operates 10 CANDU reactors at the Pickering and Darlington multiunit stations. Two additional reactors are in long term safe storage at the Pickering site. OPG's oldest unit has been in commercial service for over 40 years and underwent a major refurbishment/retubing outage in 2001.

III.2.1.1. OPG ageing approach

OPG's overall approach to life management is to:

- Ensure that appropriate maintenance, inspection and surveillance programmes are in place for SSCs;
- Ensure ongoing quality and effectiveness of the programmes implemented;
- Incorporate OPEX to continuously improve the AMP and respond to new ageing concerns as they occur;
- Carry out periodic programme reviews and initiate modifications in programmes to improve their (programme) objectives.

OPG has a systematic approach to the ageing of concrete structures in accordance with CNSC RD-334 [261].

The AMP focuses on understanding and managing degradation mechanisms that are critical to long term reliability, safety and life of the plant since they cannot easily and economically be replaced. Major civil structures that fall under the AMP include CCSs, such as the vacuum building, pressure relief duct, RBs and structures supporting containment. While performance of civil structures has generally been good, there have been incidences of localized material deterioration and increases in leakage rate during pressure tests [606]. Procedures for standardized inspection programmes and repair materials are available.

The AMP is set-up to meet the operating station licence requirements. Accompanying AMP governance is composed of:

- An integrated AMP, with an ageing management process, which provides the overall ageing management process description with specific instructions;
- Periodic inspection programme plans (PIPs);
- A leakage rate testing programme;
- On-power testing;
- Submittals to the regulator (CNSC);
- Lessons learned reporting;
- Health reporting.

Figure 190 illustrates the AMP outline, which describes the interrelationship between facets of the ageing management strategy. This AMP evaluates interfacing programmes and practices currently in place at OPG that create a systematic and integrated approach to manage CCB ageing. The programme is adapted from regulatory document RD-334 [261], operating experience from CANDU plants, outside industry, CANDU Owners Group (COG) and other industry leaders such as the IAEA.

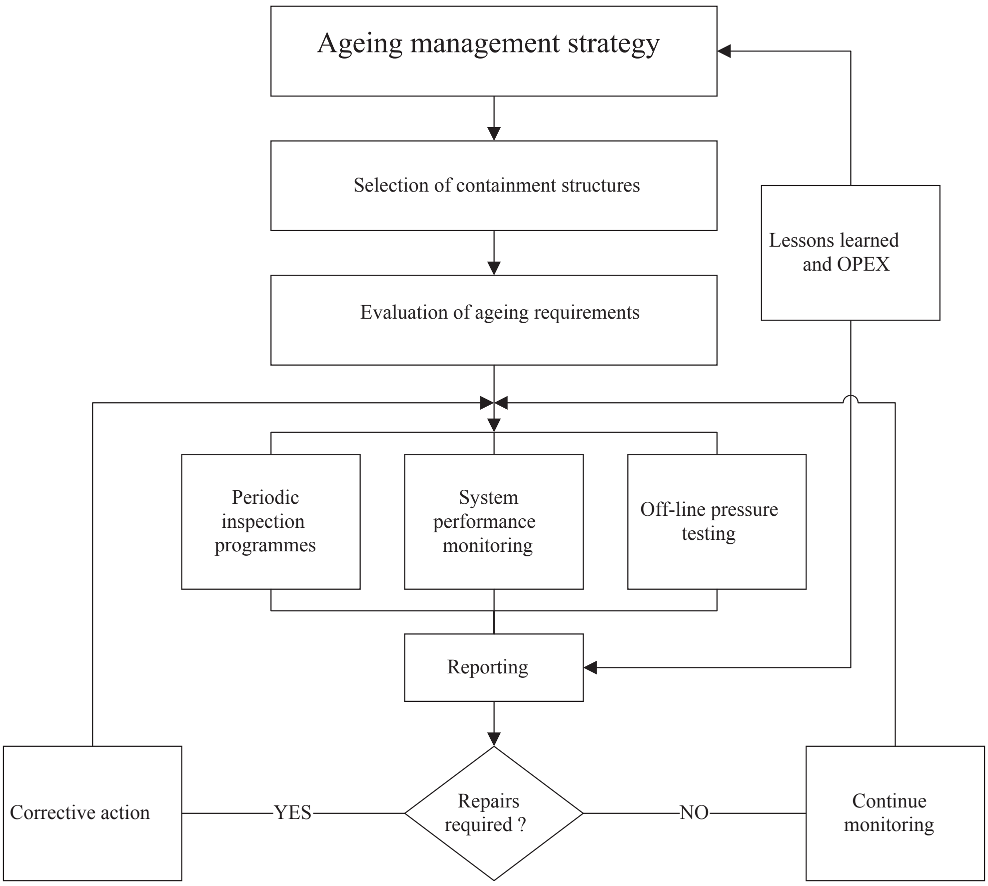


FIG. 190. OPG ageing management strategy.

Factors of an effective programme include:

- — Selection of safety structures subject to physical ageing management;
- — Understanding of ageing and ageing phenomenon;
- — Management of ageing;
- — Prevention of ageing;
- — Monitoring of ageing;
- — Mitigation of ageing.

III.2.1.2. OPG approach to understanding of ageing of concrete structures

Accompanying non-destructive inspection techniques and analytical methods may be used to determine the structure condition.

(a) Material databases

Research has been performed on various repair materials meeting the CANDU operations' environmental conditions. The following material categories were researched and tested:

- — Protective paints, coatings and non-metallic liners:
  - • Joint sealants;
  - • Injection resins;
  - • Patching and overlay materials;
  - • Expanding/non-shrink prepackaged grouts;
  - • High density grouts.

Testing is ongoing and is performed to CSA N287.2-08 [245].

(b) Impact of elevated temperature on concrete containment

Below is a summary of the impact of elevated temperature on concrete containment (IFBs could be considered as containment with leaktightness requirements).

Methods for determination of the ultimate (collapse) strength of reinforced concrete structures based on linear elasticity or non-linear plasticity are widely developed. However, if the area of concern is related to reinforced concrete containment's leaktightness and ability to contain, then analysis results need to be much more detailed in terms of accurate representations of cracking (crack patterns and opening widths).

More sophisticated and appropriate fracture mechanics approaches have been developed for applications such as crack analysis, where classical plasticity analyses encounter problems due to the phenomena of strain localization and stress-locking, as well as mesh induced orientation and refinement bias.

In recent years, the extended finite element method (XFEM) has emerged as a powerful numerical method for the analysis of problems involving cracking and has become the fundamental framework for computational fracture mechanics. This fracture mechanics approach addresses not only the strength criterion for crack initiation as used in classical plasticity analyses, but also the more important fracture energy criterion that is closely associated with crack propagation. In 2009, XFEM was first included in the finite element analysis program ABAQUS 6.9 release. Since then, ABAQUS has continued the improvement of XFEM applicability to static and dynamic fracture mechanics and low cycle fatigue cracking simulation.

The effect of elevated temperature on concrete and steel properties is accounted for by using the temperature dependent fields that are available in ABAQUS. Temperature dependent material properties used for fracture analyses are uniaxial tensile and compressive strengths, modulus of elasticity, coefficient of thermal expansions, thermal conductivity and specific heat of both concrete and reinforcing steel, as well as the concrete fracture energy.

The temperature distribution and gradient within the containment structure is attained for incremental increases of temperature. This temperature field is affected by thermal boundary conditions and interaction with ambient atmosphere and underground heat sinks. The properties of atmospheric and underground heat sinks are considered in the heat transfer analysis. These heat sinks can be simplified to be thermal boundary conditions using film coefficients on mechanical boundaries. Both steady state and transient heat transfer analyses can be performed depending on the significance and importance of transient influence on results.

The stress condition in the containment structure before temperature excursion is initialized in a linear stress analysis with pre-existing mechanical loads. The result of heat transfer analysis in terms of the temperature field is then read into the subsequent non-linear fracture mechanics analysis. Stable increment sizes are governed by numerical stability of crack nucleation and growth.

The results of fracture mechanics analysis can be used to present structural performance on concrete containments in terms of concrete crack patterns and sizes as well as reinforcement yielding for different stages throughout the thermal loading process. Crack patterns and opening sizes are used for subsequent leakage rate assessment. Safe temperature limits of containments can be determined based on crack acceptance and leakage rate.

(c) Post-tensioning systems

The Pickering vacuum building roof is designed using 64 ungrouted vertical post-tensioning rods (PTRs).

After 41 years in-service, visual inspections in 2012 of the Pickering A vacuum building PTR exposed components indicated no significant corrosion or presence of cracks in the PTRs, and no cross-section reduction or material deformation of the PTRs. No visible cracking in the exposed concrete surface around the baseplate was observed. Trending indicates no significant changes in inspection results in 2012 compared to those from 2010.

(d) R&D results

Research is ongoing to support the CANDU community regarding maintenance issues of concrete containment.

The following is a partial list of the completed research reports. Research is ongoing on these issues and there are numerous reports issued.

— COG-09-4031: Evaluation of Underwater Coatings for Application at CANDU Stations;
— TN-07-4050: Determination of CANDU Qualified Civil Structure Repair Materials;
— COG-05-4051: Strategic Direction — Concrete and Containment;
— TN-12-4041: Losses of Prestressing Force in Concrete Containment Structure;
— COG-12-4038: Concrete Containment Building — Testing of NDT Techniques and Investigation of Post-Tensioned Concrete;
— COG-10-4042: Concrete Stress and Strain Data Collection and Analysis for Ageing Management for Nuclear Containment Buildings;
— COG-07-4047: Long Term Performance of Patching and Overlay Repair Materials.

(e) OPEX and lessons learned

Pickering operating experience. The reported conclusions from 2000 to 2010 indicate that concrete is in good condition and, generally, components are within design basis acceptance criteria and continue to fulfil their design and safety functions.

Any exceptions are repaired, tracked, trended and additional research through COG is performed to identify areas for improvement. The exceptions were the vacuum building floor slab, materials such as caulking and protective coating on EPs, the elastomeric dome coating, the vacuum building roof seal, elastomeric pressure relief valve and instrumented pressure relief valve components, concrete in areas where new cracking is exhibited due to ageing and concrete areas that require trending of existing degradations. Minor repairs such as shrinkage cracks, localized spalling and fine cracks around penetrations have been dispositioned for repair and/or future monitoring.

Darlington operating experience. Trending of data indicated that concrete containment continues to fulfil its design and safety function. Inspection findings reported since 2000 show that no new degradation mechanisms have been reported.

Based on the criticality of degradation, preventive maintenance repairs were requested for cracks in concrete, localized spalling, caulking and fine cracks around penetrations. Few degradation locations were actioned for future monitoring.

Caulking was mostly in an acceptable condition. Some exceptions were missing and delaminated caulking around some EPs and construction joints.

Steel liners, including welded joints between liner plates and accessible penetrations, were found to be in good condition, with no major structural deterioration identified. Minor surface rust, rust stains and missing coating were identified in inspections.

As noted in earlier inspections, the 2009 inspection reaffirmed that examinations of anchorage components, grease cans, tendon prestress forces and concrete around the post-tensioning system provided acceptable inspection results. No remedial actions were required on the post-tensioning system.

Lift-off tendon post-tensioning forces were found to be higher than predicted forces and within acceptable limits.

Concrete degradation has been non-active with only minor changes noted as trended from year over year inspection findings. As such, it is inconclusive if leakage was due more to degradation of concrete structures or to degradation of mechanical equipment.

(f) Lessons learned summary

CCS lessons learned documents have been created to capture Pickering inspection history results from 1980–2010, and Darlington from 1992–2010.

Pickering’s lessons learned documents address containment ageing of components including:

— Leak types;
— Sealant ageing;
— Repair longevity;
— Ageing of lining systems;
— Construction workmanship;
— Post-tensioning;
— Concrete;
— Concrete readings results;
— Galvanized steel;
— Galvanic action;
— Ageing mechanisms;
— Epoxy injection techniques;
— Non-metallic liner material;
— Roof seal strength.

Other areas addressed include process gaps, deterioration of leak types and gaps in trending. Recommendations targeted the missing gaps and focused on additional inspection areas, inspection frequency, planning of inspections, post-tensioning inspection, qualifications, trending and process improvements.

The Darlington lessons learned documents address the following:

— Ageing mechanisms;
— Concrete surrounding large openings;
— Degradation types and stressors;
— Observed ageing of concrete;
— Tendon lift-off forces;
— Repairs of mortar patches;
— Trend in the dousing water tank rebound hammer readings;
— Post-tensioning system components;
— Caulking;
— Paint;

— Inaccessible areas;
— Reporting on leakage types and on the reactivity mechanism deck seal.

III.2.1.3. Regulatory requirements, codes, standards and safety criteria

The station operating licences comply with current codes and standards. To meet the licence requirements, a periodic inspection programme is implemented and maintained in accordance with CSA N287.7, In-Service Examination and Testing Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants [249] requirements.

III.2.1.4. Inspection, monitoring and assessment of structures

Pickering concrete containment is composed of the six RBs, the vacuum building and the pressure relief duct.

The intent of periodic inspection programmes is to detect service induced deterioration and/or inherent flaws of concrete components, and their parts, whose failure would undermine containment envelope leaktightness and adversely affect operability and structural integrity of the containment system.

(a) *Inspections of Pickering concrete containment*. Concrete containment is subject to regular visual inspections (6–12 year intervals) and design pressure tests to verify containment structural integrity and leakage releases meet licensing standards and satisfy regulatory requirements. Separate periodic inspection programmes that meet CSA N287.7 [249] requirements have been created for the RBs, the vacuum building, and the pressure relief duct.
(b) *Inspection techniques*. The following inspection techniques are used for periodic inspection of CCSs:
— NDT:
  • Visual inspection;
  • Rebound hammer test, PUNDIT (portable ultrasonic non-destructive digital indicating tester) instrument, light scale loupe or feeler gauges to estimate crack width;
  • Weld inspections using magnetic particle, liquid penetrant, radiography and ultrasonic methods.
— Invasive examination:
  • Invasive testing including concrete core drilling, petrography and electrical half-cell potential may be used.
— Analytical methods:
  • Approved analytical methods may be used to evaluate structural behaviour and resistance.
(c) *Inspection results.*
Darlington containment: There were no significant degradations (i.e. observations outside of acceptance criteria) identified, nor were any significant repairs requested. Leakage rate trends were within acceptable limits, indicative of containment structures' good health and material performance.

Pickering containment: Containment inspections performed from 1992–2007, including inspections of the vacuum building, common areas (Unit 0) and the eight RBs, indicate that CCSs were in good condition and within design basis. The overall containment meets its design function and is able to withstand the environment encountered at the interior and exterior of containment. Reported conclusions from 2000–2010 inspections also indicate that concrete containment is in good condition. The good quality of the containment boundary is confirmed by NDE inspection methods, indicating that components are within design basis acceptance criteria and the containment boundary continues to fulfil its design and safety functions. Historical test leakage rate data from station commissioning to the year 2000 is summarized in the Containment Leakage Rate Test Experience Report. Minor repairs such as shrinkage cracks, localized spalling and fine cracks around penetrations have been dispositioned for repair and/or future monitoring. Generally, trending for concrete and metallic components indicates little to no progressive deterioration and slight hardening of joint sealants. There is no threat to structural integrity and leaktightness of containment. Progressive concrete deterioration, as indicated by leakage trends from 1997–2003 through the concrete containment boundary, was not observed. Inspection campaigns were effective in identifying the deterioration described by the periodic inspection programme documents.

(d) *Non-metallic liners, sealers and coatings*. Pickering RB Unit1 Roof Dome was coated in 1993 with Decothane SP and inspected in 1999, 2005 and 2010. In 2010, no signs of deterioration or anomalies were observed. Similarly, no significant evidence of unacceptable deterioration (i.e. blistering, tearing, delamination or cracking) was observed in 1999 and 2005. The coating was found to be performing as expected.

(e) *Trending*. Trending is performed at every periodic inspection programme cycle. Trending for degradation mechanisms in concrete and metallic components indicates no threat to structural integrity.

III.2.1.5. Maintenance and repair of ageing effects

Exceptions are scheduled for repair and actioned for future monitoring.

Experience with operating a CANDU plant has shown varied ageing management challenges in the following areas:

— Inspection techniques of grouted prestressing reinforcement (challenges to be addressed);
— Repair materials to withstand harsh environments;
— Inspection techniques for difficult to reach concrete areas;
— Materials with faster curing periods;
— Materials for underwater repair under harsh conditions.

Continuous and lengthy research has to be performed to achieve requirements.

III.2.2. Bruce Power

Bruce Power is a private company operating eight CANDU reactors at two locations on the Bruce site under a long term lease arrangement with the government owned OPG. Four of the eight reactors have had major refurbishments. Approaches to ageing are in accordance with Canadian nuclear standards, licence and regulatory requirements similar to those taken via OPG.

III.2.3. Hydro-Québec and NB Power

Two Canadian single unit stations are Gentilly 2 and Point Lepreau. Point Lepreau underwent a major refurbishment (completed in 2012), while Gentilly 2 has (also in 2012) been permanently shut down. Both stations went into commercial operation in 1983. In 1994, New Brunswick Electric Power Corporation (NB Power), Hydro-Québec and the vendor AECL (now Candu Energy Inc.) initiated a common plant life management study addressing both stations. The study included activities evaluating concrete containment ageing.

Concrete ageing was mainly addressed through methods to control containment leaktightness. Two materials were considered to enhance the leaktightness of containment buildings — polyurethane and fibre reinforced mortar. Originally, RB concrete containment walls at Gentilly 2 and Point Lepreau had an epoxy liner on the inner surface to provide a leaktight barrier and assist in decontamination. Initial containment leakage rate test results at these stations were satisfactory, but subsequent results showed significant increases with time (e.g. 24 hour tests at Gentilly 2 in 1981, 1985 and 1987 produced leakage rates of 0.15%, 0.60% and 0.83%, respectively, of the containment free volume). The majority of leakage was attributed to an increasing number and magnitude of cracks in the containment walls. The original epoxy liner, considered too rigid to be capable of bridging the cracks, was replaced with an elastomeric polyurethane liner, which has resulted in a significant reduction in leakage rates.

Recently, an AMP for CCSs was developed to maintain integrity of the structure during life extension (for Point Lepreau).

III.2.4. CANDU Owners Group

The CANDU Owners Group is a not-for-profit corporation with voluntary funding from CANDU-owning utilities and AECL. Currently, COG membership includes five Canadian and six offshore members.

The activities of COG cover four programmes for collaborative research, information exchange, joint projects and regulatory affairs.

Research programmes were carried out that focused on the following:

- — Qualifying repair materials for concrete containment;
- — Evaluating effects of elevated temperatures on concrete;
- — Evaluating potential structural/functional issues associated with RB containment leakage rate test intervals (i.e. repeated pressure testing accelerates ageing), developing a rating and assessment of NPP concrete structure/component ageing;
- — Developing NDE techniques for inspection;
- — Establishing methods for evaluating the integrity of containment structure with grouted post-tensioning systems.

III.2.5. Canadian Nuclear Safety Commission (CNSC)

CNSC has developed REGDOC-2.6.3 [260], which recently replaced RD-334 [261] for ageing management of NPPs in Canada. Both documents are based on IAEA Safety Guide NS-G-2.12 [1] and identify ageing management requirements to be considered in the design, construction, commissioning, operation and decommissioning of NPPs. The licensee is required to develop, implement and maintain an AMP for the CCSs of NPPs. The AMP needs to meet the requirements of REGDOC-2.6.3 or RD-334 (depending on the operating organization's specific operating licence) for systematic and effective ageing management of the CCSs.

In addition, CCSs of NPPs in Canada are required to be inspected and pressure tested in accordance with Canadian national standard CSA N287.7 [249] requirements. The licensee of an NPP is required to develop, implement and maintain a period inspection programme meeting CSA N287.7 requirements for CCSs. The period inspection programme describes mandatory activities to be performed for inspection and testing of CCSs and their components, including concrete structures, caulking and sealant materials, liner material, the prestressing system, embedded parts and their attachments, among other things.

A CSA standard N290.3 [262] on requirements for the containment system of nuclear power plants is under preparation.

III.3. FINLAND

Finland has four operating nuclear reactors in two power plants. Reactors at Loviisa NPP, owned by Fortum Power and Heat Oy, went into operation in 1977 and 1980, respectively. Loviisa has two WWER-440/213 pressurized water reactors with a net power of 490 MW(e). Both units' operating licences have been renewed for a 50 year life: Loviisa-1 to 2027 and Loviisa-2 to 2030.

Teollisuuden Voima Oyj operates two identical 880 MW(e) BWR units, Olkiluoto 1 and Olkiluoto 2. The first unit was connected to the grid in 1978 and the second unit in 1980. In 1998, a 20 year operating licence was granted for both units.

At Olkiluoto 1 and 2, containment is a separate cylindrical concrete structure inside the RB and supports the reactor pools on top of it. The main structural parts are basemat on rock, outer prestressed cylinder wall, roof structures with pools and internal structures. A steel liner embedded in the concrete structure ensures containment tightness.

The Loviisa unit's RB forms a double shell containment resting on rock, which can be divided into three parts:

(a) Outer concrete shell without prestressing;
(b) Steel shell together with supporting structures;
(c) Internal concrete structures inside the steel shell.

The space between the outer concrete shell and inner steel shell is kept at a low vacuum.

With respect to ageing, concrete structures of interest are containment buildings and cooling water channels, as sea water is the coolant. In addition to ageing monitoring done by the power companies, the structures are studied and analysed by the Finnish Research Programmes on NPP Safety.

The YVL regulatory guide 1.8 [277] requires that power companies have administrative controls and related instructions for design, implementation and testing of repairs, modifications and preventive maintenance.

The power company needs to establish a programme for preventive maintenance of safety significant SSCs. The programmes need to describe SSC specific preventive maintenance and periodic inspection procedures, as well as their deadlines. The preventive maintenance programme needs to be based on guides and recommendations issued by the plant supplier and the suppliers of components and structures, as well as on the company's own experience. Programme efficiency needs to be periodically assessed based on internal experiences and experiences of similar foreign power plants.

According to the draft of regulatory guide YVL B.6 [279], a containment pressure test is to be performed prior to commissioning, using an overpressure that is at least 1.15 times that of the containment design overpressure. Containment leak tests are to be performed at regular intervals to ensure and monitor leaktightness throughout the service life of the plant.

III.4. FRANCE

As of 2013, France had 58 PWRs operating, supplying over 74% of its electricity. In France, no limit is set on plant operating licence periods, but every ten years the utility has to obtain a permanent renewal of its licence subject to numerous and continuous justifications (e.g. safety re-evaluation) [607]. Electricité de France (EDF), the operator, conducts R&D programmes and specific studies and also implements works on-site in order to extend the lifetime of each PWR as far as possible with respect to safety requirements. The French Nuclear Safety Authority, with the help of its technical support, the Institute for Radiological Protection and Nuclear Safety, checks every ten years whether safety requirements are met and gives permission for continued operation.

Over the last 30 years, several projects conducted by EDF (the operator) have studied and evaluated operating times of PWRs in France to determine what should be done to extend operation up to 60 years, including impacts of increased safety requirements, such as earthquakes and lessons learned from international events (e.g. Three Mile Island and Fukushima).

Starting in 1986, a systematic and continuous work programme (Operation Time Project) was implemented to evaluate the potential for technical and economic longevity of each of France's NPPs, and to take necessary improvement measures wherever possible, and to do so effectively while maintaining a satisfactory safety level [206]. The project remains running at EDF and addresses four topics:

(a) Ageing of materials;
(b) Impacts of severe accidents;
(c) Safety aspects;
(d) Economic aspects.

The project focuses on the major components that cannot be replaced and that are considered to be the most critical from a safety or economic point of view (e.g. containment building and reactor vessel).

The approach was to characterize ageing of each sensitive component and to address problems as they occurred. This approach was supplemented by a large database containing in-service component performance and experience derived from standardized designs.

Prior to commissioning a unit, its containment building is subjected to leakage rate and structural integrity tests to its design pressure. Tests are repeated after the first refueling (after about two years) and every 10 years thereafter.

CCBs are inspected for cracks every 10 years. Changes or evolutions can be identified if they occurred since the previous inspection. Every unit of each plant at a site is instrumented to measure ageing effects, such as concrete creep and shrinkage or relaxation of prestressing cables. Results are used in analytical and 3-D finite element studies [608] to verify that the concrete still meets safety requirements and to account for changes in material properties with time.

The instrumentation used includes:

— Vibrating wire strain gauges cast into the concrete in the dome near the junction between the wall and basemat at different elevations in the cylinder and in the basemat itself;
— Thermocouples to provide information on thermal gradients and temperatures where vibrating wire strain gauges are located (thermal corrections on the strain gauge measurements);

— Pendulums positioned at three elevations and 90° intervals to measure relative radial and tangential displacements;
— Invar wires located in the same positions as the pendulums to measure changes in vertical dimensions;
— Dynamometers (vibrating wire strain gauge based load cells) to measure prestressing forces in the four tendons that are ungrouted in the first plant unit;
— 20 hydraulic levelling pots (optical levels) to determine basemat tilt;
— Benchmarks for determination of basemat distortions.

Measurements are made at three month intervals unless an abnormal event occurs. The database contains data for more than 35 years for the oldest plants. This enables performance of these structures to be demonstrated, and also provides information for use in trending analyses and studies.

Two ageing related problems have been detected on CCBs:

(1) Corrosion of the 6 mm thick liner of the 900 MW(e) prestressed concrete containment vessels has occurred in several parts of containments in the CP (contract program) series. This has been repaired. Corrosion occurred in two areas:
   (a) Along the entire circumference of the conical portion of the liner adjacent to the upper portion of the concrete basemat;
   (b) Beneath the construction joints of the 1 m thick concrete base floor slab.

   Corrosion of steel liners was first noticed at around 10–15 years. It occurred over a 20 cm section of the liner extending from the water stop between the wall and the 1 m thick concrete slab over the liner toward the basemat (inaccessible region). Corrosion progressed to the state that 1 cm diameter holes penetrated the liner in this region. Although containments passed the integrated leakage rate tests, it was observed at the conclusion of the test that water containing corrosive substances were stagnating in some of the pressurization channels used during construction for inspecting the welds that join the liner plate sections over the basemat raft. Corrosion occurrence was attributed to a breakdown in the water stop in conjunction with the presence of high humidity during construction and operation. Holes in the liner were repaired by removing some of the concrete slab, sandblasting the liner, inspecting, welding plates over the holes and painting. In addition, the pressurization channels were filled with cement grout, the space between the liner and floor slab was filled with a corrosion inhibitor (wax) and a new water stop was installed. The new water stop consisted of a composite elastomeric material shielded by a bolted metallic sheet that can be periodically removed for inspection. Liner corrosion has also been observed at the bottom of a joint in the base floor slab (CP units only), penetrating through about 3 mm of the 6 mm thick liner. The liner corrosion was attributed to decomposition of the joint seal and the presence of water with pH5.

(2) In some prestressed concrete containments, tendon force vs. time curves obtained from the control ungrouted tendons showed larger than expected losses in prestressing forces. Losses were attributed to concrete creep and shrinkage, with one plant exhibiting twice the design value. Although prestressing force versus time results from these plants approached the design curves (lower tendon force limit), forces remained at acceptable levels, and the slope of the curves followed the same shape as obtained from plants having prestressing forces within the normal range. The force versus time curves flattened out and the embedded instrumentation showed that the concrete creep leveled off, and should ideally not intersect the lower tendon force limit during the desired service life of the plant. As the cause of the greater than expected losses of prestressing force are somewhat uncertain, modelling studies are periodically carried out to check the behaviour of the containments submitted to concrete creep and shrinkage (at least every ten years). Predicting the time dependent concrete response (creep in particular) is important because of its potential impact on the prestress level, which can possibly affect the containments' leaktightness. Since all but the four monitored tendons are cement grouted in French containments, re-establishing the required prestressing forces is impossible, which is why considerable efforts are still underway at EDF to develop methods to better predict concrete creep and shrinkage (e.g. R&D, laboratory tests and 3-D finite element studies) [608].

III.5. HUNGARY

Hungary has four NPP units of WWER 440/213 design at the Paks site. The approach at Paks for a service life extension (first application made in 2011) in conjunction with national regulations is similar to that employed via the US GALL. Ageing management programmes for structures and buildings have been set-up, a number of concrete samples have been taken and tested and a number of measures have been taken that were necessary to ensure appropriate plant operating life. These have included exclusion of moisture from concrete (coating repairs), reconstruction of concrete cover by injection and follow-up of steel plate corrosion via plate replacement and injection into cracks and voids located behind the plate.

III.6. INDIA

Civil engineering structures important to the safety of an NPP are required to be designed, constructed, tested and maintained to standards such that they perform their safety functions under postulated accidents and normal operating conditions throughout their service life [609]. Like all SSCs of an NPP, concrete structures are susceptible to ageing, which could eventually lead to impairment of their safety function. In India, suitable measures are taken relative to durability and ageing management of concrete structures to withstand the effects of various deteriorating processes, including those associated with decommissioning and extreme conditions due to the effects of both external and internal events during the life cycle of NPPs [610].

Licences of Indian NPPs are issued for their service life, which is generally 30–40 years. During the licensing process, aspects important to safety are assessed at stages, such as siting, design, construction, commissioning and operation. The Atomic Energy Regulatory Board (AERB) issues an initial licence authorizing operation for a specified period. Renewal of authorization for operation for a further period is issued after an assessment of safety performance of the NPP. A periodic safety review is carried out once in ten years of operation during the service life, in line with the guidelines given in AERB/SG/O-12 [611]. An application for life extension of the plant for the period of 20–30 years needs to be made at least 5 years before the end of service life and the requisite safety assessment is carried out following AERB/SG/O-14 [612]. Ageing management of the plant is one of the key elements for both renewal of authorization for operation and life extension of an NPP.

The objective of ageing management of the concrete structure of Indian NPPs is to predict and/or detect degradation and take appropriate corrective or mitigating actions if the extent of degradation erodes the safety margin to an unacceptable level. The concrete structures are passive type SSC. The strategy adopted in their AMP is not the same as that of active components. An AMP for concrete structures comprises two stages, pre-operational and operational, in line with the provisions of AERB/SG/O-14 [612] and AERB/SG/O-12 [611].

III.6.1. Pre-operational stage

As per Indian practice, the pre-operational stage, in relation to AMP, comprises siting, design, construction and commissioning. The objective of AMP of concrete structures in this stage is to design and construct durable concrete structures with required instrumentation for acquiring baseline data and monitoring structural performance during operation, as well as provide adequate provision for inspection, maintenance and repair/rehabilitation.

III.6.1.1. Design

Requirements under AERB/NPP-PHWR/SC/D [613] address ageing management during design. The following are major provisions related to concrete structures:

— The design is to provide appropriate margins for all structures important to safety so as to take into account the effects of relevant ageing and wear out mechanisms and potential age related degradation (during all normal and operational conditions, testing, maintenance, maintenance outages, postulated initiating events and post-postulated initiating event conditions) in order to ensure the capability of the structure to perform the required safety function throughout its service life.

— Provision also needs to be made for monitoring, testing, sampling and inspection, to assess ageing mechanisms predicted at the design stage for selected systems/components and to identify unanticipated behaviour or degradation that may occur in service.
— Required data need to be generated for the structures for ageing management and estimation of their residual life.

Provisions are specified in AERB/SS/CSE [610] for the safety design approach for postulated initiating events and design criteria for strength and serviceability. AERB/SS/CSE-1 [614] addresses design features for durability of concrete structures based on both internal and external environments and proper detailing to avoid detrimental things such as wide crack widths. AERB, 1998 [610] also specifies provisions for proper access and working space for inspection and maintenance and inclusion of instrumentation for monitoring structural integrity or other serviceability related parameters in the design stage.

AERB Safety Guide AERB/NPP-PHWR/SG/D-21 [586] for containment system design of PHWRs specifies requirements for addressing:

— Ageing effects (e.g. corrosion of metallic components, loss of prestressing force, reduction of resilience in elastomeric seals and shrinkage and cracking of concrete);
— Inclusion of provisions for monitoring ageing;
— Testing and inspection of components;
— Periodically replacing items that are susceptible to degradation through ageing, where possible.

Testing and inspection of containment structures is also covered.

III.6.1.2. Construction

The adopted construction methodology needs to ensure that the design intents of the buildings/structures are satisfied [610]. Factors considered include constituent material selection, concrete mix design, minimization of local climatic effects on the concrete properties and construction operations and quality assurance and inspection measures [615]. Delamination of the under surface of the primary containment dome of Kaiga Atomic Power Station, Unit-1 (Kaiga-1 dome) occurred during construction in 1994. A key component for re-engineering the delaminated dome was the use of silica fume based high performance concrete of characteristic compressive strength (cube) of 60 MPa and characteristic splitting tensile strength of 3.87 MPa [616]. Concrete mix design and construction techniques with silica fume were improved taking into account the experience of Civaux NPP. The M60 grade high performance concrete mix for the re-engineered Kaiga-1 dome was established by conducting a series of trial mixes [617]. Laboratory and field tests undertaken to finalize the mix have been reported [618]. Experience gained from re-engineering the delaminated dome was taken into account in the retrofitting provisions in AERB/SS/CSE [610].

III.6.1.3. Commissioning

Prior to plant commissioning, tests on some civil engineering structures are performed. Examples are pre-commissioning containment tests (structural integrity and leakage rate), a water retention test of the spent fuel pool and shielding adequacy testing. When any structure or structural component is tested, the test technical specification is developed prior to undertaking the test. The specification contains test objectives, detailed criteria and procedures formulated in line with structure safety functions [610]. Pre-commissioning test results contribute to baseline data of AMP for concrete structures at later stages.

III.6.2. Operational stage

During plant operation, activities pertaining to ageing management of civil structures are maintenance, in-service inspection and monitoring. These activities are carried out satisfying the requirements of AERB/SC/O [619]. Appropriate corrective or mitigating actions are taken if degradation erodes safety margins to unacceptable levels.

Effective maintenance of civil engineering structures ensures that the level of reliability and effectiveness of plant structures and buildings that have bearing on safety remain fit for operation (as per design intent), and the safety status of the plant is not affected due to the effect of the ageing process. Inspection for maintenance is a regular feature and is carried out at a higher frequency. Civil engineering structures of Indian NPPs are maintained per guidance given in AERB/SM/CSE-1 [620]. The document provides detailed guidance on aspects of civil engineering structure maintenance, including preventive maintenance, remedial maintenance, schedule of maintenance of civil engineering structures and modifications arising out of the maintenance programme, among other things.

The principal objective of in-service inspection is to periodically evaluate the physical condition and functionality of concrete structures/buildings, as well as their acceptability for continued safe operation, or to determine whether remedial measures are necessary. This may include a review of previously accomplished repairs, maintenance, as well as condition surveys, testing, structural analysis and consequential repairs. Detailed guidance on different aspects of in-service inspection are outlined in AERB/SM/CSE-2 [621] and include:

- Pre-service inspection;
- Schedule of in-service inspection;
- Supplementary inspection;
- Enhanced/augmented inspection;
- Acceptance criteria;
- Evaluation of inspection results;
- Repair/replacement.

In-service inspection, being very detailed, is carried out at a lower frequency and after the occurrence of abnormal events. If scheduled maintenance inspection coincides with that of in-service inspection, then both are performed concurrently.

Life extension of the Tarapur Atomic Power Station was granted in 2005. A detailed review of fitness of concrete structures including ageing management was undertaken. Overall, concrete structures were found to be well maintained with evidence of minor ageing effects. Some localized cracking and spalling of cover concrete, due to rebar corrosion, has been observed on the exterior faces of the reinforced concrete walls. Also, there have been some fine cracks observed in these walls and evidence of leaching. Ageing related effects were rectified.

III.7. JAPAN

At the end of 2012, Japan had 50 operational nuclear reactors, including the permanent shutdown of four units at Fukushima Daiichi. Of these, 26 are BWRs and 24 are PWRs.

In April 1996, the Ministry of International Trade and Industry (MITI) published a report [622], outlining basic concepts for nuclear power station ageing management which stated:

> "Ageing phenomena to be considered are strength deterioration and shielding capability reduction caused by various factors. It is recognized that elevated temperature, neutron and gamma irradiation, carbonation, chloride penetration, and alkali-aggregate reaction are the major factors that cause strength deterioration; and it also is recognized that neutron irradiation (exothermic by gamma ray) is the major factor that causes shielding capability reduction. Degradation such as cracking and spalling that may cause strength deterioration and shielding capability reduction have not been found by periodic visual observation of the concrete structures, including anchoring parts of support structures of components. It is important to understand and to confirm ageing trends by methods such as periodic visual observation and non-destructive testing."

After the Mihama pipe rupture accident in 2004 that killed a number of workers, a committee on ageing management was formed by the Nuclear and Industrial Safety Agency (NISA). Periodic safety review and ageing management evaluations have been regulatory requirements since 2005. At 30 years of life and at least every ten years following, ageing management reviews are to be conducted along with periodic safety reviews. NISA, now called the Nuclear Regulatory Authority (NRA), has published guidelines for such reviews, along with a standard review plan. The Japan Nuclear Energy Safety Organization (JNES) has published various age related technical evaluation review manuals including one on strength and shielding capability degradation of concrete [623]. Ageing management technical evaluations are done with an assumed service life of 60 years. Following the Fukushima Daiichi accident, the new nuclear safety regulations legally define the time limit for NPP operation as 40 years, with the possibility of a one-time extension by the NRA of no longer than 20 years (limit to be defined by a government order).

III.8. RUSSIAN FEDERATION

At the end of 2012, the Russian Federation had 33 operating NPPs, a mixture of PWRs (WWERs) and RBMKs, plus a fast breeder reactor. Most Russian power plants have or are extending their original service lifetimes of 30 years. WWER-1000 reactors are typically extended by 30 years (total operational lifetime of 60 years), and RBMK-1000 reactors by 15 years (total operational lifetime of 45 years).

The main Russian regulation related to NPP lifetime extensions is NP-017-2000, Basic Requirements for Nuclear Power Plant Unit Lifetime Extension. This regulation states that to extend NPP unit lifetime, the operator needs to:

— Carry out a comprehensive survey.
— Draw up a preparation programme for lifetime extension.
— Prepare the unit for operation during the extension period. This includes confirmation of component safety and residual service life, replacement of equipment which has reached the end of its service life, and where necessary, upgrade or reconstruction of the unit.
— Carrying out necessary tests.

An appendix of the above regulation provides more detail with respect to structural components, specifically that:

> "Residual service life of equipment, buildings, installations, structural components and foundations of NPP units which the General Regulations on Ensuring Safety of NPP Units classify as belonging to Safety Classes 1 and 2 must be demonstrated by means of methodologies drawn up and approved by the Operator and accepted by Gosatomnadzor (Rostechnadzor) in the prescribed manner".

In order to meet this requirement, the operator, together with Russian materials science and research organizations, has developed numerous standards, methodologies and guidelines, such as:

— STO 1.1.1.01.006.0327-2008, Nuclear Power Plant Unit Lifetime Extension;
— RD EO 0526-2004, Routine Requirements for Programme Content of NPP Unit Preparation to Lifetime Extension;
— RD EO 1.1.2.22.0283-2008, Typical Programme. NPP Unit Comprehensive Survey for Lifetime Extension;
— RB-027-04, Content and Composition of Report on NPP Unit Comprehensive Examination for its Operational Lifetime Extension;
— RB-029-04, Content and Composition of Documentation on Justifying Residual Lifetime of NPP Unit Elements for Operational Lifetime Extension;
— RD EO 0462-03, Methodology (Procedure) for Justification of Residual Service Life of Structural Components, Buildings and Installations of the RBMK-Type NPP;
— RD EO 0447-03, Methodology (Procedure) for Assessment of Residual Service Life of Safety-Significant Concrete Structures of NPP.

The assessment of concrete structure condition for long term operation is performed by analysing plant operational engineering documentation, examining concrete structure condition and performing structure check calculations. This includes:

— Analysis of engineering documentation (concerning design, operation and repair);
— Visual examination of concrete structure condition;
— Examination of concrete structure condition by instrumentation and analysis of results;
— Checking structure calculations (strength estimation).

In accordance with methodologies RD EO 0462-03 and RD EO 0447-03, a structural analysis of critical construction parts needs to be included as part of concrete structure condition examination. For this, a concrete sample is taken and analysed from the most strained (stressed) point.

Category I buildings provide a radiation shield for the population and the environment during both normal operation and accidents. Category II buildings ensure trouble-free operation. The residual service life of Category I concrete structures relevant to safety, and the biological shield, are estimated using mechanical concrete fracture methods that use physical concrete data. Residual service life of concrete structures relevant to safety belonging to Category II is estimated by comparing actual concrete structure condition with design and regulatory requirements.

Results of the above work are submitted as a document called, Statement/Findings of Assessments of Operating Conditions and Residual Service Life of Safety-Significant Concrete Structures of NPP, in which the following are presented:

— Assessments of operating conditions and methods used for confirmation of concrete structure residual service life;
— Date of next inspection or regulated repair;
— List of documents upon which the assessments were based;
— List of found defects;
— Recommendations for defect elimination/repair and condition of concrete structures for long term operation;
— Information on calculations and computer analyses (strength estimation) used for residual service life estimation.

III.9. SWITZERLAND

Operating licences for the five nuclear power reactors (2 BWR, 3 PWR) in Switzerland are, according to Swiss law, not formally restricted time limited licences. There is a requirement for systematic annual safety assessments and a comprehensive periodic safety review every 10 years.

The licensing authority, Swiss Federal Nuclear Safety Inspectorate (ESNI), formerly HSK, requires that utilities report periodically on their plant’s condition to demonstrate that safety requirements continue to be met. The approach used by utilities to meet this requirement has focused on preventive measures and immediate repair of any visible degradation. For reinforced concrete structures, this has included:

— Periodic visual inspections;
— Periodic testing of special structural elements (e.g. anchorages and joint sealants);
— Early and complete repair of any detected degradation.

Examples of problems experienced at Swiss NPPs include leakage of borated water from the fuel pool, failure of water stops, cracking of coating materials and cracking in the roof or dome of RBs. Because the older plants in Switzerland had been in commercial operation for over 20 years, in 1991, HSK required that all utilities develop AMPs addressing the structural condition as well as electrical and mechanical components for their plants [624].

In developing AMPs, HSK recommended:

— Identification of potential degradation modes and plant areas subject to degradation;
— Listing of periodic testing and maintenance programmes that are in effect;
— Assessment of data provided by state of the art inspection techniques;
— Cataloguing of additional measures for use in surveillance and assessment of the ageing process.

Initiation of these actions was via preparation of a state of the art report that defined the basic problem being addressed, reviewed world-wide experience and provided technical requirements for the AMP that is to be developed [625]. Report conclusions included the following:

— The most important degradation mechanism was rebar corrosion, with structural elements not accessible for inspection being of most concern (e.g. basemat and outer walls embedded in soil).
— Inspection methods used for general civil engineering reinforced concrete structures are also applicable to NPP structures.
— Plant service life is not likely to be limited by concrete structure degradation.
— Anchorage elements embedded in concrete need to receive special attention.
— Decommissioning needs to be included in service life predictions.

AMPs based on these guidelines are in place for each of the five NPPs.

III.10. UNITED KINGDOM

NPPs in the UK are operated by EDF Energy Nuclear Generation, Limited (part of EDF Energy plc), and Magnox Limited. In the year ending 2011, 18.8% of UK electricity generation [626] was from 9 nuclear power station sites consisting of 4 Magnox reactors, 14 advanced gas-cooled reactors and 1 pressurized water reactor. In 2013 there remained one Magnox reactor operational (at Wylfa).

The Nuclear Installations Act 1965 (as amended) [627] lays down legal requirements that must be satisfied as a condition for operation of a nuclear reactor. The Office for Nuclear Regulation (ONR) is the UK independent regulator for safety and security at nuclear facilities and for the transport of nuclear materials. The ONR is an agency of the Health and Safety Executive and discharges its regulatory functions by applying a rigorous permission and compliance inspection regime, which requires organizations wishing to carry out prescribed nuclear activities to apply for, and be granted, a nuclear site licence before they start construction of a nuclear power plant [628].

The licence holder must comply with the standard set of 36 site licence conditions [292] that require the licensee to make and implement adequate arrangements to address the matters identified. Each licensee can develop licence condition compliance arrangements that best suit its business, while demonstrating that safety is being managed properly. Similarly, the arrangements made to comply with licence conditions may change as the plant progresses through its life from initial design to final decommissioning. The licence conditions provide the basis for regulation by ONR, but the licensee retains absolute responsibility for nuclear safety. The licence conditions are generally non-prescriptive and set goals that the licensee is responsible for meeting by, among other things, applying detailed safety standards and procedures. ONR efforts are directed toward ensuring that appropriate expert and detailed assessments of design and construction are undertaken and that necessary precautions are taken to ensure that plants are operated safely. The ONR retains the power to close down a station, or prevent its startup, if it is not satisfied with any item of plant safety. A regulatory view of nuclear containment on UK licensed sites is given in Ref. [629].

Particularly relevant to ageing management is site Licence Condition 28, which requires that, “The licensee shall make and implement adequate arrangements for the regular and systematic examination, inspection, maintenance and testing of all plant [structures] which may affect safety.”

A nuclear site licence is granted for an indefinite period and, providing there are no material changes to the basis on which the licence was granted, it can cover the entire lifecycle of a site from installation and commissioning through operation and decommissioning to site clearance and remediation. The licence basis is, therefore, continually reviewed to consider plant suitability for continued operation.

Licence Condition 15 requires the licensee to make and implement adequate arrangements for the periodic and systematic review and reassessment of nuclear safety cases. The current system of periodic safety reviews every ten years covering concrete structures was developed from long term safety reviews (LTSRs) established for the Magnox reactors when the earliest reactors were reaching ages of 20 years. The objectives of LTSRs [630] were to:

- Confirm a plant is adequately safe for continued operation by examination of original design standards, operational history and plant modifications;
- Identify and evaluate any factors that may limit safe plant operation in the foreseeable future (i.e. plant ageing effects over the projected operating period and any 'cliff-edge' features just beyond that period);
- Assess plant safety standards and practices against modern standards and introduce any reasonably practicable improvements.

Currently, the principal civil engineering structures in operating UK power reactors are the prestressed concrete pressure vessels in later Magnox and AGR plants, and the prestressed CCV for the PWR. Other structures may have an impact on overall plant safety, however, and this influences their design and assessment requirements. Such structures, often referred to as safety related civil structures, include buildings housing critical plant components where structure failure could lead to consequential damage or active waste contaminants. These buildings and structures include spent fuel cooling ponds and supporting structures (e.g. crane platforms).

Continued fitness for purpose of containment and other safety related concrete structures has been demonstrated by an extensive system of monitoring, inspection and maintenance. Routine reports and assessments have enabled a pattern of behaviour to be recorded over time, which is used to support long term operation (thus supporting the periodic safety review) and gives confidence in a structure's continued ability to perform required safety functions. Reference [631] provides details of in-service monitoring of AGR and PWR safety related structures.

Inspectors from ONR use safety assessment principles (SAPs) [632], together with the supporting technical assessment guides (TAGs) [633], to guide regulatory decision making in the nuclear permission granting process. Underpinning such decisions is the legal requirement for nuclear site licensees to reduce risks so far as is reasonably practicable, and the use of these SAPs need to be seen in that context.

Research activities have targeted the UK's ageing stock of nuclear facilities, where it is recognized that continued operation relies on effective plant life management aimed at maintaining the integrity and safety function of safety related civil structures [629]. This work has included research in the following areas:

- Maintenance and improvement of analysis and design practices for assessing the integrity of existing (old) safety related structures;
- Identification and quantification of ageing, degradation mechanisms and time dependent phenomena relevant to the materials used to construct nuclear facilities (e.g. concrete reinforcement, prestressing tendons and concrete);
- Development and evaluation of techniques for establishing the current condition of structures and monitoring progress of any ageing mechanism (e.g. NDT of thick sections and corrosion monitoring);
- Evaluation of techniques for repairing or controlling the progression of damage to civil structures and monitoring international research activities.

It is recognized that the civil engineering industry already possesses considerable knowledge in many of the above areas. However, an important objective of the programme is the application of this knowledge to nuclear safety related concrete structures.

III.11. UNITED STATES OF AMERICA

III.11.1. In-service inspection and testing requirements

As of October 2013, there were 100 operational nuclear power reactors in the USA with three units under construction. In-service inspection programmes for NPP structures are intended to ensure structures have sufficient margins to continue performing in a reliable and safe manner [299, 634]. A secondary goal is to identify environmental stressors or ageing factor effects before they reach sufficient intensity to degrade structural components.

A condition of all operating licenses is that the primary reactor containment needs to meet containment leakage test requirements set forth in Appendix J of 10 CFR 50, Primary Reactor Containment Leakage Testing for Water-Cooled Power Reactors [299]. These requirements provide for pre-operational and periodic verification by tests of containment leaktightness, and establish acceptance criteria for such tests. The purpose is to ensure that:

- — Leakage through the primary reactor containment, and systems and components that penetrate primary reactor containment do not exceed allowable leakage rate values as specified in technical specifications or associated bases;
- — Periodic surveillance of reactor containment penetrations and isolation valves is performed so that proper maintenance and repairs are implemented in a timely manner.

Contained in Appendix J are requirements for Type A, B and C leakage rate tests that must be performed by each licensee as a condition of their operating licence. Type A tests measure the primary reactor containment overall integrated leakage rate after the containment has been completed and is ready for operation, and at periodic intervals thereafter. Type B tests detect local leaks and measure leakage across each pressure containing or leakage limiting boundary for primary reactor containment penetrations (e.g. penetrations that incorporate resilient seals, gaskets or sealant compounds and air lock door seals). Type C tests measure containment isolation valve leakage rates. Requirements for system pressure testing and criteria for establishing inspection programmes and pressure test schedules are contained in Appendix J to 10 CFR 50 [299].

In September 1995, the NRC amended Appendix J to provide a performance based option for leakage rate testing as an alternative to existing prescriptive requirements. Option A (prescriptive approach) requires that tests be conducted at approximately equal time intervals during each ten year service period. Option B (performance based approach) allows licensees with good integrated leakage rate test performance history to reduce the Type A testing frequency, on a plant specific basis, to one test in ten years. Recently, several plants with good integrated leakage rate test performance histories have been granted a relief request to extend the test frequency interval to 15 years.

Appendix J to 10 CFR 50 also requires a general visual inspection of accessible interior and exterior surfaces of containment structures and components to uncover evidence of structural deterioration that may affect either containment structural integrity or leaktightness. For conventionally reinforced (non-post-tensioned) concrete containments, general visual inspections currently are the primary in-service inspection required for concrete structures. Subsection IWL of ASME Code Section XI [634] addresses reinforced and post-tensioned concrete containments (Class CC). Two examination categories are provided in Subsection IWL. Examination Category L-A addresses accessible concrete surfaces and examines them for evidence of damage or degradation, such as cracks. Concrete is examined at 1, 3 and 5 years following the containment structural integrity test, and every 5 years thereafter. The primary inspection method of Category L-A is visual examination (general or detailed). Examination Category L-B addresses unbonded post-tensioning systems. The unbonded post-tensioning system examination schedule is the same as for concrete. For post-tensioned concrete containments, tendon wires are tested for yield strength, ultimate tensile strength and elongation. Tendon corrosion protection medium is analysed for alkalinity, water content and soluble ion concentrations. Prestressing forces are measured for selected sample tendons. Subsection IWL specifies acceptance criteria, corrective actions and inspection scope expansion when degradation exceeding the acceptance criteria is found. Additional requirements for inaccessible areas are specified in 10 CFR 50.55a(b)(2)(viii). Acceptability of concrete in inaccessible areas is to be evaluated when conditions exist in accessible areas that could indicate the presence of, or result in, degradation to such inaccessible areas.

Guidelines for evaluation of existing nuclear safety related concrete structures (other than containments), including acceptance criteria, have been developed by organizations such as the ACI [241]. Information on AMPs for NPP masonry walls [635, 636] and water control structures [305] is also available. Inspection requirements for concrete and steel containment liners are contained in Subsection IWE of ASME Code Section XI [634]. Acceptable editions and addenda of the ASME Code are identified in 10 CFR 50.55a.

III.11.2. Renewal of operating licenses

In the USA, the Atomic Energy Act and NRC regulations limit commercial power reactor licenses to a 40 year period and allow these licenses to be renewed for an additional 20 year period with no limit to these renewals. The original 40 year term for reactor licenses was based on economic and antitrust considerations, not on technical limitations. Due to this selected period, however, some structures and components may have been engineered on the basis of an expected 40 year service life. In order to ensure safe operation of NPPs it is essential that age related degradation effects of plant structures, as well as systems and components, be assessed and managed during both the current operating licence period as well as subsequent licence renewal periods [637].

In order to help ensure an adequate energy supply, the NRC has established a timely licence renewal process and clear requirements that are needed to ensure safe plant operation for an extended plant life. These requirements are codified in Parts 54 (License Renewal Rule) and 51 (Environmental Regulations) of Title 10, Energy, of the Code of Federal Regulations and provide for a renewal of an operating licence for an additional 20 years. The basic principles of licence renewal are (a) the regulatory process is adequate to ensure safety of all currently operating plants (with the possible exception of the detrimental effects of ageing), and (b) the plant specific operating basis must be maintained during the renewal term in the same manner and extent as during the original licensing term.

The scope of the rule (10 CFR 54 [34]) includes:

— Safety related SSCs that are relied upon to maintain integrity of the reactor coolant pressure boundary, to ensure capability to shut down and maintain a safe shutdown condition, and to prevent or mitigate offsite exposures comparable to 10 CFR 100 [638];
— Non-safety related SSCs whose failure could prevent safety related functions as noted above;
— SSCs relied upon for compliance with regulations (i.e. fire protection, environmental qualification, pressurized thermal shock, anticipated transients without scram and station blackout).

The licence renewal application (LRA) identifies reactor SSCs that would be affected by licence renewal, demonstrates that it can manage the adverse effects of ageing during the renewal period and analyses environmental effects of extended reactor operation during the renewal term [34]. Applicants wishing to submit an LRA are responsible for preparing a plant specific LRA that includes both general information, similar to that provided with the initial plant operating licence application, and technical information, including an integrated plant assessment (IPA) (10 CFR 54.21(a)(1)), TLAAs (10 CFR 54.3), a supplement to the final safety analysis report and technical specification changes (10 CFR 54.22).

The IPA identifies and lists structures and components subject to an ageing management review (AMR) that perform intended functions without moving parts or without change in configuration or properties (passive), or that are not subject to replacement based on a qualified life or specified period (long lived). Containments and seismic category I structures are identified as components subject to an AMR. Intended functions are those that in-scope SSCs must be shown to fulfil that would form the basis for including the SSCs within the scope of the rule. Methods used to identify SSCs subject to an AMR are to be identified. Finally, the applicant must demonstrate that ageing effects will be adequately managed so that their intended function(s) will be maintained consistent with the continuing licence bases for the period of extended operation.

In addition to the rule (10 CFR 54 [34]), basic documents providing regulatory requirements and the regulatory framework for ensuring continued plant safety for licence renewal include:

— Regulatory Guide 1.188, Standard Format and Content for Applications to Renew Nuclear Power Plant Operating Licenses [35];
— Generic Aging Lessons Learned (GALL) Report [306];
— Standard Review Plan (SRP) for Review of License Renewal Applications for Nuclear Power Plants [71].

These guidance documents are living documents, which are periodically updated. Additional sources of guidance related to licence renewal include:

— Inspection manual chapters;
— Inspection procedures;
— Information notices;
— Licence renewal interim staff guidance;
— Regulatory guides;
— Office instructions;
— Nuclear plant ageing research reports;
— Technical reports in NUREG series.

Provided in the following is a brief description of the GALL Report that is central to an evaluation of the concrete structures for continued service. Information on the other guidance documents is available on the NRC web site (http://www.nrc.gov/reactors/operating/licensing/renewal/guidance.html).

The GALL Report lists generic ageing management reviews (AMRs) of SSCs that may be in the scope of LRAs and identifies AMPs that are determined to be acceptable to manage ageing effects of SSCs in the scope of licence renewal as required by 10 CFR 54 [34]. The GALL Report provides a technical basis for the licence renewal SRP and contains the NRC staff's generic evaluation of existing plant programmes and documents the technical bases for determining where existing programmes are adequate without modification and where existing programmes need to be augmented for the extended operation period. Each structure and/or component is identified, as are the materials of construction, the environment, ageing effects and mechanisms and the acceptable programmes to manage the effects of ageing and a determination is made if further evaluation is required. The adequacy of generic AMPs in managing certain ageing effects for particular structures and components is based on a review of ten programme elements of an AMP for licence renewal [71] including:

(a) Scope of programme;
(b) Preventive actions;
(c) Parameters monitored or inspected;
(d) Detection of ageing effects;
(e) Monitoring and trending;
(f) Acceptance criteria;
(g) Corrective actions;
(h) Confirmation processes;
(i) Administrative controls;
(j) Operating experience.

The GALL Report contains 11 chapters and an appendix [306]. Chapter I addresses application of the ASME Code [634] to licence renewal. Some ageing AMPs referenced in the GALL Report are based entirely, or in part, on compliance with the requirements of ASME Code Section XI, Rules for Inservice Inspection of Nuclear Power Plant Components. Chapters II through VIII contain summary descriptions and tabulations of AMPs evaluation for a large number of structures and components in major plant systems found in light water reactor NPPs. Chapter IX contains definitions of a selection of standard terms used within the GALL Report. Chapter X contains TLAA evaluations of AMPs under 10 CFR 54.21(c)(1)(iii).

AMPs related to NPP containment structures are identified in Chapter XI. This chapter provides a description of each of these programmes as well as the evaluation and technical basis related to a review of the ten programme elements of an AMP for licence renewal identified in the SRP-LR [71]. AMPs related to concrete containment include ASME Section XI, Subsection IWL (GALL AMP XI.S2) and structures monitoring (GALL XI.S6).

AMPs related to post-tensioning systems include ASME Section XI, Subsection IWL (GALL AMP XI.S2) and concrete containment tendon prestress TLAA (GALL TLAA X.S1).

AMPs related to liners of reinforced concrete containments (and steel containments) include: ASME Section XI, Subsection IWE (GALL AMP XI.S1), 10 CFR Part 50, Appendix J (GALL AMP XI.S4) and Containment Liner Plate, Metal Containments and Penetrations Fatigue Analysis Time-limited Ageing Analysis (see SRP-LR [71] section 4.6.1.1).

III.11.3. Operating experience

As part of the licence renewal process, the NRC conducts an AMR that includes an assessment of proposed programmes to manage ageing of structures and components. An element associated with this assessment addresses OPEX, and for concrete structures this includes a walkdown of areas of interest (e.g. buildings, SFP, supports, masonry walls and water control structures). Occurrences of age related degradation have been observed for NPP concrete structures and containments. Tables 7 and 8 show degradation mechanisms that can impact the performance of concrete structures and concrete problem areas in NPPs. Some comments are provided in the following relative to operating experience in the USA. More detailed information on US NPP concrete structure degradation is available in Refs [9, 221, 639].

As a result of the maturity of US NPPs, degradation due to initial construction, materials selection or design problems has been addressed; however, degradation occurrences related to environmental effects are likely to increase. Primary factors that have led to concrete containment degradation include:

— Corrosion of embedded steel reinforcement and post-tensioning systems (i.e. anchor head fractures and wire breaks);
— Groundwater intrusion into the containment building and tendon galleries with associated leaching of the concrete;
— Larger than anticipated loss of tendon prestressing forces;
— Concrete cracking and spalling due to freeze-thaw cycles.

The primary degradation factor that has produced degradation in metal liners of reinforced concrete and steel containments has been corrosion due to water presence, in some cases in the form of boric acid. Water in inaccessible areas leading to degradation of coatings and corrosion of steel liners of reinforced concrete and steel containments has been an ageing concern.

Appendix IV

DOCUMENTED CONCRETE RELATED EVENTS AT NUCLEAR POWER PLANTS

This Appendix contains a listing (Table 34) of sample events that have been documented related to concrete at NPPs. It is not intended to be a comprehensive list but rather a sampling of the issues encountered in Member States. The table is sorted by country and NPP name.

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS

Plant	Problem area	Remedial measure implemented	References
Doel 3, Belgium	Defective electrical resistance strain gauges inside concrete.	New vibrating wire strain gauges to be installed on external face of shell.	[640]
Doel 3, Belgium	Shell outer face incurring too fast a carbonation rate (penetration of carbon dioxide gas in the concrete). This phenomenon can lead to a change in concrete alkalinity that protects the reinforcements. With time, reinforcements can be corroded.	Propagation of carbonation arrested by application of coating that prevents carbon dioxide gas penetration into the concrete. Impervious coating for the inner surface so as to avoid ingress of water vapour into, and diffusion within, the concrete, as this would induce paint flaking on the outside.	[641]
Tihange 3, Belgium (1998)	Concrete on external side of shell of Tihange 3 cooling tower became ochre coloured. Percolation formed a permanent wet patch presenting traces of calcite, several hundred square metres covering first 15 casting rings (each 1.5 m high). Percolation observed in wet area resulted from the high porosity of concrete.	Inner waterproofing coating applied. Outer face left as is so not to mask a possible evolution.	[642]
Tihange 3, Belgium (2002)	Partial plant flooding due to clogging of cooling tower outlet induced by concrete beam rupture. Two horizontal concrete beams ruptured and fell. Overload from mud and organic fragments and poor design led to debris accumulation at tower outlet and overflow of cooling tower basin. Tower had been designed to be self-cleaning. Designer had accepted less concrete reinforcement than initially foreseen.	Plant flood protection measures strengthened. Cooling tower internal structures inspected. Basin high level alarm installed. Cooling tower packing replaced/cleaned.	[641, 642]
Pickering A, Canada	Cracks in vacuum building floor due to premature exposure to cold temperature during curing.	Epoxy grout injection.	[606]
Pickering A, Canada	Polysulphide sealant in vacuum building floor joints deteriorated due to attack by sulphate reducing bacteria contained in plant water.	Joints replaced with different formulation of sealant.	[606]
Pickering A, Canada	Epoxy liner in spent fuel storage bay damaged by local overexposure to radiation.	Not structurally significant.	[606]
Pickering A, Canada	Corrosion of reinforcing steel in one vacuum building column due to insufficient cover/tie wire exposure.	Affected concrete removed and area patched with new concrete.	[606]
Pickering A, Canada	Shrinkage cracks in RB domes lead to increased leakage rates with time (detected during RB pressure testing).	After 20 years monitoring, structural evaluation conducted and polyurethane overlay applied to part of dome of one building.	[606, 643]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Pickering B, Canada (2003)	Cracks in Unit 8 main output transformer concrete foundation discovered and subsidence of soil beneath slab. Believed to be caused by episodic collapse of voids that had formed within fill beneath slab (due to ongoing erosion) and subsequent subsidence.	Monitoring programme for slab subsidence initiated. Cementitious grouting programme performed to increase soil stability and reduce settlement rate. Transformer connections modified to account for dimensional changes.	[568]
Bruce A, Canada	Steel intake roof structure damaged due to overload.	Concrete slab applied to strengthen roof.	[606]
Bruce A, Canada	Steel intake roof structure corroded due to aggressive lake water.	Cathodic protection system applied, coupon monitoring programme implemented (subsequent structures protected by coating).	[606]
Bruce A, Canada (1981)	During vacuum building roof seal inspection, it was found that 4 of 32 tendons had failed (fallen off), and that the rubber seal splice had deteriorated. Causes were poor choice of materials, as well as poor protection from weather conditions, resulting in cable corrosion which led to failure.	All 32 tendons, as well as the rubber seal, replaced. Tendons covered with a more appropriate type of grease. The inspection frequency increased.	[644]
Gentilly 1, Canada	Cracking and degradation in RB of a permanently shutdown NPP that was in storage with surveillance phase of decommissioning due to AAR and freeze-thaw cycling. Secondary (non-structural) concrete protecting prestressing anchorages in ring beam at top of containment structure severely affected.	Secondary concrete had to be removed and replaced, followed by wrapping with a glass fibre reinforced polymer fabric.	[645]
Gentilly 2, Canada	Increasing leakage rates through RB containment walls due to presence and increase in number and width of cracks in epoxy liner over time due to concrete cracking.	Portions of epoxy liner recoated with polyurethane.	[646]
Temelin 1, Czech Republic (2007)	Complete loss of prestress on both ends of one dome prestressing cable.	Corrective actions have to be directed against creation of wire crossings on eye ring surface and against all interactions that consume wire plastic deformation capacity.	[647]
Bugey 4, Cruas 2 and other 900 MW(e) Units, France (1992)	Containment liner corrosion along circumference adjacent to upper portion of concrete basemat; attributed to breakdown in waterstop and presence of high humidity during construction and operation.	Repaired by removing part of concrete slab, sandblasting liner, inspecting, welding plates over areas of significant material loss and painting; new waterstop design developed.	[557]
Various 900 MW(e) Units, France	Tendon force versus time results showing larger than expected loss of prestressing force.	No immediate problem, prestressing forces well above design minimums, rate of force loss decreasing with time, modelling studies of concrete creep and shrinkage being conducted.	[557]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Various 900 MW(e) units, France	Corrosion of some prestress tie rods of reactor pit lateral supports and loose nuts.	Increased inspections, tighten loose nuts and redo calculations underpinning safety report to show that safe shutdown earthquake criterion can be met.	[648]
Nogent, France (1993)	Leak in underground auxiliary pipework of essential service water (concrete pipe) caused by differential subsidence between civil engineering structures and poor installation and design practices.	Extent of condition investigation performed for other French plants (three similar cases discovered), instrumentation installed to monitor movement, traffic (heavy vehicle) restrictions over area put in place, pipe repairs and reinforcements.	[649]
Cruas Unit 2, France (1990)	Spill of 110 m^3 of water in nuclear auxiliary building risked groundwater contamination. Flooding affected many rooms and contaminated lower rooms. Faults uncovered included sealing band not adherent to concrete, damage to seal band from objects such as metal plates lying around during construction, concrete cracks allowing water to seep around the seal.	Repair techniques developed including: Injecting gel into concrete to fill cracks. Sticking portions of rubber over damaged areas of the sealing band. Campaign of controls of seals between buildings for all 900 MW(e) and 1300 MW(e) reactors in France initiated.	[650]
Gravelines, France (1984)	Leak from stainless steel spent fuel liner (likely from microcrack in weld) possible damage to concrete structure behind liner.	Experimental programme initiated to determine cause(s) and impacts of borated water on concrete (behind the pool liner). Impacts appear only in localized area along leak path. Improvements made for future designs.	[651]
NPP not identified, France (1998)	Essential service water pipe rupture flooded piping gallery with sea water. Rupture was 1 m × 3 cm long in a concrete pipe. Prestressing wire and steel cylinder were found severely corroded. Condensation had entered a small surface crack.	Extent of condition review found 16 other pipe spools that required replacement.	[652, 653]
Gravelines 2, France (2010)	Containment building: detection of a limited poor concrete zone during a maintenance inspection.	Analysis to determine that the building was still functional and safe; calculations to evaluate whether this zone of poor concrete was or was not acceptable for safe long term plant operation; work initiated to repair area and to prevent further ageing degradation.	[654]
Various 1300MW(e) and 1450 MW(e) Units, France (1998)	Containment building shrinkage and creep measurements versus time shown to be larger than postulated in design for some units.	Not an immediate problem as force loss rate decreasing with time. Modelling studies of concrete creep and shrinkage are conducted periodically to demonstrate that the building is still in a safe state. Repairs implemented as necessary for long term operation.	[654]
Obrigheim, Germany (1991)	Humidity penetrated into insulation on inner sides of containment, causing corrosion of containment steel shells and of insulation coverings made of galvanized sheet metal.	Corroded spots of containment steel shells were reconditioned and provided with new corrosion-preventing coatings.	[655]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Tarapur, India	Localized cracking/spalling of cover concrete and initiation of rebar corrosion at a few locations on exterior faces of secondary containment RCC walls. Narrow diagonal cracks on exterior as well as interior faces of RCC walls at one location, leaching marks at one location on interior face of external wall. Tarapur is a BWR based plant with a primary containment pressure suppression system. Secondary containment does not play a significant safety role as such. Leakage rate testing is carried out per technical specifications and secondary containment has met all requirements to date.	Tarapur management advised to conduct condition survey (i.e. mapping of all deterioration with description of nature and extent, and monitoring of typical cracks). Based on condition survey findings and monitoring, an assessment of the structure can be carried out.	[656]
Kaiga 1, India (1994)	Delamination of inner (primary) containment dome of Kaiga NPP Unit-1 occurred during construction. Under surface of dome in central portion delaminated and fell down completely. Failure was due to action of radial tension coupled with the effect of membrane compression that was higher than the tensile load carrying capacity of dome concrete in a radial direction. Evidence of too close spacing of reinforcement and honeycombs/voids in concrete.	Delaminated dome was subsequently rehabilitated (redesigned and reconstructed).	[616]
Ikata 1, Japan	Expansion produced by alkali–silica reaction has been observed in the turbine generator foundation. The foundation is a reinforced concrete frame structure. Abnormal movements were detected between generator, rotor and their bearings during the first annual inspection in 1979. ASR cracks were observed and confirmed by core sampling. The foundation had expanded in the longitudinal direction with the maximum expanded rate being about 50 mm.	TG foundation confirmed to have sufficient structural soundness. Recommended to repair exterior cracks to prevent reinforcement corrosion and continuous monitoring of the TG foundation.	[141, 142]
Hanul-1 (originally Uljin), South Korea (1998)	Shutdown caused by 1.7 m × 7 cm crack of essential service water system concrete pipe during testing that flooded the essential service water gallery with sea water.	Filled fine cracks discovered on horizontal and vertical direction along essential service water (SEC) pipe with epoxy material. Painted SEC pipe to prevent any corrosion. Installed manhole for inside inspection of SEC pipe. Revised inspection programme.	[657]
Borselle, Netherlands (2011)	Ageing related degradation of cooling water lines. Surveys indicated degradation of main cooling water system. Indications included subsidence of pipelines, corrosion of steel reinforcements, erosion and damage of concrete cladding of pipeline inner and outer surfaces, small leaks, deformation and cracks of profiles used as joints. Gradual degradation of cooling water intake piping appeared, caused by ageing as well as by deficiencies in original design and construction.	Combination of temporary repair, systematic testing, measurements and on-line monitoring of main parameters demonstrating condition of piping and sand bed. Construction work to reach a final solution by an ongoing total upgrade and reroute of cooling lines.	[658]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Vandellos 2, Spain (2004)	Plant experienced essential service water system manhole pipe break that caused plant shutdown. Pipe was a buried BONNA design using steel with concrete on both sides. Manhole had filled with surface water and corroded exposed carbon steel pipe neck (which was unprotected).	Extent of condition review found two other similar cases, all of which were repaired by installing new pipe or by adding a reinforcing concrete collar around the manhole neck.	[652, 659, 653]
Barsebeck 2, Sweden (1993)	Leakage was found close to an electric cable containment penetration. Concrete refill did not fill out the intended volume and trapped water produced porous concrete quality, which together with high water content facilitated local corrosion.	Repair made.	[157, 660, 576]
Forsmark 1, Sweden (1997)	Cracking of steel liner welds at bottom of reactor pool situated above the toroid permitted pool water to leak into mineral wool, where it was trapped adjacent to the toroid by plastic film. Corrosion started in an area where epoxy paint had been damaged; paint damage due to higher than expected temperatures in the area.	Pool liner welds repaired by welding. Inner plate repaired via welding. Ventilation system installed to ensure a dry air space. Relative humidity is monitored for changes.	[576]
Ringhals 2, Sweden (2004)	Leakage from toroid plates, which are two parallel double bent plates connecting metallic liner in cylindrical wall to liner in ground concrete plate of containment. Corrosion defects (holes) were found in almost all parts of inner toroid plates. Construction was found not in accordance with issued documentation, and corrosion was owing to an unfavourable environment caused by a sediment layer.	Both toroid plates replaced. Residual water leakage measured on a regular basis and reported to regulator.	[661]
KKG Gosgen, Switzerland	Minor cracks in outer dome shell detected by visual inspection.	Remote crack measurement vs. time (data logging) to determine appropriate protective coating.	[662]
KKL Leibstadt, Switzerland	Extensive microorganic vegetation on dome causing dark areas.	Detailed investigations showed limitation of effect to 2 mm depth, no further action taken.	[662]
Arkansas 1, USA	Triangular wood wedge found in containment structure (not ageing related, construction error).	Void grouted flush to surface to prevent water intrusion and long term degradation.	[157]
Arkansas 1, USA	Exposed rebar found below personnel airlock (not ageing related, construction error).	Rust scale removed and epoxy coating applied.	[157]
Beaver Valley 1, USA	Cracks, water infiltration and calcium deposits in ceilings and walls of service water building.	No report.	[221]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Beaver Valley 1, USA (1982)	Void ~0.9 m long and 0.9 m deep in outer containment wall in concrete ring around equipment hatch.	No threat to structural integrity, void repaired with dry pack.	[81]
Beaver Valley 1, USA (2006)	Three areas of corrosion in containment liner uncovered during creation of temporary construction opening in containment structure for steam generator replacement. Measured wall loss between 1.1–5.8 mm. Probably caused during construction phase, possibly by foreign material.	Affected liner plate sections replaced.	[157]
Beaver Valley 1, USA (2009)	Steel containment liner thinning within a 10 in (25.4 cm) square area discovered on Unit 1 following investigation of a 3 in (7.6 cm) paint blister on the liner. Thinning was caused by pitting type corrosion (rust) originating from concrete side that was caused by foreign material (wood) left behind during original construction (early 1970s). This wood was in contact with the containment carbon steel liner, decomposed over time, and initiated liner rusting.	Removed embedded wood, grouted concrete area that was displaced by the wooden debris, and welded steel plate to replace damaged concrete liner section. One hundred percent of accessible liner area inspected on Units 1 and 2 during next refuelling outages.	[157, 663]
Bellefonte 1, USA (2009)	RB containment vertical tendon experienced failure of rock anchor/tendon coupling due to hydrogen induced stress corrosion cracking. Cause of failure was determined to be water containing sulphides in contact with grease surrounding coupling in a high stress area.	No report.	[221]
Bellefonte 1/2, USA (1984)	Expansion shell anchor failures occurred in control building concrete due to low surface concrete strength.	Anchors replaced by more deeply embedded bolts or grouted anchors.	[81, 664]
Bellefonte 1/2, USA, (1975/76)	Eight rock anchor heads failed during construction because of possible stress corrosion cracking.	Anchor heads replaced with cleaner steel.	[81, 221, 664]
Braidwood 2, USA	Localized bulges and buckling at multiple locations on containment liner.	No indications existed that degradation was present.	[157]
Brunswick 1/2, USA	Area around service water intake subject to aggressive environment due to high chlorides and sulphates in intake water.	Increased monitoring frequency of concrete structures. Repairs made as required.	[221]
Brunswick 1/2, USA (1974)	Voids occurred behind liner during construction of suppression chamber.	Grout injected into voids through holes drilled in liner; some grout to Unit 1 did not harden but was left in place to provide limited resistance.	[81]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Brunswick 2, USA (1999)	Corrosion identified at 5.5 m and 21.3 m elevations at Unit 2 where pitting corrosion had penetrated drywell liner as a result of a break in protective coating film. Through wall defect originating from concrete side identified at 17 m elevation resulting from debris (cloth work glove) that wicked moisture to back of liner plate and provided a collection point for oxygen.	Damaged areas of liner repaired and resealed.	[157]
Brunswick 1/2, USA (2004/2008)	Many areas of thinned containment penetration sleeves (below minimum wall thickness) uncovered around personnel airlock, apparent via bulges observed during visual inspections. Outside sleeve wrapping (felt) designed to allow for thermal expansion apparently had become wet during original construction and initiated corrosion product buildup between the sleeve and the concrete backing.	Installation of new concentric penetration sleeve to repair current sleeve on U1 (new sleeve became new containment boundary). Weld overlays for local wall thinning and pitting repairs.	[157, 663]
Byron 1, USA (1979/80)	Four anchor head failures occurred in first year after stressing, cause was use of vanadian grain refinement process in conjunction with temperatures not high enough.	Anchor heads replaced.	[81, 664]
Callaway 1, USA (1977)	Nineteen randomly located areas of honeycomb extending to bottom layers of rebar of RB basemat in annular area of tendon access area, cause was use of low slump concrete in congested area.	Defective material removed from 33 of 172 tendon trumplates and voids repaired.	[81]
Calvert Cliffs 1/2, USA	During 20th year surveillance of the prestressing system, low lift-off values were found for vertical tendons. Inspections of adjacent tendons revealed broken tendon wires. Cause was attributed to hydrogen induced cracking.	About 30% of vertical tendons in Units 1 and 2 replaced.	[73, 221]
Calvert Cliffs 1/2, USA (1971/72)	Eleven top bearing plates at Units 1 and 2 depressed into concrete because of voids, 190 plates of each retensioned containment exhibited voids upon further inspection.	Tendons detensioned, plates grouted and tendons retensioned.	[81]
Catawba 1/2, USA (1976)	Cement used in reactor building base slab had been contaminated by fertilizer, 7 day strengths exceeded 28 day design values.	Cement feed transferred to another silo.	[81]
Catawba 1/2, USA (1999)	Coating damage and base metal corrosion at interface between shell and annulus floor. Cause believed to be boric acid attack that had leaked from instrument line compression fittings.	Not reported.	[157]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Clinton 2, USA (1984)	Embed plate on outside of drywell wall pulled from concrete because of failure of several 'Nelson' studs occurring as result of weld shrinkage.	Concrete excavated along plate edges, ebed plate redesigned and grout placed into area where concrete was removed.	[81]
Columbia, USA (2002)	During performance of a fire protection system surveillance test, water spilled from a firewater drain through cracks in a concrete floor and a spalled area around a penetration seal into a Division II switchgear room. Thousands of similar small cracks were observed on the cable spreading room floor. Licensee determined cracks in concrete floor resulted from flexing and shrinkage of floor, and spalled areas were caused by insufficient distance between concrete anchors installed as part of original construction.	Additional inspections were performed and all appropriate unsealed concrete floors were sealed with an approved flexible epoxy coating.	[665]
Comanche Peak 1/2, USA (1975/76)	Cold joint formed in reactor mat. Inadequate concrete compaction under containment wall for 58 m at 1.8–2.1 m below top of mat, 3.7 m by 6.1 m area south of reactor pit, 1.4 m by 3.1 m area south of north sump, and 1.2 m by 1.8 m area north of north sump.	Concrete removed, rebar exposed and new joint poured. Core holes drilled for inspection in conjunction with evaluations revealed basemat was adequate for all loading conditions, core holes filled with mortar and interconnecting voids grouted.	[81]
Cooper, USA	Cracking and spalling of concrete in service water booster pump room and in exterior walls of diesel generator and RBs.	No report.	[238]
Crystal River 3, USA (1974)	Twenty-eight day concrete tensile strength was low due to failure of cement to meet specifications.	Design review revealed strength attained to be adequate, cement inspection increased.	[81]
Crystal River 3, USA (1976)	Dome delaminated over ~32 m diameter area due to low concrete properties and no radial reinforcement to accommodate radial tension due to prestressing.	Upper delaminated section removed, additional rebar provided, concrete replaced, dome retensioned and structural integrity test conducted.	[81]
Crystal River 3, USA	Bulging of liner plate observed at several locations with hollow sounds during hammer tapping.	Detailed visual examinations performed with results indicating satisfactory coating integrity and absence of deterioration. Ultrasonic thickness measurement results indicated liner thickness was above minimum design thickness.	[666]
Crystal River 3, USA (1997)	Hairline crack on the concrete wall of the SFP was noted. The hairline crack was reinspected after 10 years, in 2007, and found to be dry and stable.	Preventive maintenance periodically conducted (i.e. snake runs) to verify that each of the 19 leak chases is clear. Sampled deposits are analysed for products of concrete degradation. Hairline crack reinspected after 10 years, (in 2007) and found to be dry and stable, and is reinspected yearly.	[221]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Crystal River 3, USA (2003)	Leaching was identified in the tendon gallery.	Concrete core samples taken up to level of rebar and examined for water soluble chlorides and rebar examined visually for corrosion.	[221]
Crystal River 3, USA (2007)	Several prestressing tendon surveillances produced lift-off forces in hoop tendons that were consistently below ninety-five percent predicted values. Cause was excessive steel relaxation due to elevated temperature.	Adjacent tendons examined and results considered to still meet acceptance criteria.	[221]
Crystal River 3, USA (2009)	During work to cut an opening for steam generator replacement, delamination of containment wall observed, and as tendons were being retensioned following repair, additional delaminations occurred. Cause of cracking was sequence and scope of detensioning steel containment tendons and absence of radial reinforcement. Plant was decided to be permanently shut down due to potential cost of repair.	Not an ageing related incident. Delaminated concrete was planned to be repaired via removal and replacement of delaminated part of containment wall. Licensee's repair plans had included: (a) Additional detensioning of containment; (b) Removal of delaminated concrete; (c) Installation of reinforcement, including radial reinforcement through the delamination plane; (d) Placing of new concrete; (e) Retensioning containment; (f) Post-repair confirmatory system pressure testing Licensee had developed new analytical methods to identify redistribution of stresses in containment wall and identify an acceptable expanded detensioning scheme to perform the repair. The company however decided to permanently shut down the facility.	[221, 667, 237]
D.C. Cook, USA (1974)	Cracking in spent fuel pit wall and slabs framing into pit walls, cause was thermal expansion and hydrostatic pressure, no threat to structural integrity.	None documented.	[81]
D.C. Cook, USA (1998)	Pitting corrosion of containment liner at moisture barrier seal where liner becomes embedded in concrete. More than 60 areas where thickness was below design minimum (Unit 1 and 2).	New liner to floor moisture barrier seal installed.	[157]
D.C. Cook 2, USA (2001)	Through wall hole in containment liner plate having diameter of about 0.47 cm on exterior surface and 1.9 cm on interior surface. Removal of liner plate revealed a piece of wood (wire brush with wooden handle) embedded in concrete. Hole appeared to be human-made.	Area repaired, new liner section installed, and local leak rate test conducted.	[157]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
D.C. Cook 1, USA (2002)	Three pieces of wood and one piece of plastic shallowly embedded in CCB.	Structural integrity evaluated as non-affected.	[157]
Davis Besse 1, USA (1982)	Two concrete expansion anchors and upper part of base plate pulled from wall approximately 1 cm because of improper installation.	Anchors replaced and torque checked.	[81]
Davis Besse, USA (2001)	Six SFP leak chases were found to be totally blocked. Upon clearing of their blockage, a significant amount of trapped fluid was drained.	Leak chase channels exhibiting the largest drainage kept constantly open, with the rest closed to reduce likelihood of boric acid solidifying and blocking the valves and piping.	[221]
Davis Besse, USA (2002)	Corrosion identified where containment meets floor.	UT examinations done to confirm minimum design thickness. Moisture barrier installed at containment to floor junction.	[157]
Davis Besse 1, USA (2010)	Auxiliary feedwater pump turbine exhaust missile barrier has spalled concrete and exposed rebar due to its periodic exposure to a harsh environment.	Evaluation determined missile barrier continues to perform its design function and repair scheduled.	[221]
Davis Besse 1, USA (2010)	Several tower and disconnect switch concrete foundations in switchyard are degraded to the point that concrete has spalled and rebar is visible. Moisture accumulation occurred because of insufficient drainage.	No report.	[221]
Davis Besse, USA (2011)	Discovery of laminar subsurface cracks in the reinforced concrete shield building while performing hydrodemolition operations in support of reactor vessel head replacement. Causes of cracking determined to be related to: environmental factors associated with a 1978 blizzard, lack of an exterior moisture barrier, and structural design elements of shield building. Weather records, core boring sample results, impulse response testing, and shield building analytical modelling provided a sufficient basis to support causes of laminar cracking.	Design and implement a shield building exterior sealant system/external moisture barrier and an associated routine inspection programme.	[668, 669]
Diablo Canyon 1/2, USA (1996)	Seawater intake structure placed twice since 1996 under increased watchfulness (maintenance rule) due to adverse impacts of saltwater attack on concrete.	Degraded conditions documented on drawings and updated after each refuelling outage. Refurbishment plans include concrete repairs and installation of cathodic protection anodes at various locations.	[221, 238]
Duane Arnold, USA (1974)	Hairline cracks in floor under torus.	Cracks permitted to self-heal.	[81]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Farley 1, USA (1985)	Cracks detected in six containment tendon anchors during refuelling outage. Attributed to stress corrosion cracking caused by a combination of high strength low-alloy steel under high stress in presence of moisture and impurities.	Anchor heads replaced.	[81, 664]
Farley 2, USA (1985)	Three anchor heads on bottom ends of vertical tendons failed and 18 cracked with several tendon wires fractured, occurred about 8 years after tensioning, cause was attributed to hydrogen stress cracking.	All tendons and anchor heads from same heat were inspected with no further problems noted, 20 tendons replaced.	[81]
Fermi, USA (1972)	Cracks <0.8 mm wide in basement floor slab permitted groundwater to seep into building, cracks caused by shrinkage.	Cracks repaired by pressure grouting after determining that they were no threat to structural integrity.	[81]
Fermi, USA (1984)	Voids detected around one of auxiliary building watertight doors.	Defective concrete removed by chipping and area-grouted, other doors inspected.	[81]
Fitzpatrick, USA (1973)	Horitzontal crack from hairline to 0.9 cm wide in reactor pedestal extending into concrete 0.2 to 0.7 m; cause believed to be welding procedure causing tension; structural integrity of pedestal not impaired.	Crack sealed by epoxy injection.	[81]
Fort St. Vrain, USA (1984)	Tendon wire failures noted because of tendon corrosion caused by microbiological attack of corrosion inhibitor, analysis revealed sufficient tendons intact to provide structural integrity.	Surveillance increased and tendons inerted by nitrogen blanket.	[81]
Ginna, USA (1981)	Excessive loss of prestressing force. Average higher than anticipated temperatures around tendons attributed to consistently lower prestressing forces.	Tendons retensioned with no recurrence noted in subsequent inspections.	[73, 81]
Grand Gulf 1/2, USA (1975)	Seven of 19 cylinders for control building basemat concrete did not meet 28 day design strength.	Ninety day values were acceptable.	[81]
Grand Gulf 1/2, USA (1976)	Voids found beneath drywell wall embed and shear key because too stiff a concrete mix used.	Holes drilled through embed and used to fill voids with high-strength grout, voids below shear key repaired by removing central portion of plate, chipping to good concrete, adding rebar, replacing concrete and liner and leak testing liner.	[81]
H.B. Robinson, USA	Grout under several bearing plates failed while tensioning tendons.	Grout replaced.	[83]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
H.B. Robinson, 2, USA	Cracking and spalling of concrete in limited areas in walls and ceilings of the reactor auxiliary bay, emergency diesel generator building and intake structure.	None reported.	[221]
H.B. Robinson, 2, USA (1992)	Discoloration of vertical portion of containment liner indicating possible corrosion.	None reported.	[157]
H.B. Robinson, 2, USA (1996)	Containment liner protective coating degraded and some liner corrosion had occurred.	Liner evaluated to still meet design requirements.	[157]
Harris 1, USA	Liner buckling under transfer canal.	Hammer tapping indicated adjacent areas within a few feet where studs existed were solid. Structural integrity determined to not be impacted.	[157]
Hatch 1, USA (1981)	Cracks in concrete wall around base plate. Concrete in pedestal for several recirculation line snubbers exhibited spalling and cracking due to design deviation.	2.5 cm plates with four wedge anchors installed on top of existing plates.	[81]
Hatch 2, USA (1979)	Approximately 101 failures occurred during testing of 183 anchor bolts because of improper installation.	Failed bolts replaced with wedge anchors.	[81]
Hatch 2, USA (1982)	Main steam pipe hangers had significant concrete spalling around embedded plate with concrete missing approximately 5 cm adjacent to plate, cause was defective concrete material or faulty placement.	Plate redesigned.	[81]
Indian Point 2, USA (1974)	Concrete temperature local to hot penetration >66°C but <93°C.	No safety problem due to relatively short periods of exposure.	[81]
Indian Point 1/2, USA (1990)	Discovery of a small amount of contaminated water leaking from Unit 2 SFP, and subsequent discovery of additional subsurface groundwater contamination from Unit 1 SFP system. U1 leakage first detected 1990 (U1 had been shut down in 1974 but fuel remained in SFP). U2 liner likely perforated in 1990 by diver.	Long term programme to investigate leakage and subsequent groundwater contamination. Crack in SFP assessed to not impact structural integrity. Groundwater investigation and monitoring programme initiated. Fuel in U1 SFP to be moved to dry storage.	[221, 670]
Indian Point 2, USA (2001)	Exterior concrete containment exhibited concrete spalling in 6 to 7 areas with minor rebar rusting. Locations were in areas where cadweld sleeves had insufficient cover.	Evaluations indicated no impact on structural integrity. Inspection of areas was enhanced with repairs to be implemented if there is a change in appearance.	[157, 671]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Kewaunee, USA (1997)	Concrete cracking with leaching observed in screen house and tunnel. Specific deficiencies noted included cracking, leaching, pattern cracking and surface offset.	Follow-on inspections revealed no change in surface conditions with cracks apparently passive.	[221]
Kewaunee, USA (2003)	Minor leaching of calcium hydroxide in main transformer walls, turbine building exterior wall, shield building exterior concrete wall, and forebay exterior concrete wall.	No report.	[221]
Kewaunee, USA (2007)	Identification of white deposits on wall and ceiling of waste drumming room adjacent to the SFP, indicating leakage.	After a year of monitoring wall and ceiling of waste drumming room, it was concluded that residue formation remained slow and constant, and therefore no near-term concern for structure. Actions would be implemented if any change in leakage trend or other signs of concrete distress were observed. Additional groundwater monitoring indicated no detectable level of tritium outside or in groundwater.	[221]
La Salle 1/2, USA (1976)	Low concrete strength at 90 days.	In-place strength determined acceptable from cores, cement content for future pours increased, strength low only in a small percent of pours so did not threaten structural integrity.	[81]
Marble Hill, USA (1979)	Construction related issues, including: • High concrete pour rate may have bowed liner; • 0.3 m deep void extending 6.1 x 1.4 m in axial direction in base slab for auxiliary building; • Numerous surface defects (4000) and inadequate patching resulting from poor concrete compaction and improperly prepared construction joints; breakdown in quality control and construction management attribute as cause; internal concrete inspection revealed it to be of high quality with higher than required strength.	Void repaired by shotcrete injection. Patches removed and replaced using good construction practices; providing good workmanship is used in repair and procedures followed, consultants determined structural integrity and shielding requirements need to be met.	[81]
McGuire 1, USA (1976)	Two buttonheads failed during stressing of control rod drive mechanism missile shield hold down tendons at underside of bottom plate and two wires failed in another tendon near base anchor, additional failed wires found during checking, cause was excessive corrosion.	Design modified to replace tendons with 3.5 cm diameter threaded rods that were grouted into place.	[81]
McGuire 1, USA (1990)	Degradation of steel containment consisting of general coating failures and localized pits in vicinity of ice condenser floor gap that was filled with cork (which trapped moisture).	Not reported.	[157]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
McGuire 2, USA (1999)	Significant coating damage and base metal corrosion due to lack of sealant at interface between shell and annulus floor.	Not reported.	[157]
Midland 2, USA (1975)	Rebar spacing deficiencies in reactor containment building.	Determined error not significant enough to affect safety.	[81]
Midland 2, USA (1977)	Leaking water pipe in exterior wall caused bulging of liner plate up to 0.6 m inwards over an area of about 195 m^2 producing concrete spalling of 7.5–25.4 cm deep.	Bulged liner plate and concrete removed.	[81]
Millstone 3, USA (1987)	Erosion of porous concrete from containment subfoundation structure detected via examination of sludge (accumulated white residue) in drainage sumps. Concerned raised on the ability of eroded porous media to transfer containment loads to bedrock.	Quantity of eroded cement was determined to be minor, sumps monitored for cement erosion, strength tests on cores obtained from mock-up tests. Settlement monitored and none detected. Extent of condition at other US plants investigated and surveillance programmes implemented.	[37, 221, 672, 673]
Monticello, USA	Honeycomb voids in basemat.	Repaired by epoxy injection.	[83]
Monticello, USA (1998/2001/2002)	Structural inspections identified concrete spalling, cracking, surface deterioration and flaking, grout deterioration, corroded rebar or other steel components, and cracked welds.	Not reported.	[221]
Nine Mile Point 1/2, USA (2005)	Cracking observed in service water pipe tunnel permitting in-leakage of groundwater.	Cracks repaired with follow-up inspections.	[221]
North Anna 2, USA (1974)	Cracks >1.6 mm wide in containment floor slab occurred around neutron shield tank anchor bolts following pressure testing of seal chambers due to inadvertent pressurization; cores showed cracks extended into concrete vertically.	Cracks no structural threat, routed and sealed to prevent fluid penetration.	[81]
North Anna 2, USA (1999)	Through wall hole discovered in containment liner at 75 m elevation of containment building during investigation of rusted spot on liner. Removal of liner in area revealed a piece of wood 10 cm × 10 cm × 1.8 m that was left in contact with liner after construction.	Wood removed, void grouted and liner plate replaced.	[157]
North Anna 2, USA (2001)	Six pieces of wood discovered embedded in concrete dome.	Pieces removed. In some locations no repairs necessary; in others areas repaired with grout.	[157]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Oconee 1, USA	Lack of concrete consolidation beneath a bearing plate and lack of bonding between two pours at construction joint.	Not reported.	[157]
Oconee 1/2/3, USA	Tendon grease leakage.	Monitor grease quantity.	[73]
Oconee 1/2/3, USA	Water infiltration into tendon galleries.	Purge tendon galleries periodically to remove excess water.	[73]
Oconee 1/2/3, USA	Failed and corroded tendons observed during final containment testing prior to start up, cause attributed to stress corrosion cracking.	Defective tendons replaced and drains installed on anchorage grease caps.	[83]
Oconee 1/2/3, USA	Tendon anchor head thread problems due to overstressing.	Failed anchor heads repaired, other anchor heads surveyed.	[83]
Oconee 1/2/3, USA (1998)	Concrete spalling below anchor-bearing plate with cavity found below plate. Cracks found in concrete beneath outer edge of tendon bearing plates observed for a number of tendons.	Concrete repaired.	[221]
Oyster Creek, USA (1980)	Water discovered in gap between reactor drywell liner and concrete shield.	Seals and gaskets replaced. Liner cleaned, repaired and recoated using submersible coating.	[157]
Oyster Creek, USA (2001)	Cracking of drywell shield wall observed. Cause attributed to high temperatures in upper elevation of drywell.	Engineering analysis concluded that stresses remained were well below allowable limits. Recent inspections indicated no significant change in cracked area.	[221]
Palisades, USA (1975)	Sixty-three out of 3780 buttonheads inspected found split.	No threat to structural integrity.	[81]
Palisades, USA (1999)	Floor to liner moisture barrier seal found as never installed. Moisture confirmed in crevice.	New liner to floor moisture barrier seal installed. Liner determined not to be appreciably degraded.	[157]
Palo Verde 2/3, USA (1984)	Honeycombing around vertical tendon sheath blockouts with most voids at buttress/shell interface above last dome hoop tendon.	Condition was localized so area was repaired with grout.	[81]
Palo Verde 1, USA (2005)	Water observed seeping from SFP wall into clean-up pump area and outside fuel building. SFP tell-tale drains were opened and about 4.54 m^3 of water were released from drains. Cause of water backing up in leak chase system was a pressure test plug that had been lodged in drain basin drain line since construction.	Contamination removed from outer walls. Operations and inspection procedures implemented to ensure drains are kept clear and water does not leak through the concrete.	[221]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Peach Bottom 2/3, USA (1969)	Aluminium pipe used to place concrete caused concrete strength reduction up to 50%.	Low strength concrete in biological shield wall and floor slab of turbine building replaced.	[81]
Peach Bottom 2/3, USA	Emergency cooling tower and reservoir reinforced concrete walls were exposed to aggressive raw water and experienced leaching of calcium hydroxide.	Reported corrective actions taken prior to loss of function.	[221]
Pilgrim	Intake structure degradation reported.	Not reported.	[238]
Point Beach 1, USA	Piece of wood found near spare electrical penetration.	Visual examination under IWE inspection programme completed.	[157]
Point Beach 2, USA	General concrete cracking in pump house walls, auxiliary building and emergency diesel generator building.	Not reported.	[238]
Point Beach 2, USA	Groundwater seepage into underground portions of safety related structures.	Not reported.	[238]
Point Beach 2, USA	Liner plate separated from concrete at several locations.	Evaluation indicated no action required.	[83]
Point Beach 1 & 2, USA	Exposed rebar reported (construction defect).	Evaluated as not affecting structural integrity.	[157]
St. Lucie 1, USA (1974)	Concrete spalled because of scaffolding fire in annulus between containment vessel and shield building, area affected -3.4 m x 0.6 x 2.5 cm. Temperature reached 148 to 177°C inflicting only superficial damage.	Spalled area replaced.	[81]
St. Lucie 1, USA (1978)	Hairline crack 1 mm wide by 1 m long in east wall of reactor containment refueling canal near embedded steel plate.	Crack repaired by grouting and column added to support platform girder.	[81]
Salem 2, USA (1974)	Incomplete concrete pour near equipment hatch due to use of wrong concrete mix.	Voids repaired with high strength non-shrink grout.	[81]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Salem, USA (2002)	In 2002, radioactive water leakage through an interior wall was detected, and in 2003 evidence of tritium in groundwater in two test locations was identified. Leakage was traced to Unit 1 SFP, and was as a result of obstruction of leakage detection and collection system of SFP stainless steel liner.	Leakage into ground outside building was remediated and potential degradation of concrete structure from exposure to boric acid was evaluated. A comprehensive groundwater sampling and analysis programme was implemented. A cleaning and maintenance process for SFP drains implemented.	[221, 674]
Salem, USA (2009)	Heavy corrosion observed at seal between containment floor and liner within 15 cm of concrete floor. Area normally considered inaccessible (and thus exempt from inspection) due to it normally being covered by an insulation package. Source of corrosion was determined to be service water leakage from fan coil units and associated piping.	UT performed at 440 locations on bottom 15 cm of floor. Operability confirmed and safety significance deemed minor due to significant design margin in liner thickness. Frequent walkdowns to identify and repair service water leaks. Revised procedures to ensure inspections are performed when containment service water leaks are identified. Recoating containment liner during refuelling outage.	[221, 663]
Salem Unit 1/2, USA (2010)	Spalling of service water intake structure concrete with rebar exposed and exhibiting corrosion.	Condition evaluated and repair implemented with follow-on inspections.	[221]
San Onofre 1, USA (1976)	Voids at 14 locations in diesel generator building center wall; areas from 0.09 m^2 with 7–10 cm penetration to several square meters with full penetration.	Repaired with dry pack, grout or concrete.	[81]
San Onofre 3, USA	Exterior concrete walls of intake structure and concrete beams supporting service water pumps cracked extensively due to chloride ion penetration that caused corrosion of embedded steel reinforcement.	Walls were reinforced with exterior steel plates anchored to concrete and cathodic protection sacrificial zinc anodes were placed into steel plates to protect against corrosion. Later inspections found new areas of cracking and rebar corrosion, but degradation was not as great.	[221, 238]
San Onofre 3, USA (1983)	Tendon lift-off forces in excess of maximum value listed in plant technical specifications, cause was lower relaxation rate than expected.	No threat to structural integrity.	[81]
Seabrook 1/2, USA (1983)	Cracking occurred in walls at end of stiffening slabs separating pump cells in category 1 service water and circulating water pumphouse; cause was shrinkage and temperature variations.	Stiffening slabs were modified.	[81]
Seabrook, USA (2006)	Indications of leaching in below grade concrete.	No report.	[221]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Seabrook, USA (2010)	During licence renewal application, licensee identified concrete degradation in the facility in the form of an ASR. Certain below grade structures experienced groundwater infiltration that had in turn induced an ASR, which can result in a gel formation than can expand and form microcracks.	Extensive investigation to determine impact of ASRs on plant performance. Condition assessment criteria being developed through experimental studies. Short and long term concrete aggregate expansion testing and other R&D performed. Structure monitoring programme enhanced. Crack measurement and indexing completed and anchor test programme developed. Remedial actions to mitigate ASR effects.	[221, 675]
Sequoyah 2, USA (1978)	Concrete in outer 2.5–5 cm of Unit 2 shield building was under strength because of exposure to freezing temperatures at early concrete age.	Determined to not affect shield building capability.	[81, 221]
Sequoyah 2, USA (2002)	Areas of steel containment vessel with degraded coatings and rust. One floor drain clogged.	No report.	[157]
Shoreham, USA (1974)	Unconsolidated and honeycombed areas in first lift of reactor support pedestal.	Voids repaired after determining that they were not a threat to structural integrity, placement procedures improved.	[81]
South Texas 1/2, USA (1977)	Crack in fuel handling building wall due to shrinkage.	No structural significance.	[81]
South Texas 1/2, USA (1977)	Rebar improperly located in buttress region of Unit 1 containment.	Detailed analysis of as built condition determined that no safety hazard to public occurred.	[81]
South Texas 1/2, USA (1978)	Voids occurred behind liner plate of Unit 1 reactor containment building exterior wall because of planning deficiencies, long pour times and several pump breakdowns.	Sounding and fibre optic exam through holes drilled in liner plate were used to determine extent, areas were repaired by grout injection.	[81]
Summer 1, USA	During fourth tendon surveillance, prestressing forces of a number of vertical tendons were found to be lower than expected. Cause attributed to excessive wire relaxation due to sustained elevated temperatures around tendon.	Vertical tendons retensioned.	[221]
Summer 1, USA (1976)	Voids located behind liner plate of reactor containment building wall, windows cut in liner revealed voids up to 22 cm deep; cause was use of low slump concrete with insufficient compaction.	Voids chipped, cleaned to sound concrete and filled with non-shrink grout, liner repaired and all welds leak tested.	[81]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Summer 1, USA (1977)	Excessive heat from welding caused liner attached to concrete on inside face of concrete primary shield wall cavity to buckle and fail stud anchors and crack concrete.	Liner and concrete to depth of 15 cm removed, new liner plate welded in place and space filled with high strength grout.	[81]
Surry 1, USA (1979)	Cracking in concrete supports for two heat exchangers caused by thermal expansion of heat exchanger shells.	Cracks repaired and supports modified.	[81]
Surry, USA (2003)	Degraded coatings and rust found on containment liner at junction of metal liner with concrete floor. Moisture barrier was also found to be damaged.	Moisture barrier repaired.	[157]
Susquehanna 1/2, USA (1976)	Coarse aggregate with excessive fines used because of quality control deficiency, concrete strength exceeded requirements so structural integrity not affected.	Aggregate material for future batches replaced.	[81]
Three Mile Island 1, USA (1974)	Cracking <0.02 cm wide in containment building ring girder and around tendon bearing plates.	Cracks repaired and monitored during subsequent surveillance.	[221, 81]
Three Mile Island 1, USA (2004)	Water seeping under three embedded plates on dome has resulted in leaching of concrete.	Concrete to embed surface area interface was sealed with a caulking compound to prevent further entry of water.	[221]
Three Mile Island 1, USA (2004)	Grout patches have detached from dome surface at two locations leaving depressions that can accumulate water.	Depressions were filled with epoxy grout to prevent ponding and the consequent possibility of progressive freeze-thaw damage.	[221]
Three Mile Island 2, USA (1974)	Four of six sets of compression cylinders had low FC because of mishandling and inventory control at cement silo, 90 day strengths were acceptable and concrete in place determined to have adequate strength.	Cement storage and sampling techniques improved.	[81]
Three Mile Island 2, USA (1975)	Void 7 cm high x 1.8 m wide x 0.9 to 1.5 m deep occurred in south exterior wall of fuel handling building, cause was improper placement, void determined not to be a threat to structural or shielding effectiveness.	Void refilled.	[81]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Three Mile Island 2, USA (1976)	Void 0.9 to 1.2 m into concrete 0.4 m high by 1.8 to 2.4 m wide in north exterior wall of fuel transfer canal; no structural or shielding effectiveness threat.	Void repaired.	[81]
Turkey Point 3, USA (1970)	Dome delamination.	Delaminated concrete removed, additional rebars provided, concrete replaced.	[81]
Turkey Point 3, USA (1974)	Grease leakage from 110 of 832 tendons at casing.	Tendon casings repaired and refilled.	[81]
Turkey Point 3, USA (1975)	Concrete spalling of horizontal joint at containment ring girder with cavities 3–5 cm wide by 7–10 cm deep.	No threat to structural integrity, repaired by dry packing.	[81]
Turkey Point 3, USA (1982)	Small void under equipment hatch barrel.	No threat to structural integrity, repaired by grouting.	[81]
Turkey Point 3, USA (1987–89)	Extensive cracking of reinforced concrete beams supporting recirculating water pumps of intake structures. Cause related to corrosion of embedded steel reinforcement due to harsh environmental conditions (i.e. salt water). Although reinforced concrete intake structure walls were subjected to a similar environment, they did not exhibit degradation, most likely because rebar was protected by adequate concrete cover.	Licensee assessed significance of degraded supporting beams and began corrective programmes to ensure the structural integrity of the beams and minimize future penetration of chloride ions into the concrete.	[221, 238]
Turkey Point 3, USA (2007)	Buttress 3 observed to have three concrete spalls on its face with exposed rebar.	Engineering disposition found concrete damage was superficial and in concrete cover. Area cleaned and protective coating applied.	[221]
Turkey Point 4, USA (1981)	Approximately 0.1 m^3 of concrete with inadequate fine material content.	Area removed and refilled with correct concrete mix.	[81]
Vogtle Unit 1, USA (1998)	Concrete damage discovered on roof to diesel generator 1A and 1B exhaust pipe concrete barriers. Concrete spalled off inside of barrier in several locations exposing rebar in some areas. Additional inspections found similar degraded barriers on two Unit 2 diesel generators. Different expansion and contraction rates caused degradation.	Preliminary evaluation determined degradation had no immediate impact on diesel generator operability.	[221]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Vogtle Unit 2, USA (1999)	Visual examination detected an area of spalling on the edge of containment buttresses for Unit 1	Engineering evaluation.	[221]
Vogtle Unit 3, USA (2012)	Non-compliant rebar installed at site of Vogtle NPP's new AP1000 Unit 3 reactor (not ageing related)	Request submitted to the NRC to change concrete specified compressive strength from 4000 psi to 5000 psi for basemat to give the structures the desired resistance to seismic activity and bring into full design compliance.	[676, 677]
Waterford 3, USA (1976/77)	Construction related events, including: • Improper concrete placing sequences used in foundation met forming a cold joint and not achieving stepped bedding planes, core drilling revealed fine cracks and honeycombed areas; • Improper placement of concrete in reactor auxiliary building interior wall resulted in honeycombed areas, voids, and cold joints; • Crane boom fell during construction on common foundation structure wall causing concrete cracking and spalling over area 0.3 m x 10 cm x 2.5 cm, rebars and concrete removed and replaced over entire height of damaged area for a length of 9.5 m; • Low concrete compressive strength in 4.2 m^3 of concrete in wall contiguous with portion of condensate storage pool wall and wall of refuelling water pool; • Low concrete strength in reactor auxiliary building slab, cores yielded satisfactory strength.	Depending on event, defective concrete removed and replaced, supervision and inspection increased, amount of sand in future mixes increased as well as mixing requirements.	[81]
Waterford 3, USA (1984)	Spalled concrete observed in corbel exposing and displacing rebars and cracking in plane of anchor bolts, no loss of structural support.	Area repaired.	[81]
Wolf Creek, USA (1978)	Voids up to 1.8 m wide and through wall thickness occurred under equipment and personnel hatches in reactor containment building.	Voids repaired and quality assurance programme updated.	[81]
Yankee Rowe, USA (1967)	4.6 m shrinkage crack discovered.	Crack covered with fibreglass and recoated.	[81]
Zion 1, USA (1972)	Excessive pitting in some tendon wires of Unit 2 during installation; cause was outdoor storage in conjunction with high precipitation and inadequate protection.	Defective tendons replaced.	[81]

TABLE 34. SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS (cont.)

Plant	Problem area	Remedial measure implemented	References
Various prestressed concrete pressure vessels, UK	Small surface cracks observed during biennial inspection. Minor spalling in vicinity of helical prestressing system anchorages.	Monitored to show cracks were passive, no further action required. Judged to have no structural significance, no further action.	[678]
Dungeness B, UK	Corrosion of prestressing tendons due to stray electrical currents at time of construction.	Tendons replaced.	[678]
Hunterston B, UK	Circumferential crack at construction joint attributed to plastic settlement and shrinkage.	Repaired by injecting resin grout, success confirmed by subsequent coring and through-life monitoring.	[679]
Oldbury, UK	Localized honeycombing behind prestressing anchorages leading to loss of prestressing force.	Injected with epoxy resin.	[680]
Oldbury, UK	Circumferential crack at joint due to construction sequence.	Injected with resin grout.	[681]
Oldbury, UK	Local concrete temperatures higher than design values.	Changes made to thermal shielding, analytical and experimental evaluations to demonstrate acceptability of local hot spots.	[680]
Oldbury, UK	Settlement of circular raft foundation exceeded design predictions.	Floors jacked and pipework adjusted, monitoring used to show that settlement has stabilized.	[680]
Sizewell B, UK (2013)	Localized degradation in the reactor building sump liners. One defect was shown to be ‘through wall’ of the bottom steel liner plate of the sump. Steel liners form part of the containment pressure boundary, and degradation was a threat to containment integrity. Liner paint coating suffered damage in a number of locations. As a result, localized corrosion of the mild steel liner occurred, causing material loss. In one location, the extent of material loss was such that the defect became through wall.	Leak rate testing of defect confirmed that leakage was within safety case limits. Repair technique was developed and undertaken before return to service.	[682]

Appendix V

AGEING EXPERIENCE OF NON-NUCLEAR CONCRETE STRUCTURES

V.1. INTRODUCTION

In considering performance of NPPs' reinforced or prestressed concrete containments, parallels may be drawn with other civil engineering structures. These structures have generally performed well when properly designed, constructed and maintained. Nevertheless, degradation has been observed in a wide range of structures. In the absence of mitigation, this has been seen to progress until, ultimately, failure has occurred.

In making comparisons, it is necessary to recognize that design and construction requirements are often more stringent for nuclear applications. This has a direct, positive, impact on the major causes of degradation in civil engineering structures. However, the cost of failure is more significant in NPPs; this is a feature of both the difficulties associated with repair in a nuclear environment and, more significantly, the costs linked to lost generation.

V.2. NON-NUCLEAR INDUSTRY PRACTICES

Since the 1950s, design, specification and construction of reinforced and prestressed concrete has been marked by a progressive improvement in the understanding of durability issues. This has been reflected, to some extent, through increasingly explicit design measures to help durability (e.g. concrete cover requirements in design codes may be specified in terms of the structure's environment). Some recent guides include requirements for monitoring construction quality and through-life performance of concrete structures.

It is useful to consider monitoring practices for NPP civil structures in the context of other relevant industries; those with structures that have a high cost of failure (in terms of either financial or safety implications), such as:

- The water industry;
- The process industry;
- The transportation industry;
- Offshore industry/marine structures.

Table 35 summarizes the ageing management practices of these industries. It may be seen that:

- All four industries have routine inspection regimes in place, which trigger appropriate reactive maintenance if a fault is found.
- Frequencies of visual inspection are between four months and ten years; the frequency reflects the detail of inspection (more detail, less frequent).
- Prescriptive acceptance criteria for any defects found during inspection are not defined; defect assessments are based on engineering judgment.
- Although assessment and inspection frequency historically have been based on a deterministic approach, the trend is toward probabilistic assessments. These assessments are used to (a) define inspection periods for structures where a large database exists (e.g. bridges), and (b) assess risk from structures with significant consequence of failure (e.g. dams).

The following summarizes additional information related to ageing management practices of the relevant industries noted above. It was compiled primarily in the 1990s, thus it may not completely reflect current practice.

V.2.1. Water industry

Reinforced concrete dams, locks and spillways are routinely inspected for structural safety, stability and operation adequacy [683]. A condition survey of these structures generally includes a comprehensive review of structure design, construction techniques and materials, and operational and maintenance history. Information is obtained from available structure engineering data as well as from on-site investigations.

Data are analysed and an evaluation report written that includes conclusions and/or repair recommendations. Condition assessment frequency and the approaches that are followed vary from country to country.

V.2.1.1. United States of America

Organizations such as the Army Corps of Engineers, the Bureau of Reclamation and Tennessee Valley Authority set standards for their civil structures.

Facilities are diverse due to factors such as intended purpose and use, amount and type of building material and geographical location, most procedures use checklists for review and maintenance operations and several organizations have developed manuals of practice. Although some organizations have developed rating systems for use in their assessments, the majority generally are subjective in that responses are of the form:

— Yes/no;
— Satisfactory/unsatisfactory;
— High/medium/low;
— Excellent/good/fair/no change/bad/critical.

Some use a computer database to assist in maintenance operations and photographs are required as part of some technical evaluations. Most organizations require professional engineering services for performing the inspection and maintenance and have a time schedule for overall inspection frequency for their structures.

V.2.1.2. Canada

Guidelines for dam evaluation have been developed by the Canadian Dam Safety Association [684]. Regulation of Canadian dams is a provincial/territorial responsibility. Unlike other countries, Canada does not have a federal regulatory agency or over-arching programme that guides the development of requirements for safe dam management. The Canadian Dam Association (CDA), a volunteer organization formed in the 1980s, provides dam owners, operators, consultants, suppliers and government agencies with a national forum to discuss issues of dam safety in Canada. Dam safety guidelines, developed by the CDA, provide regulators with a basis for evaluating dam safety within their respective jurisdictions.

During the 1990s, Ontario Hydro Technologies (now Kinectrics) conducted a seven-year comprehensive review of Ontario Hydro (now Ontario Power Generation) dam structures in accordance with the policy of the International Commission on Large Dams (France). Reviews of dam safety included:

— Site inspections;
— Hazard classification;
— Design reviews;
— Surveillance, monitoring and dam performance;
— Maintenance adequacy;
— Operation history;
— Emergency response procedures.

Adequacy of a structure is reviewed against established criteria and a strategy is proposed if shortfalls are identified. Risk based methods have been incorporated into the assessment process. The maximum period between dam safety reviews is governed by consequences of failure (i.e. high, significant and low relates to intervals of 5, 7 and 10 years, respectively). Criteria for use in the evaluation are contained in the guidelines.

TABLE 35. SUMMARY OF INSPECTION PRACTICES FOR CONCRETE STRUCTURES IN SELECTED NON-NUCLEAR INDUSTRIES

Industry	Structure(s)	Routine inspection regime	Frequency (typical)	Acceptance criteria clearly defined	Assessment inspection techniques			Comments
					Deterministic	Probabilistic	Risk based	
Water	Dams/ reservoirs	Yes	4 month (minor) 10 years (major)	No	Yes	Yes	Yes	Increasing trends toward probabilistic and risk based assessment techniques
Process	Storage tanks	Yes	5 years	No	Yes	No	No	Little published data available
Transportation	Bridges	Yes	2 years (general) 6 years (principal)	No	Yes	Yes	Yes	Increasing trends toward probabilistic and risk based assessment techniques
Offshore	Production platforms	Yes	Not determined	No	Yes	No	No	Probabilistic techniques in use for steel platform only

V.2.1.3. United Kingdom

As is typical in Europe, in the UK there are two levels of inspection [685]. At the lower level, routine visual inspections are performed, typically at a four month cycle, by an engineer employed by the owner. The upper level, required every ten years, involves a thorough inspection by an independent civil engineer selected from a panel of approved engineers. A complete investigation of any apparent degradation is required and may require the use of non-destructive and destructive testing techniques. No criteria are available to assess the significance of any defects or degradation noted. The independent civil engineer makes the assessment and recommends the remedial measure requirements.

V.2.2. Offshore industry/marine structures

V.2.2.1. Offshore oil/gas production structures

The oil industry has built thousands of offshore and coastal platforms, with construction of deep water rough sea platforms becoming routine [686]. In-service performance of offshore platforms is monitored through a combination of inspection and instrumentation programmes.

Inspection of platform substructures is undertaken regularly to monitor overall structural condition as well as any components that have been identified as having some risk for damage (e.g. specific welds in metal structures that may be vulnerable to fatigue). Reference [687] notes that the same standards of inspection are not required in all areas of the structure and three classes of visual and NDT method requirements are provided for underwater and atmospheric inspections. It is generally recommended that structures be surveyed annually for damage or deterioration, paying particular attention to areas that required previous repairs or modifications and parts exposed or subjected to fatigue loadings or alternate wetting and drying [688]. In addition, results of surveys need to be reviewed in detail every five years and need to cover:

— Visual inspection of general conditions;
— Concrete deterioration or cracking;
— Condition and function of the corrosion protection system (if present);
— Condition of exposed metal components;

— Condition of foundation and of the scour protection system;
— Amount of marine growth and presence of debris.

Visual signs indicating the need for future surveillance or repair include rust stains on concrete surface, cracking or splitting of concrete, spalling or erosion and damage due to impact. No detailed acceptance criteria are provided. Specific information regarding areas to inspect and what to look for is provided in documents such as Ref. [689].

In-service performance of offshore structures is also monitored by instrumentation systems. The primary objective of these systems is to monitor response of the structures to loadings (e.g. wave and impact). Results also have application toward the design of future structures.

V.2.2.2. Marine structures

Concrete structures in a marine environment are exposed simultaneously to the physical and chemical deterioration processes. Four environmental zones are usually considered: underwater, tidal, splash and atmospheric [690]. Tidal and splash zones have many similarities and are often combined. Splash (and tidal) zones are considered the most susceptible to degradation because of their repeated wetting and drying. The atmospheric zone follows in susceptibility ranking. The most significant threat is rebar corrosion. Structure durability is covered through minimum design requirements for concrete quality and rebar cover. Concrete quality is addressed in terms of minimum cement contents, and concrete cover is specified based on exposure classification and concrete strength. Concrete condition assessments in marine structures are based primarily on visual inspections and follow the general guidance previously discussed for offshore oil/gas production structures.

The main cause of deterioration of marine structures is ingress of chloride ions from sea water, which in time will cause rebar corrosion.

In some countries (e.g. Scandinavia), tidal variations are quite small with the result that splash zones essentially remain in a permanent state of high moisture content (i.e. no significant drying period). This may inhibit oxygen ingress, thus reducing the risk of corrosion.

Submerged structures normally have a low risk of rebar corrosion due to low oxygen levels. If, however, concrete is of poor quality (e.g. a high water to cement ratio and low resistivity) a macrocell may be created between above and below water reinforcement. Reinforcement above water will receive atmospheric oxygen and fuel the corrosion process at the submerged anodic reinforcement.

V.2.3. Transportation industry

Principal concrete items include highway bridges, pavement and railway/transit structures.

V.2.3.1. Highway bridges

Both reinforced and prestressed concrete have been used extensively in highway bridges. Service life design for these structures generally involves a combination of:

— Design strategy (protection against deterioration);
— Type and composition of materials used;
— Workmanship;
— Maintenance strategy;
— Level of quality assurance [691].

Although this approach has been used to design bridge structures that are expected to have service lives of at least 50 years in the USA and over 100 years in the UK, problems have occurred within 10 years or less of construction. Factors contributing to premature deterioration of bridges include:

— Corrosion of metallic embedments (e.g. rebar) due to environmental conditions and use of de-icing salts;
— Fatigue of structural members;

— Inadequate maintenance;
— Changing performance requirements for older bridges as a result of increased vehicle sizes and frequency of use.

Deterioration of concrete that led to the collapse of a portion of la Concorde overpass in Montreal, Canada, killing five people and injuring six others, was linked in part to low quality concrete that was used for the abutments. The concrete was too porous and the air bubble network was deficient [692].

A systematic approach is required to effectively address the problem of bridge deterioration. Some activities toward development of such an approach have occurred.

Systems or guidelines for development of bridge management and structural assessment systems have been developed by the Organisation for Economic Co-operation and Development (OECD) for use by Member Countries. Countries also have developed systems for their own use, for example the UK and the USA.

The OECD has developed basic recommendations for a bridge management system, which includes optimization of inspection and maintenance of bridges, prediction of deterioration and service life, development of data banks and management systems and formulation of guidelines for bridge management practices [693]. Bridge inspections are carried out according to a general inspection plan that includes inspection work scheduling, decisions on different inspection types, inspection intervals, data collection, financial possibilities and inspector's training. Bridge inventory, collection of bridge data and easy access to this information are considered important for optimization of the inspection. An automated archive (e.g. electronic database) is the preferred medium for data storage and retrieval. Optimization of maintenance requires introduction of criteria for maintenance operations, access to information on previous repair actions and adequate training of personnel. Although considerable progress has been made with respect to repair actions, the effectiveness of repair actions is difficult to assess. Maintenance needs to be carried out at timely intervals to ensure safety and avoid undue traffic obstruction. Assessing the remaining service life of a bridge is an essential factor in overall bridge management and requires development of more detailed deterioration models. Service life can be based on statistical information from actual service lives or methods based on log-normal distributions with respect to variables such as materials, bridge conditions and structural aspects.

Reliability assessments of existing bridges require data on such things as actual geometry, material parameters and performance, to which probabilistic methods can be applied. Data banks have been developed by many Member Countries for use in bridge management. Two basic types of data have been incorporated: administrative and technical information for each bridge (constant data), and variable data related to daily activities such as inspection and maintenance actions.

(a) *United Kingdom*

Inspection procedures for concrete bridges are primarily contained in Refs [694, 695] and are of two types:

— General inspections carried out every two years by a bridge inspector;
— Principal inspections involving a complete examination of all accessible elements that are carried out every six years by a chartered engineer with results provided in a detailed report.

As a result of the general or principal inspection, there may be special inspections. Guidance on defect detection (e.g. chloride ion attack, carbonation, ASRs and cracking), testing methods and repair are provided in Ref. [695]. Any defect present is generally repaired. If the bridge is assessed to be in poor condition, the responsible authority undertakes a detailed assessment that may result in weight restrictions, strengthening or replacement.

A new inspection reporting system has been developed [696–699]. The bridge condition indicator algorithms evaluate a bridge condition index (BCI) for an individual bridge or a stock of bridges using the element condition data. Standardization of bridge inspection reporting will, in general, provide for higher accuracy.

(b) *United States of America*

Under a bridge inspection programme that began in March 1968, a biennial inspection of most of the bridges in the country is conducted. Although each state has its own approach to bridge inspection, a structural integrity and appraisal (SI&A) sheet, which is used to develop the national bridge inventory (NBI), is contained as a part of each. The NBI provides a national centralized assessment of bridges and is used to justify solicitation of funding from Congress for repair or replacement of structurally deficient or obsolete bridges. Of the 88 items that are included on the SI&A sheet, 14 describe bridge location, 6 list the agencies responsible and 16 list structural improvements. Forty-four of the remaining items address structural data with a somewhat subjective approach used to develop a single digit rating for the condition of the deck and superstructure, among other things. Bridge inventory and condition assessment data included in the SI&A sheets are of insufficient detail to provide the basis for a comprehensive appraisal of bridge performance (trending analysis) to determine why some bridges deteriorate prematurely and require extensive rehabilitation, while others require only minor repairs throughout their service lives, which may be 50 years or more. In general, rating systems applied to bridge structural systems are somewhat subjective in nature and also do not develop structural information in sufficient detail to make a quantitative assessment of structural condition or to predict future performance (service life) [700]. To indicate future performance (e.g. residual life) of bridge structures and how remedial actions may influence future conditions, at least 11 states have completed bridge management systems [701].

The most recent and comprehensive development related to bridge management systems is AASHTOWare Bridge Management software BrM (formerly Pontis), which was developed for the Federal Highway Administration [702]. The BrM software is a network optimization system developed to address the improvement and maintenance of bridge networks. Several of BrM's objectives include:

— Providing a systematic procedure for establishing current maintenance, repair and rehabilitation (MR&R) budget requirements;
— Providing a priority listing and sequencing for bridges in need of MR&R and improvement;
— Accommodation of differing inspection and repair needs for the major structural components for bridges as well as the differing needs of the various types of bridges;
— Addressing the probabilistic nature of bridge deterioration;
— Allowing updating of predictive probabilities as data becomes available;
— Considering the relative importance of various bridges in terms of safety, risk exposure and public convenience;
— Providing the basis for short term and long term MR&R and improvement in budget planning and resource allocation.

A set of interrelated predictive, optimization and economic models form the basis of the system. It continues to focus on the complete bridge management cycle, including inspection, inventory data collection and analysis, recommending an optimal preservation policy, predicting needs and performance measures and developing projects to include in agency capital plans.

V.2.3.2. Highway pavements

The aim of pavement ageing management is to compensate for degradation resulting from traffic load as well as from weather and other natural impacts. Requirements for highway pavement ageing management are in large the same as for highway bridges (e.g. nomenclature, rating system, photographs and survey form) [703]. Maintenance is carried out taking into account position and danger of pavement defects detected on the basis of regular road controls and according to the requirements of road management regulations. The Strategic Highway Research Program, initiated in the USA and participated in by Europe and Canada, has developed two identification manuals that are based on reference photographs [704, 705]. Since 1991, the Federal Highway Administration has continued the management and funding of the programme.

Defects characteristic of road conditions are recorded and evaluated by on-site expert estimation, measurement and other analyses, as well as by expert calculation.

Pavement defect identification is generally executed once a year, usually in spring. In the course of the inspection, an error map is prepared by reference to slab numbers. On the basis of this, the severity, type and amount of defects can be assessed. Based on visual inspection, the condition of defective structure parts and characteristics of structure part materials can be determined by NDT and destructive sampling. On the basis of data, defect causes can be diagnosed (e.g. design, construction, applied materials, installation conditions, temperature, precipitation, misuse, accident and ground motion, among other things).

According to the above, the method of intervention and repair technology can be specified. Rehabilitation and reconstruction need to be designed according to the maintenance strategy, considering both traffic volume and its role in the road network. A minimum maintenance activity is determined by traffic safety requirements. A separate study [706], also based on photography, mapped and recorded highway features in the USA and Canada for use in determining road conditions, prioritizing repairs, making cost predictions, planning traffic flow and coordinating work crews.

The Hungarian Road Society has developed a technical specification for concrete pavement maintenance [707] that specifies the required road condition at the time of handover. It also specifies road conditions at which road pavement requires renewal. The application of this technical specification is mandatory for trunk roads and is recommended for local and private roads open to the public.

The specification prescribes maintenance work and classifies pavement deteriorations or defects and their reasons. Some excerpts from the specification are in Table 36. The specification deals with pavement maintenance technologies with renewal of bridge concrete pavement. It also covers renewal and strengthening of concrete pavement with bituminous or concrete overlays. An important part of the technical specification is the maintenance strategy for delaying appearance of defects.

Measurements for monitoring the in-service skid resistance of roads, in line with this specification, are to be made with a Sideway-force Coefficient Routine Investigation Machine (SCRIM). See Fig. 191 for an example of such a machine.

FIG. 191. SCRIM test vehicle (left) (courtesy of W.D.M. Ltd) [708]; test wheel (right) (courtesy of TRL Ltd) [709].

The SCRIM uses the sideway-force principle to measure skid resistance [710] (the force developed on a rubber tire or slider passing over a wetted road surface) and derive a value that is related to the coefficient of friction and the state of polish of the road surface.

A freely rotating wheel fitted with a smooth rubber tire, mounted mid-machine in line with the nearside wheel track and angled at 20° to the vehicle travel direction, is applied to the road surface under a known vertical load. A controlled flow of water wets the road surface immediately in front of the test wheel so that, when the vehicle moves forward, the test wheel slides in the forward direction along the surface.

The force generated by the resistance to sliding is related to the wet road skid resistance of the road surface. Measurement of this sideways component allows the sideway-force coefficient (SFC) to be calculated. SFC is the sideway-force divided by the vertical load.

TABLE 36. HUNGARIAN ROAD SOCIETY CONCRETE PAVEMENT TECHNICAL SPECIFICATION EXCERPTS [707]

Pavement defects

Structural defects

Usability of pavement by traffic, and service levels expected by traffic participants, is affected by:

- Cracks (hairline cracks, cracks larger than 2 mm)
- Defects of gaps (chipping edges, gap stair)
- Surface defects (defects related to durability/material quality)

Functional defects

Impact traffic safety and environmental load performance of pavement (rolling noise) through:

- Unevenness (displacement of concrete slabs)
- Changes of surface texture (surface roughness change)
- Changes of surface structure (scaling, chipping, potholing, shrinkage gaps)

Maintenance strategy delaying the appearance and spread of pavement defects

Regular observations

Defect detection

- Gaps: observation of filling material; measurement of relative displacement of slab edges and size of the stair when the design axle load passes; average frequency of maintenance is 5 years
- Cracks: average frequency of maintenance is 3 years
- Displacements: condition indicator is the displacement measurable relative to the environment
- Slip resistance: condition indicator is the measuring data of the size of Sideway-force Coefficient (SFC) (by a SCRIM measuring car)
- Scaling: usually needs immediate intervention

Preventive measures

- Maintenance of drainage system (ditches, chutes, ducts, draining channels, desiccators)
- Regular maintenance of abutments

V.2.3.3. Railway and transit structures

(a) *United Kingdom*

Bridges are the most common railway structures fabricated of reinforced concrete. Railway bridges are subjected to a wide variety of environments that have produced many concrete deterioration mechanisms (e.g. steel corrosion, ASRs, sulphate attack and frost attack). Occurrence of steel corrosion due to the presence of chlorides or carbonation has been so prevalent that British Rail Research developed a portable on-site test for rapid assessments of chloride levels and carbonation depths [711].

Criteria for interpretation of results from chloride content and half-cell potential tests are essentially the same as used by the general concrete industry.

(b) *USA*

A structural monitoring system was developed for the Miami Metrorail System encompassing a 37 km elevated rapid transit system and ten open-air stations [712]. It was developed to address long term durability and reliability of the transit system, which has a 60 year service life. The inspection programme is designed to provide procedures for the following:

— Inspecting structural subelements (routine format);
— Evaluating changing conditions and adding new ones;
— Maintaining and updating a database;
— Identifying key items requiring frequent monitoring, repair or maintenance;
— Using data collected in analysis routines in the event of derailment or other incident causing structural damage.

Guidelines for data collection were prepared. Structural and cosmetic defects observed by inspectors are recorded in the field to scale on drawings. Data generated in the field is managed by a database system. Data evaluation is performed by the responsible engineer who defines repair procedures, stipulates which items are to be repaired and establishes priorities. Items requiring repair are monitored for at least one additional inspection cycle to ensure effectiveness of the material and technique used. A complete inspection of the Metrorail system takes two years.

V.2.4. Process industry

Inspection procedures and acceptance criteria used by chemical and petrochemical industries generally follow internal company guidelines. One organization, for which information was provided in Ref. [685], inspects concrete structures every five years. Either an in-house engineer or an outside consultant performs inspections to check for a wide range of concrete defects (e.g. cracking, rebar corrosion, scaling and spalling). No acceptance criteria for resolution of defects or deterioration have been established. It is left to the judgment of the inspecting engineer whether additional testing or remedial measures are required.

Appendix VI

COORDINATED RESEARCH PROJECT SURVEY RESULTS REGARDING AGEING MANAGEMENT PRACTICES

VI.1. BACKGROUND

To assist Member States in understanding ageing of SSCs important to safety and their effective ageing management, the IAEA, in 1989, initiated a project on safety aspects of NPP ageing. This project integrated information on evaluation and management of NPP ageing safety aspects that had been generated by Member States into a common knowledge base, derived ageing management guidelines and assisted Member States in the application of these guidelines. Results of this work are documented in Refs [10, 214, 314, 572–574].

Managing physical ageing of NPP components important to safety requires predicting and/or detecting when a plant component will have degraded to the point that the required safety margins are threatened and taking appropriate corrective or mitigative actions. The methodology identified for the management of ageing of NPP components important to safety consisted of three basic steps [573]:

(1) Selecting, from the safety perspective, as-built plant components in which ageing needs to be evaluated by assessing effects of ageing degradation on the ability of components to perform design functions and crediting existing programmes and activities that manage ageing effectively.
(2) Performing ageing management studies for selected components to determine appropriate ageing management actions. A two-phased method reviewed understanding, monitoring and mitigation of component ageing, and identified or developed effective and practical technology and practices for monitoring and mitigation.
(3) Using results of ageing management studies to take appropriate ageing management actions (i.e. improving existing operations and maintenance practices and design) and to improve relevant codes, standards and regulatory requirements.

To facilitate exchange of information and collaboration among international organizations (including utilities, architect/engineers and vendors), in 1989, the IAEA initiated pilot studies to develop a methodology to perform ageing studies on primary nozzles of the reactor pressure vessels, motor-operated isolation valves, CCBs and instrumentation and control cables inside containment. The objective of each pilot study was to identify dominant ageing mechanisms and develop an effective strategy for managing ageing effects caused by the mechanisms. The studies identified the following technical issues:

— Understanding of relevant ageing phenomena and how research results and OPEX are being applied to operational plants;
— Potential safety impacts of ageing if ageing mechanisms are not properly mitigated;
— Effectiveness of existing techniques used to monitor and to mitigate ageing degradation;
— Effectiveness of current procedures to predict future component performance;
— Need to develop methods and criteria to predict remaining service life of nuclear SSCs.

Phase 1 pilot studies were completed via technical committee meetings held in 1990 and 1991 and consisted of paper assessments of the current state of knowledge on age related degradation, its detection and mitigation and recommendations for Phase 2 studies. Results obtained from Phase 1 for the CCB [10] summarized (a) the design functions of a concrete containment and (b) structure service conditions during normal and abnormal operations, including external events such as tornadoes and seismic activities, and internal events such as design basis accidents. OPEX was compiled for CCBs and used to conclude that the performance of CCBs had been excellent during normal operations and periodic pressure tests. Typical deterioration identified included minor cases of concrete cracking, corrosion of steel reinforcement and larger than anticipated loss of prestressing force. Most of the degradation mechanisms identified had been anticipated in the design and were mitigated by using conservative design margins. A list was compiled of ageing mechanisms that could degrade structural components (concrete and steel) and ultimately impact CCB performance. Based on an understanding of these ageing mechanisms and observed deterioration effects, the study compiled a list of techniques to monitor the condition of various components. As a result of the Phase 1 findings, the IAEA determined that the following areas required additional research:

- Current experience and ageing management practices including in-service inspection, testing and repair of all containment structural components;
- State of the art repair techniques on concrete and other containment structural components;
- Crack mapping and acceptance criteria for repair including a correlation of crack category to repair methods;
- Condition indicators and guidelines for monitoring CCB ageing.

A separate coordinated research project was set-up to implement the Phase 2 pilot study on CCBs.

VI.2. COORDINATED RESEARCH PROJECT ON MANAGEMENT OF AGEING OF CONCRETE CONTAINMENT BUILDINGS

The coordinated research project on CCB ageing management had the objective of using the research and engineering expertise of participants to develop the technical basis for a practical AMP for CCBs. Specific objectives of the CCB project were to:

- Produce a summary of current ageing management practices and experiences for CCBs;
- Compile a state of the art report on concrete repair techniques and materials specifically applicable to NPP CCBs;
- Develop crack mapping and acceptance/repair guidelines applicable to NPP CCBs;
- Develop a set of practical condition indicators and associated guidelines for monitoring CCB ageing.

The project focused on the following:

- Compiling and evaluating information on age related degradation of CCBs as well as current ageing management methods and practices aimed at documenting CCB ageing;
- Identifying significant ageing mechanisms and effective methods for timely detection and mitigation of age related degradation.

The first step involved a literature survey and preparation of a questionnaire for a worldwide survey of NPP operators. Information from responses representing over 150 NPP units was compiled into a database, and then interpreted and evaluated [214]. Figure 192 presents a summary of responses received by country and unit type. As noted in Fig. 193, plant design dates ranged from pre-1965 to 1985 with in-service dates from 1964 to 1994.

Results obtained from the questionnaire provided information on OPEX at that time with respect to degradation, inspection and repair.

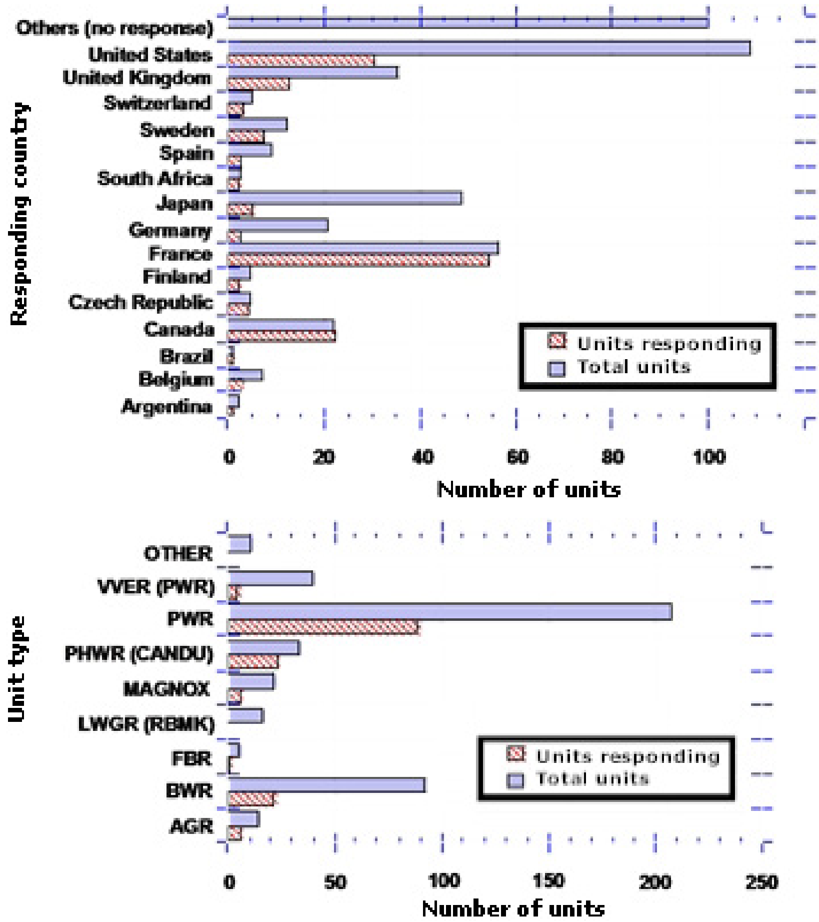
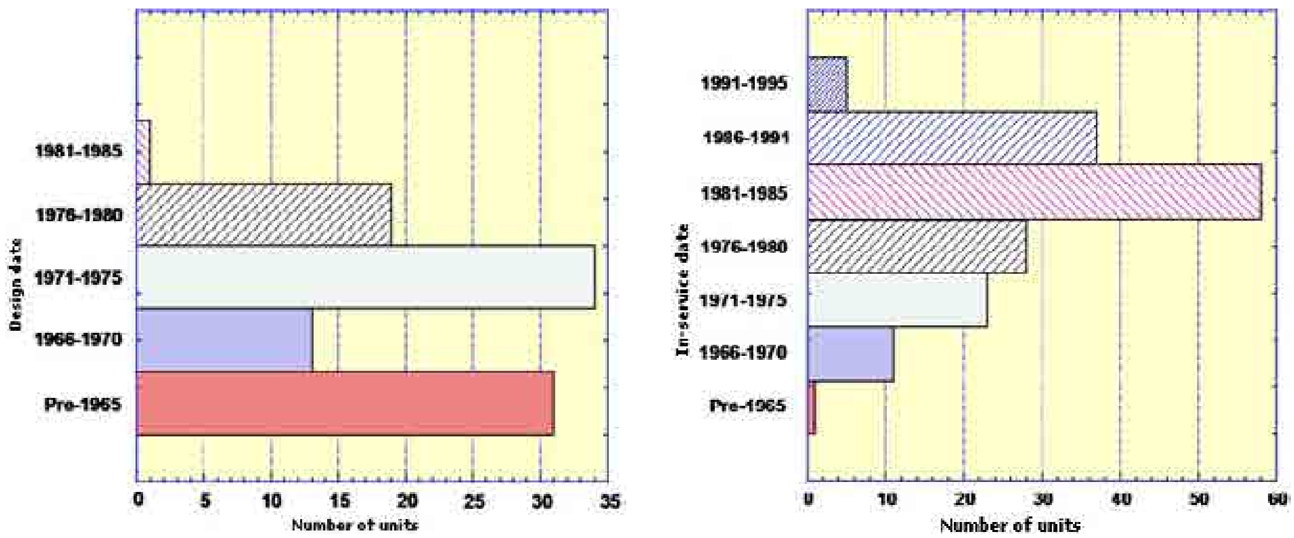


FIG. 192. Summary of countries responding to questionnaire and distribution with respect to unit type.



(a) Plant design dates

(b) Plant in-service dates

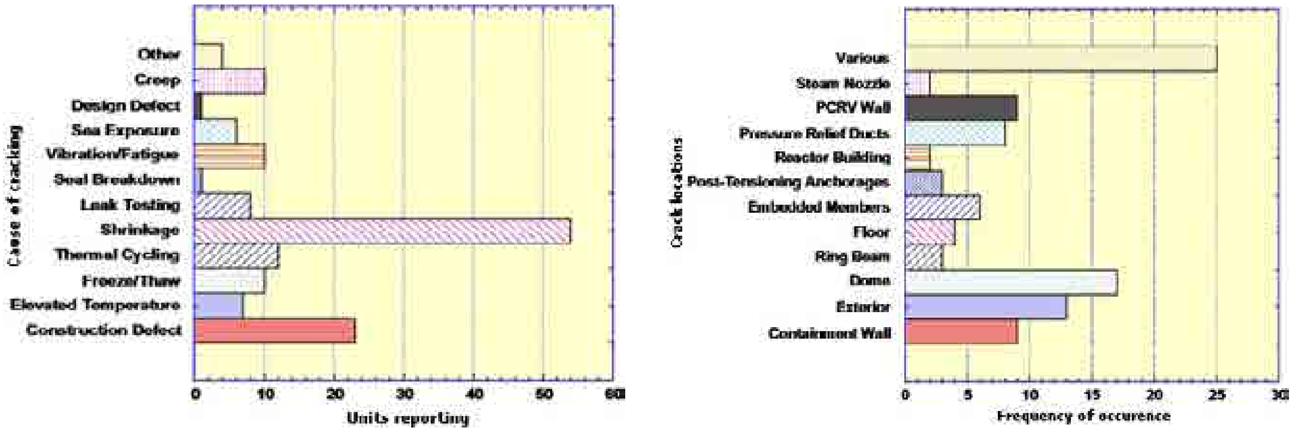
FIG. 193. Plant design and in-service dates.

VI.2.1. Degradation experience

In general, performance of CCBs was reported to have been very good. Table 37 provides a summary of reported degradation occurrences. These results indicate that cracking was the most frequently reported manifestation of degradation. Figure 194 provides a summary of causes and locations identified for concrete cracking. Figure 195 presents the unit age when cracking was first reported and corrective actions implemented, with 45% of incidences not requiring action.

TABLE 37. OCCURRENCES AND MANIFESTATIONS OF DEGRADATION FACTORS FOR CONCRETE MATERIALS

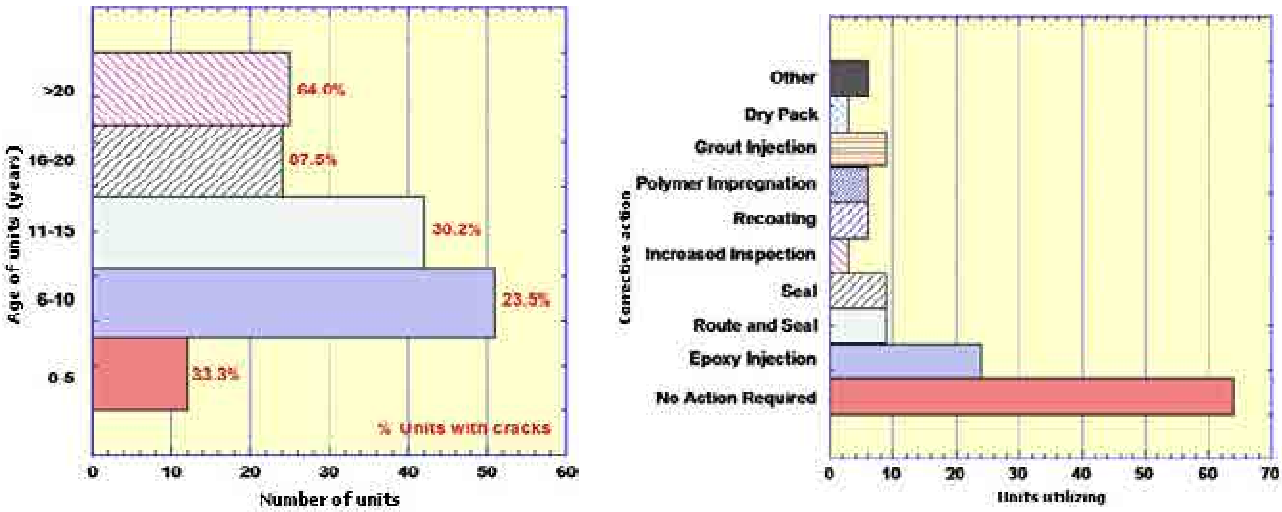
Degradation factor reported \ Manifestation of degradation factor	Cracking	Scaling	Delamination	Staining	Spalling	Efflorescence	Pop-out	Dusting	Voids/honeycomb	Increased permeability	Other	Subtotal of events
Freeze-thaw	10	—	—	—	7	—	1	—	2	—	—	20
Elevated temperature	7	—	—	—	—	—	—	—	—	—	—	7
Thermal gradient	12	—	—	—	2	—	1	—	—	—	—	15
Sea water exposure	6	—	—	—	—	—	—	—	—	—	—	6
Chemical attack	—	—	—	—	—	—	—	4	—	—	4	8
Leaching	—	—	—	3	—	11		—	—	—	—	14
Shrinkage	54	—	—	—	2	—	—	—	—	—	—	56
Sealant failure	1	—	—	—	—	1	—	—	—	—	—	2
Creep	10	—	—	—	—	—	—	—	—	—	—	10
Leakage test	7	—	—	—	—	—	—	—	—	3	—	10
Chloride penetration	—	—	1	—	2	—	—	—	—	—	—	3
Carbonation	—	—	—	1	—	—	—	—	—	—	—	1
Fatigue/vibration	10	—	—	—	—	—	—	—	—	—	—	10
Stray electrical currents	—	—	—	—	—	—	—	—	—	—	1	1
Construction defects	23	—	2	—	7	—	3	—	9	10	—	54
Design defects	1	2	1	7	2	—	2	1	—	—	—	16
Other	4	—	—	3	2	—	—	—	—	2	—	11
Subtotal of events	145	2	4	14	24	12	7	5	11	15	5	244



(a) Cause of concrete cracking and number of units identified.

(b) Crack locations and frequency of occurrence.

FIG. 194. Causes and locations of concrete cracking.



(a) Age at which concrete was identified and frequency of cracking.

(b) Corrective actions implemented and frequency of use.

FIG. 195. Age at which cracks were identified and corrective action implemented.

Table 38 presents a summary of reported degradation occurrences for steel materials. The most frequently reported manifestation of metallic material degradation was corrosion.

TABLE 38. OCCURRENCES AND MANIFESTATIONS OF DEGRADATION FACTORS FOR STEEL MATERIALS

Degradation factor reported \ Manifestation of degradation factor	Reinforcing steel corrosion	Prestressing tendon corrosion	Containment penetration	Prestress loss	Subtotal of events
Freeze-thaw	—	—	—	—	—
Elevated temperature	—	—	4	—	4
Sulphate attack	—	—	—	—	—
Seawater exposure	—	2	—	—	2
Chemical attack	—	—	—	—	—
Shrinkage	—	—	—	—	—
Sealant failure	—	—	2	—	2
Creep	—	—	—	—	—
Leakage test	—	—	—	—	—
Irradiation	—	—	—	—	—
Chloride penetration	4	—	—	—	4
Carbonation	—	2	—	—	2
Fatigue/vibration	—	—	—	—	—
Stray electrical currents	—	—	—	—	—
Construction defects	6	—	—	—	6
Design defects	—	1	1	2	4
Other	2	—	—	—	2
Subtotal of events	12	5	7	2	26

VI.2.2. Inspection experience

VI.2.2.1. Methods utilized by owners/operators

Although numerous non-destructive and destructive evaluation methods have been identified for detecting ageing, only a limited number are actually used, and then only on a restricted basis by plant owners/operators to gain information about the condition of CCBs. Table 39 was developed from information provided by a survey questionnaire sent to NPP owners/operators. Table 40 was developed using summaries of inspection techniques for the primary containment elements in terms of the percentage of responding units that apply these techniques, and the percentage of the responding owners who utilize them. As noted in these tables, visual inspection was the most common method used to inspect the different parts of the CCB.

Visual inspection of concrete structures was supplemented by crack mapping and instrumentation/NDT. Figure 196 presents visual inspection/crack mapping intervals summarized by percentage of units and owners responding to the survey questionnaire. More information on the crack mapping procedure used is provided in Fig. 197.

TABLE 39. PERCENTAGE OF COMMONLY USED INSPECTION TECHNIQUES FOR ALL CONTAINMENT ELEMENTS OUT OF TOTAL NUMBER OF RESPONDING UNITS

Inspection technique	Percentage of responding units utilizing the technique (%)				
	Concrete	Anchorage elements	Reinforcing steel	Prestressing steel	Liner/ penetrations
Visual inspection	84	73	26	26	53/—
Instruments	52	—	—	—	—/—
NDT	28	—	—	—	—/10
Coring	<1	—	—	—	—/—
Pull-out tests	—	1	—	—	—/—
Half-cell potential	—	—	6	—	—/—
Cover meter	—	—	3	—	—/—
Lift-off tests	—	—	—	18	—/—
Load cell	—	—	—	45	—/—
Mechanical tests	—	—	—	16	—/—
Grease inspection	—	—	—	16	—/—
Leakage rate test	—	—	—	—	53/30
Other tests	5	5	—	3	—/8
Percentage of units with this element (%)	99	65	99	65	68/100

TABLE 40. PERCENTAGE OF COMMONLY USED INSPECTION TECHNIQUES FOR CONTAINMENT ELEMENTS OUT OF THE TOTAL RESPONDING OWNERS USING THE TECHNIQUES

Inspection technique	Percentage of responding owners utilizing (%)				
	Concrete	Anchorage elements	Reinforcing steel	Prestressing steel	Liner/ penetrations
Visual inspection	59	34	27	22	51/—
Instruments	20	—	—	—	—/—
NDT	24	—	—	—	—/10
Coring	<2	—	—	—	—/—
Pull-out tests	—	5	—	—	—/—
Half-cell potential	—	—	5	—	—/—
Cover meter	—	—	7	—	—/—
Lift-off tests	—	—	—	27	—/—
Load cell	—	—	—	12	—/—
Mechanical tests	—	—	—	22	—/—
Grease inspection	—	—	—	22	—/—
Leakage rate test	—	—	—	—	51/37
Other tests	10	7	—	5	—/12
Percentage of owners with this element (%)	98	56	98	56	66/100

Table 41 shows that thermocouples and strain gauges were the most common types of instrumentation employed. Table 42 shows that leakage rate testing was the most frequently reported NDT method utilized. The amount of instrumentation contained in a CCB varied due to the particular Member State’s code requirements. In some countries, instrumentation was used mainly during the startup phase to verify compliance with construction codes and assumptions for the plant response (i.e. pre-operational testing). In other countries, monitoring of the plants continues through the operation phase.

Additional monitoring or testing was used to supplement visual inspections of the other elements. Half-cell potential testing of mild steel reinforcing systems was performed by about 6% of the units at intervals of 10 years. Where load cells were employed with prestressing systems, readings were taken at intervals that ranged from one month to five years, with the most common interval being two years. Leakage rate tests to investigate liner materials were conducted at intervals ranging from weekly to 10 years, with the most common interval being 40 months. Leakage rate tests of penetrations were performed at intervals of 1–2 years.

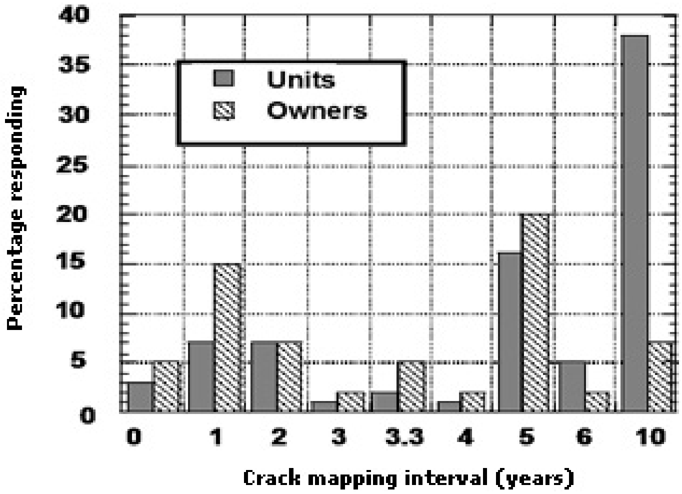


FIG. 196. Concrete visual inspection/crack mapping frequency summarized by percentage of units/owners responding.

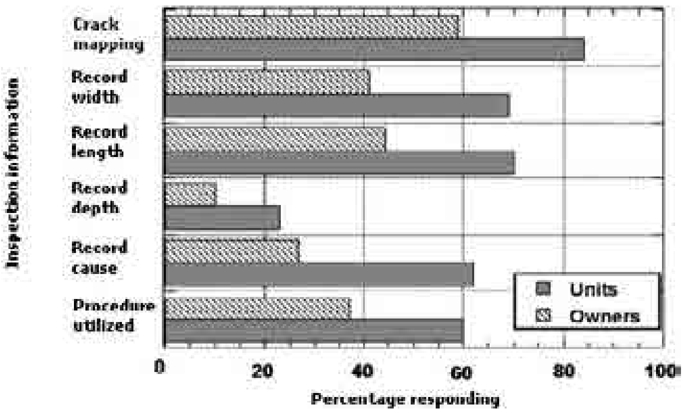


FIG. 197. Crack mapping information summarized by percentage of units/owners responding.

TABLE 41. INSTRUMENTATION TYPES EMPLOYED BY PERCENTAGE OF UNITS AND OWNERS UTILIZING THE INSTRUMENTATION

Instrumentation type	Percentage of units utilizing this instrumentation (%)	Percentage of owners utilizing this instrumentation (%)	Number used	
			Range	Median
Thermocouple	52	20	8–1335	27
Strain gauge	46	17	18–240	103
Stress cell	5	10	4–80	8
Humidity gauge	3	5	1–2	1
Invar wire	12	5	4	4
Other	42	10	—	—

TABLE 42. NON-DESTRUCTIVE TESTING METHODS EMPLOYED BY PERCENTAGE OF UNITS AND OWNERS UTILIZING THIS METHOD

Test type	Percentage of units utilizing this test (%)	Percentage of owners utilizing this test (%)
Leakage rate	26	24
Impact hammer	17	7
Pulse velocity	17	10
Permeability	5	5
Others	8	12

VI.2.2.2. Observations on methods used

Based on responses from 41 owners/operators representing 154 nuclear units, the following are general observations related to inspection of CCBs:

— Most commonly used methods of inspection (visual, leakage rate tests and prestressing system evaluations) were those prescribed by codes or regulations. Few owners used methods beyond those normally associated with code requirements.

— Inspections of various elements were performed at uniform intervals, with intervals and number of inspection methods varying between owners and countries. This reflects different refuelling cycles as well as regulatory requirements. Visual inspections of concrete were most commonly performed at five year intervals.

— Longer intervals (five and ten years) between inspections usually were associated with owners using instrumentation, supplemental inspection techniques or other methods to augment visual inspections and leakage rate testing requirements.

— There were few responses to the part of the questionnaire requesting information on specific inspection procedures, programmes and/or acceptance criteria. It could be that most of the inspections on CCBs (and other building structures relevant to plant safety) are often not formally structured. Plant owners/operators, therefore, have to rely heavily on the inspector’s experience and judgment.

VI.2.3. Repair experience

Results of the IAEA survey questionnaire sent to Member States, plant owners/operators [214], as well as those from a survey specifically addressing US utilities [235] indicated that many of the repair activities were associated with problems during initial construction (e.g. cracks, spalls and delaminations). As noted in Table 43, which summarizes concrete repair methods identified in the IAEA questionnaire [214], concrete cracking was the most common form of degradation reported. The most frequent approach used to address cracking was to take no action at present; however, when a repair action was implemented, epoxy injection was most frequently used. Little information was provided on materials used for repair, repair procedures or the durability of repairs. When performance of a repair procedure was evaluated, visual inspections were used. Additional information on remedial measure(s) implemented as a result of observed degradation was provided earlier in Table 8, which presented a summary of documented concrete problem areas.

TABLE 43. SUMMARY OF CONCRETE REPAIR METHODS UTILIZED

Type of repair action	Type of degradation (number of repair actions)					
	Voids/honeycomb	Cracking	Delamination	Spalling	Pop outs	Rebar corrosion
Epoxy injection	—	13	2	—	—	—
Routing and sealing	—	8	—	—	—	—
Drilling and plugging	—	1	—	—	—	—
Flexible sealing	—	7	—	—	—	—
Grout injection	—	1	1	—	—	—
Dry pack/crack	—	2	—	—	—	—
Polymer injection	—	1	—	—	—	—
Other/crack	—	6	1	—	—	—
Concrete replacement	—	—	—	5	4	2
Dry pack/spall	2	—	—	9	—	3
Sealers	—	—	—	6	—	—
Other/spall	—	—	—	6	—	2
Coating	—	1	1	—	—	1
Total repair actions	2	40	5	26	4	8

Appendix VII

SELECTED CASE STUDIES RELATED TO CONCRETE STRUCTURE REPAIR AND ANALYSIS

VII.1. Gentilly 1 Reactor Building Ring Beam Repair

Gentilly 1 is a permanently shut down 250 MW(e) CANDU NPP and is currently in the storage with surveillance (SWS) phase, the objective of which is to maintain structure integrity and allow for radioactive materials to decay. At approximately 30 years of age, Gentilly 1 underwent repair of the prestressed containment concrete ring beam.

During visual inspection of the containment structure in the 1990s, deterioration in the form of cracking, spalling and delamination of concrete was observed, predominantly in the ring beam. Evaluation of the structure to determine the cause and extent of deterioration consisted of the following activities:

— NDTs and laboratory analysis of concrete cores;
— Overcore concrete stress measurements;
— In situ stress measurement and inspection of post-tensioned cables.

For post-tensioning anchorages, concrete serves mainly a secondary function of protecting anchorage heads from corrosion. Despite the very poor concrete quality in the post-tensioned recesses, the anchorage heads were in reasonably good condition. Deterioration was mainly due to the alkali–aggregate reaction.

Water ingress had to be prevented to minimize future expansion due to AAR, and to protect reinforcing and prestressing steel from corrosion. Deteriorated ring beam concrete was replaced using cement based grout material. Subsequently, glass fibre reinforced polymer sheets were applied using the pattern shown in Fig. 198. Prequalifying work was carried out to test the repair method prior to application. Figure 199 shows the ring beam condition before and after the repair.

In order to provide assurance of structural integrity and to investigate suitability of instruments for long term monitoring, instrumentation was embedded within the new concrete repair patches, and also mounted on the exposed surface of the glass fibre reinforced polymer material. Instrumentation included vibrating wire strain gauges and fibre optic sensors for measuring local and global deformation as well as thermocouples for measurements of temperature. Assessment of instrumentation data obtained periodically as part of the AMP has proved it to be reliable, attesting to satisfactory repair performance [483].

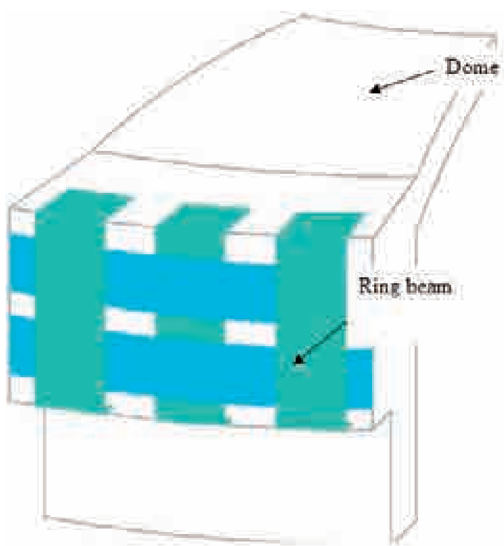


FIG. 198. Application pattern of glass fibre reinforced polymer sheets to the ring beam.

(a) Before repair, 1999. (b) After repair, 2008.

FIG. 199. Ring beam before and after repair (courtesy of Candu Energy).

VII.2. REPAIR OF COOLING WATER INTAKE/CIRCULATION STRUCTURES

Cooling water intake/circulation structures in power plants located at a seacoast typically consist of pumphouse structures, water delivery conduits and discharge structures and conduits. These structures frequently experience accelerated deterioration due to rebar corrosion resulting from chloride intrusion into the concrete. A comprehensive maintenance programme has been implemented at a plant on the US Pacific coast to address and arrest corrosion in these complex structures [713].

In developing the plan for repair and maintenance, it was necessary to determine both the rate at which corrosion was proceeding and its effects on the safety of the structures. A condition survey involving detailed inspection/testing was implemented to address corrosion activity. The objective of the inspection and testing was to establish baseline data regarding the extent of deterioration and to serve as a basis for future monitoring and service life prediction. It consisted of the following:

— Visual inspection;
— Hammer soundings to identify concrete delaminations;
— Rapid chloride testing to determine soluble chloride content;
— Half-cell potential tests to detect corrosion activity;
— Impact echo testing to detect concrete defects;
— Polarization resistance to estimate corrosion rates.

Corrosion deterioration effects on structural integrity were evaluated by identification of structurally critical areas, evaluation of reductions in safety margins in the degraded areas, and categorization of damaged areas for repair prioritization (e.g. critical in terms of structural adequacy and safety, critical in terms of rapid corrosion occurrence and high economic impact areas where repair delays will cause substantially higher costs, non-critical areas showing corrosion damage and non-critical areas with corrosion absent). Structurally, critical areas were repaired as soon as practicable.

Patching repair was the basic method employed to restore structural integrity in local areas in the corrosion damaged structures. Selection of patch materials, surface preparation and curing are important to implementing an effective repair. To ensure durable repairs, zinc silicate primer and zinc ribbon attachments were used at the perimeters of the patched areas to eliminate or mitigate the reversal of galvanic cells. Epoxy primer was applied to the substrate surface to minimize any adverse interaction between the patch and its substrate. Well constructed dense concrete with pozzolan addition was used as the patch material, followed by proper curing and sealing of the patch perimeter.

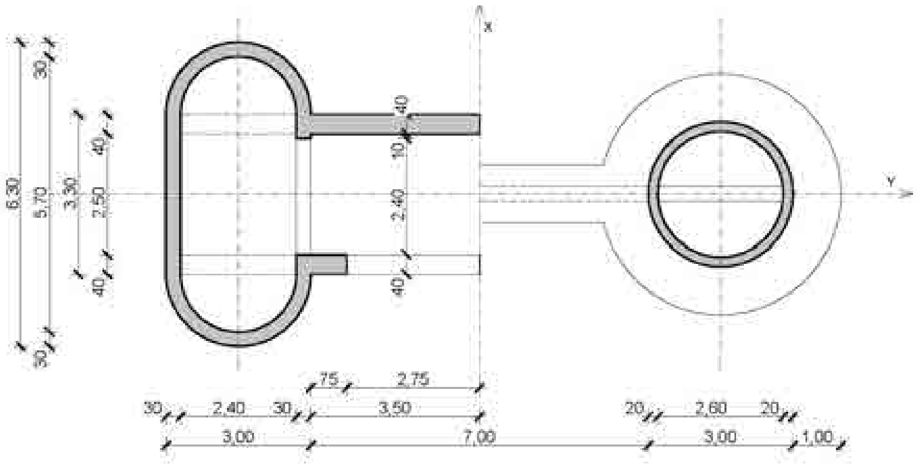
Since the intake structures were located in a seawater environment, chloride permeation had progressed globally to contaminate extensive portions of the structure resulting in general deterioration. Additional measures were implemented to control deterioration including keeping the pump house structure dry to maintain concrete resistivity at a higher level than that associated with steel corrosion, choking off the oxygen supply to cathodic areas through application of epoxy coatings and cathodic protection of reinforcing steel in appropriate areas. A general monitoring programme was implemented after completion of repairs consisting primarily of visual inspections and hammer soundings supplemented by other tests as deemed necessary, primarily in local areas. Objectives of the monitoring programme were to confirm trends and progression of plant corrosion, detect severely damaged areas as early as possible and develop guidelines for planning and prioritization of future repairs.

VII.3. REPAIRING OF A GROUP OF VENTILATION CHIMNEYS AT PAKS NPP

The 100 m high ventilation chimneys at the Paks NPP were in very bad condition due to poor concrete technology and insufficient compaction during construction. This was a serious threat to structure stability, and retrofitting and strengthening was necessary [714].

According to structural analysis, the structures were safe against wind load but not earthquake load. Analysis also showed that, instead of building new chimneys, retrofitting the old ones was feasible.

The chimneys were 15–20 years old and built using sliding formwork technology (Fig. 200). Failures included falling concrete parts, appearance of holes through the wall, and starting of rebar corrosion (Figs 201 and 202).



cross-section at +19.00m *cross-section at 100m*

FIG. 200. Chimney cross-sections.

FIG. 201. Conditions before repair.

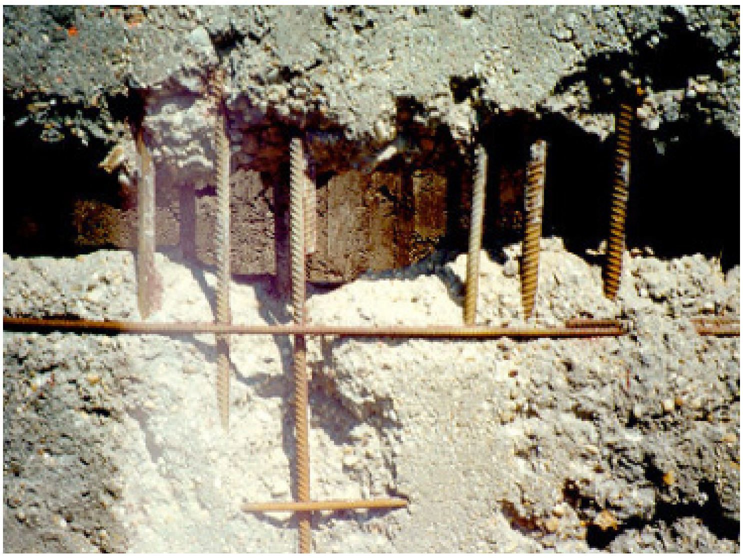


FIG. 202. Conditions before repair.

The loose, not compacted, concrete parts were injected first with micro cement. Retrofitting and strengthening was done using 4 cm thick reinforced concrete on both sides, which were connected to each other. Rebar (Ø5/10 and Ø5/20) was used in hoop and vertical directions, respectively. A flexible repair system was adapted to the circumstances and included a highly organized quality control programme, which significantly contributed to the success of the retrofitting and strengthening work (Figs 203 and 204).

FIG. 203. Condition during repair.

FIG. 204. Condition after repair.

VII.4. Service Lifetime Substantiation Examples using the Finite Element Model for WWER-1000 NPP Containment Design Life Analysis

VII.4.1. Introduction

In connection with lifetime extension reviews of the Novovoronezh (Nov) NPP Unit 5 containment, work on substantiation of containment service lifetime extension for an additional 30 years was performed by CMSLM Ltd. This was the first time this had been done in the Russian Federation.

The Nov Unit 5 CCB was constructed from prestressed reinforced concrete between April 1976 and October 1978. It contains a cylindrical part and a low pitched dome. The inner containment volume is 85 000 m^3; diameter is 45 m and height is 76 m (Fig. 205). The cylindrical part wall thickness is 1.2 m, with a low pitched dome of 1.1 m. Metal cladding thickness is 8 mm.

Present reinforced concrete knowledge doesn’t allow precise evaluation of durability and service life. It is only possible to establish whether there is additional service life left. For the service lifetime evaluation of the Nov Unit 5 CCB, experimental work was performed to evaluate its technical condition and to justify additional lifetime calculations.

VII.4.2. Normative base

The main regulatory document for containment service lifetime evaluation is RD EO 0538-04, Methods for Containment Service Lifetime Substantiation of NPP with WWER-1000, which was developed by specialists from CMSLM Ltd, together with Atomenergoprojekt, under contract with Rosenergoatom.

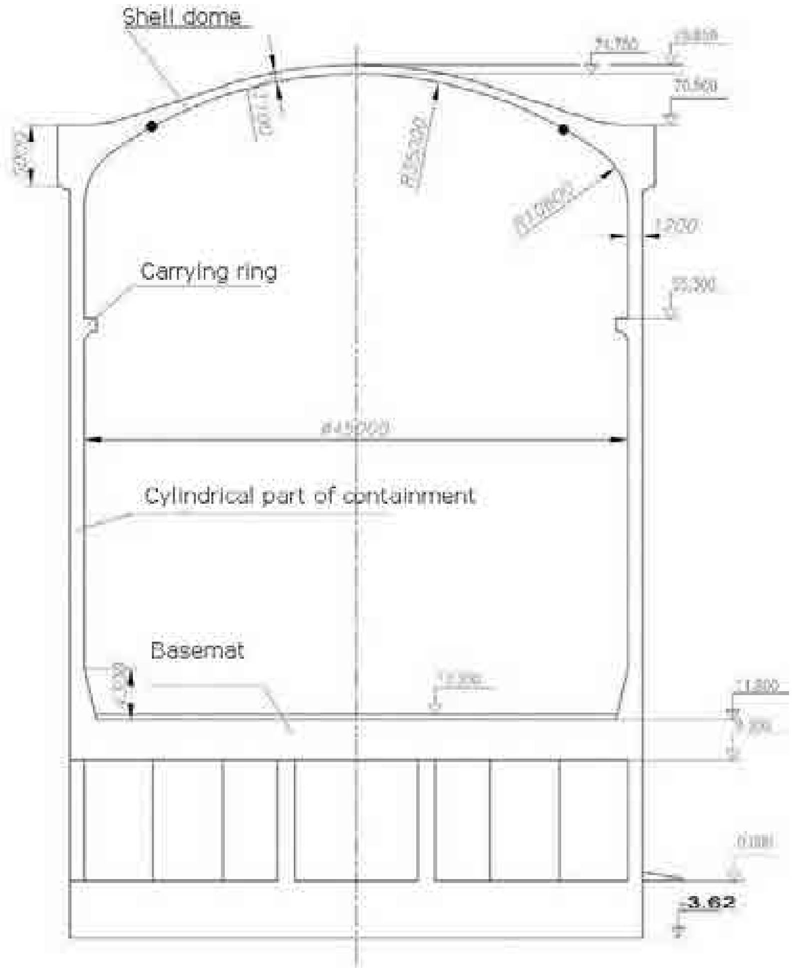


FIG. 205. Novovoronezh NPP Unit 5 containment dimensions (adapted from image courtesy of CMSLM Ltd).

VII.4.3. Main stages of work

The principal stages of work for evaluating service life are:

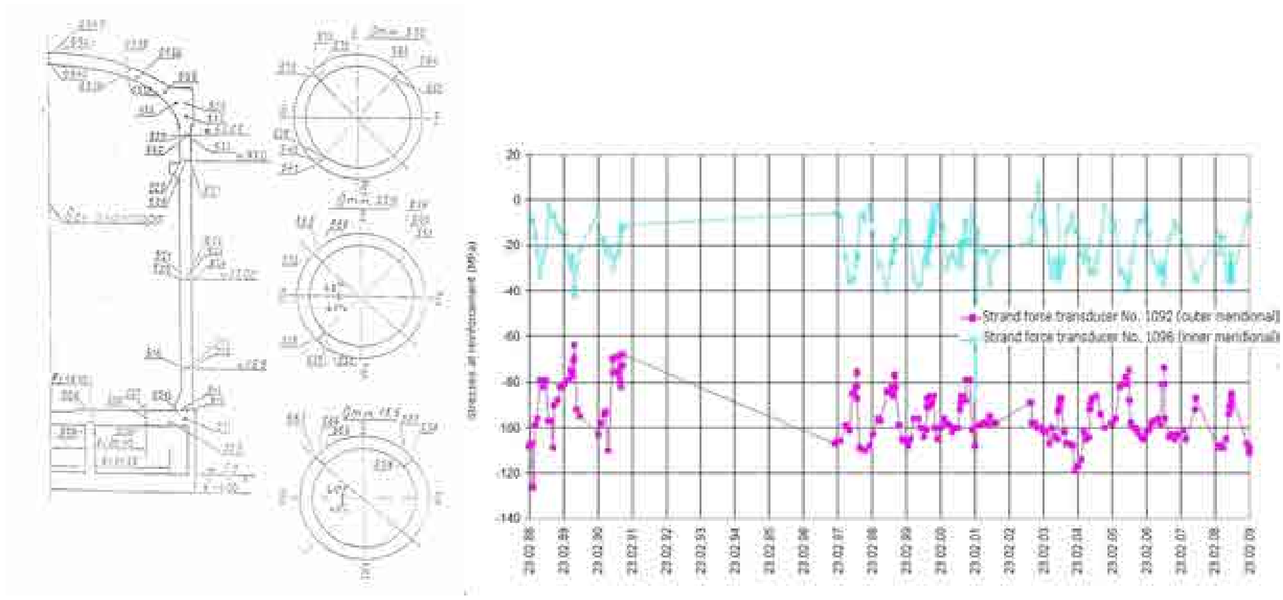
— Collection and analysis of initial data about the containment's state;
— Determination of design analysis for most loaded areas for which visual and instrumental control are necessary;
— Determination of areas for material sample collection for laboratory investigations;
— Elaboration of the investigation work programme;
— Carrying out visual and instrumental, laser scanning and laboratory investigations;
— Performing additional measurements, including laser scanning of the CCB in winter;
— Carrying out calculations taking into account performed inspections;
— Development of a conclusion and engineering solution project for service lifetime extension for an additional 30 years.

VII.4.4. Collection and analysis of initial data

For CCB condition evaluation, compilation and analysis of the following data are performed:

— Project, construction, assembly and operating documentation (planned and actual geometrical containment parameters, layout of prestressed concrete strands and rebar, actual number of installed prestressed concrete strands), including design strength calculations and load cases, among other things;

— Layout of control and measuring equipment installed in containment (Fig. 206(a));
— Initial and regular inspection results for the operating lifetime;
— Control and measuring equipment indications from installation until present (Fig. 206(b));
— Temperature data inside and outside of containment;
— Force sensor HB005 indications (Fig. 207), installed at prestressed concrete strands;
— Information about repairs, replacement and reconstructions of containment elements;
— Data on actual load cases at containment elements;
— Data on controlling preventive measures.



(a) (b)

FIG. 206. (a) Layout for control and measuring equipment sensor; (b) plot for meridional and circular stresses at reinforcement for an operating time of 30 years (courtesy of CMSLM Ltd).

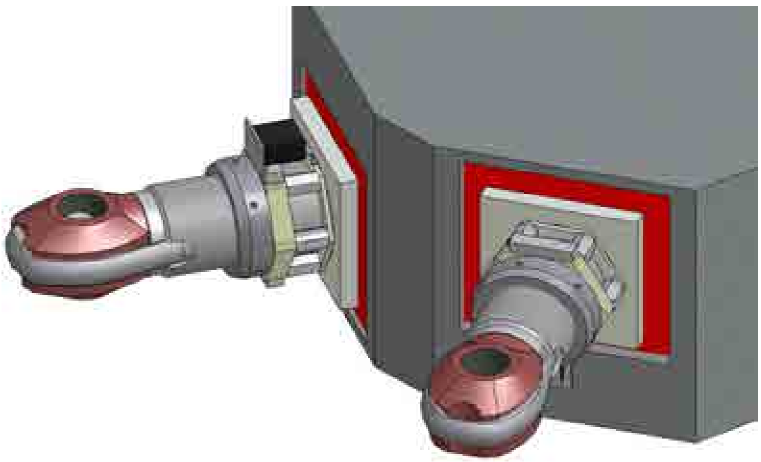


FIG. 207. Force sensors HB005, installed at prestressed concrete strands.

Based on data about CCB state, the next stage, detailed numeric calculation using a volumetric calculation finite element model, was performed.

VII.4.5. Containment calculation model

To optimize the use of instruments on such a large CCB, a calculation of the stress-strain state during construction was needed to determine the most loaded areas.

Unit 5 CCB has number of features that were taken into account while performing stress-strain state calculations. For example, the CCB is not axisymmetric because of two large (4 m and 3 m diameter) technological openings and significantly complex external thickenings. The prestressing loading patterns are also not axisymmetric, and include a very complex character in the region of the technological openings. These features led to the need for creating a volumetric containment finite element model.

Figure 208 shows a general view of the finite element model for the Nov Unit 5 CCB. This model was created for stress-strain state analysis and consists of 366 992 elements, 402 378 nodes (more than 1 200 000 degrees of freedom).

The next items taken into account in the calculations were:

- — Non-uniform structure of CCB materials;
- — Prestressed concrete strand prestressing load, taking into account location in the uniform area and in the area of large containment openings;
- — Different values of strand loads and decreases in tensile force along strand length, which depend on the frictional coefficient value, strand bending angle and load decreases at strand anchorages;
- — Formation of the stress-strain condition during the construction and prestressing periods;
- — Apparent changes in stress-strain conditions because of strand stress relaxation during operation;
- — Load from the CCB's own weight;
- — Results of multiannual inspections and investigations of physical mechanical properties and CCB elements;
- — Action from external and internal temperature loads on the CCB.

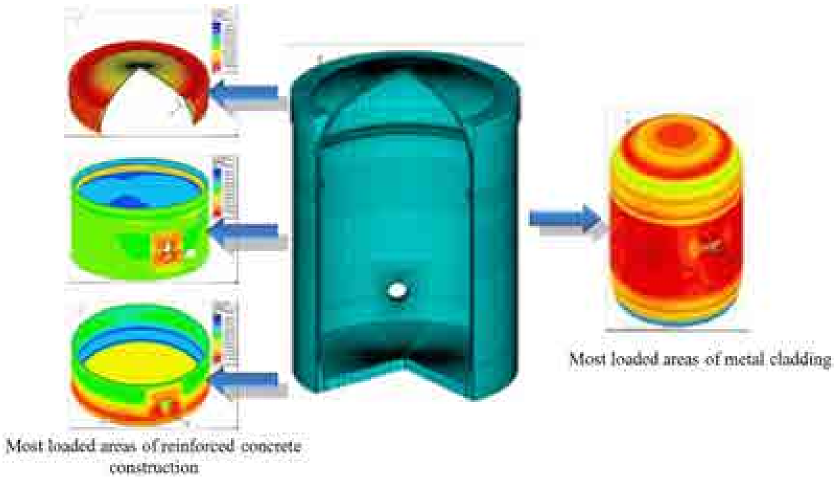


FIG. 208. Containment finite element model (courtesy of CSLM Ltd).

Calculation results showed that the joint area between the dome and carrying ring needed to be inspected because this area had maximum compressive stresses, which increase during operation (due to concrete creep and shrinkage). The entire cladding, and particularly cladding joining to concrete inside the CCB, needed to be inspected.

The next areas that needed additional attention were determined to be:

- — Middle part of the 3 m diameter (emergency lock);
- — Bottom part of the 4 m opening (default gateway);
- — Point where the cylinder joins the basemat;
- — Point where the cylinder joins the carrying ring under the crane console.

VII.4.6. Containment integrated survey work programme development

A CCB inspection programme for technical state evaluation and residual lifetime evaluation was developed based on database analysis and results of numeric calculations. An in situ control strategy was determined as follows:

- — A list of areas for inspection is made;
- — Methods and tools for in situ control are chosen;
- — In situ control work scope is determined.

VII.4.7. Containment inspection

CCB inspection consisted of two main stages:

- — Stage 1: in situ inspection in summer;
- — Stage 2: additional inspection in winter.

VII.4.7.1. Stage 1

The first CCB inspections were performed during the summer to record visible defects and to adjust the complex inspection programme. Inspection zones need to be adjusted to take accessibility into account.

Instrument inspection consisted of the following:

- — Technical state determination (physical properties and entirety of inspected elements) of reinforced concrete construction by non-destructive control methods and by laboratory specimen investigations;
- — Technical condition determination (physical properties and entirety of inspected elements) of metal construction;
- — Determination of prestressing system's technical condition, which includes determination of physical, mechanical and rheologic characteristics of prestressed concrete strands;
- — Laser scanning of the inner and outer CCB surface (Fig. 209).

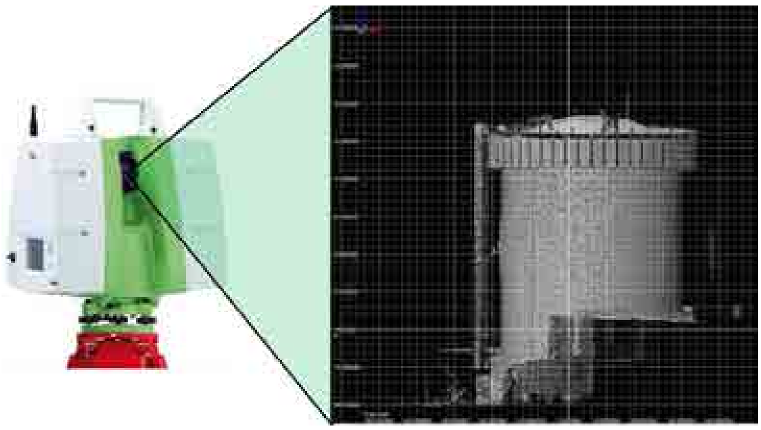


FIG. 209. Containment laser scanning (courtesy of CMSLM Ltd).

VII.4.7.2. Stage 2

To strengthen and complete the stress-strain condition calculation model, it was necessary to perform CCB inspections during the winter and to perform analysis using the actual temperature deformations observed. For this, laser scanning was performed during both summer and winter (Fig. 210).

Calculation analysis was performed on indicated areas where, during the winter, (at maximum temperature gradient) tensile stresses appear on the CCB surface (Fig. 23 from Section 4.3.1 of this publication).

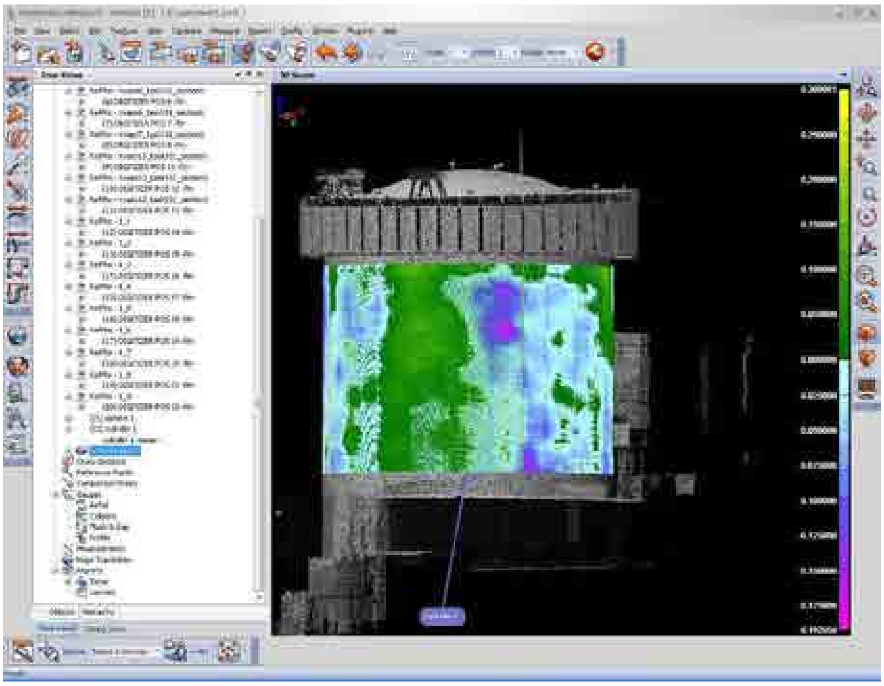


FIG. 210. Determination of geometrical dimensions changing depending on season.

VII.4.7.3. Check calculation taking into account performed investigation results

Calculations of Nov Unit 5 stress and strain for maximum design basis accident conditions were performed, taking into account possible concrete cracks in the basemat. The following factors and loads were taken into account:

— Weight of CCB;
— Load from prestressing system;
— Load from accident internal pressure;
— Load from non-uniform, non-stationary thermal fields;
— Load from cladding bracing;
— Load from prestressing system strand self-tension when containment deforms;
— Seismic loads.

A containment stress-strain calculation was performed for the application of the maximum possible accident, non-stationary load (Fig. 211) taking into account a crack in the basemat foundation and the most unfavourable initial state.

Analysis of obtained results shows that at the mean value of prestressing system force, formation of through cracks in CCB walls does not occur, which justifies the CCB's operational serviceability.

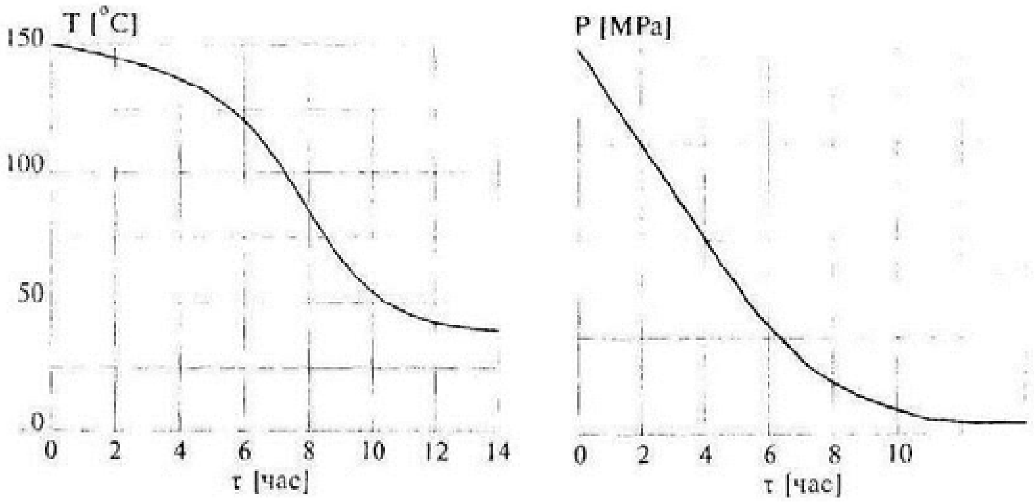


FIG. 211. Temperature and pressure changes versus shear stress within containment during maximum possible accident condition (courtesy of CMSLM Ltd).

VII.4.7.4. Conclusion and lifetime extension of 30 years

Analysis of obtained initial data from Nov Unit 5 CCB enabled an evaluation of changes in stress-strain conditions occurring during operation. Volumetric models were created, and methods were developed for stress-strain state calculations that include non-linear effects connected with concrete creep, shrinkage and cracking. A calculation of results at operational loads allows for the determination of most loaded areas, for which visual and instrumental control, and laboratory investigations, are needed.

As a result of visual and instrumental control and laboratory investigations, data regarding the CCB's state and its material properties were obtained. This established that its material properties meet the requirements of valid regulatory documents. Laser scanning of inner and outer CCB surfaces allows for the determination of actual geometrical building dimensions and their seasonal changes (winter to summer) due to temperature. Calculations at operational and accidental loads that were performed taking into account inspection results showed that operational serviceability of the CCB is ensured.

Based on calculations and experimental evaluation of the technical state of the CCB, it was established that the building currently meets requirements of operational, engineering design and regulatory documentation, and that an additional operating life of the Nov Unit 5 containment is justified.

REFERENCES

[1] INTERNATIONAL ATOMIC ENERGY AGENCY, Ageing Management for Nuclear Power Plants, IAEA Safety Standards Series No. NS-G-2.12, IAEA, Vienna (2009).

[2] INTERNATIONAL ATOMIC ENERGY AGENCY, Assessment and Management of Ageing of Major Nuclear Power Plant Components Important to Safety: Concrete Containment Buildings, IAEA-TECDOC-1025, IAEA, Vienna (1998).

[3] ELECTRIC POWER RESEARCH INSTITUTE, Common Ageing Terminology, EPRI, Palo Alto, CA (1993).

[4] OECD NUCLEAR ENERGY AGENCY, Glossary of Nuclear Power Plant Ageing Terminology, OECD, Paris (1999).

[5] INTERNATIONAL ATOMIC ENERGY AGENCY, IAEA Safety Glossary, Terminology Used in Nuclear Safety and Radiation Protection, 2007 Edition, IAEA, Vienna (2007).

[6] INTERNATIONAL ATOMIC ENERGY AGENCY, Safety of Nuclear Power Plants: Commissioning and Operation, IAEA Safety Standards Series No. SSR-2/2, IAEA, Vienna (2011).

[7] INTERNATIONAL ATOMIC ENERGY AGENCY, The Management System for Facilities and Activities, IAEA Safety Standards Series No. GS-R-3, IAEA, Vienna (2006).

[8] INTERNATIONAL ATOMIC ENERGY AGENCY, The Management System for Nuclear Installations, IAEA Safety Standards Series No. GS-G-3.5, IAEA, Vienna (2009).

[9] NAUS, D., Primer on Durability of Nuclear Power Plant Reinforced Concrete Structures: A Review of Pertinent Factors, Rep. NUREG/CR-6927 ORNL/TM-2006/529, Oak Ridge National Laboratory, Washington, DC (2007).

[10] INTERNATIONAL ATOMIC ENERGY AGENCY, Pilot Studies on Management of Ageing of Nuclear Power Plant Components, Results of Phase I, IAEA-TECDOC-670, IAEA, Vienna (1992).

[11] FEDERATION INTERNATIONALE DE LA PRECONTRAINTE, FIP State of the Art Report: Concrete Containments, FIP/3/4, FIP, London (1978).

[12] FEDERATION INTERNATIONALE DE LA PRECONTRAINTE, An International Survey of In-Service Inspection Experience with Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors FIP/3/5, FIP, London (1982).

[13] HESSHEIMER, M., DAMERON, R., Containment Integrity Research at Sandia National Laboratories, Rep. NUREG/CR-6906, Sandia National Laboratories, Washington, DC (2006).

[14] VISOLA ELECTRICAL INSULATION TECHNOLOGY INC., Electrical Penetration Assemblies for VVER Type Nuclear Power Plants (2012), http://visola.hu/en/fields-of-activity/manufacturing/cable-penetrations/schott

[15] KLAMERUS, E., BOHN, M., WESLEY, D., KRISHNASWAMY, C., Containment Performance of Prototypical Reactor Containments Subjected to Severe Accident Conditions, Rep. NUREG/CR-6433, Sandia National Laboratories, Washington, DC (1996).

[16] LAMBERT, L., PARKS, M., Experimental Results from Containment Piping Bellows Subjected to Severe Accident Conditions, Rep. NUREG/CR-6154, Sandia National Laboratories, Washington, DC (1994).

[17] NUCLEAR REGULATORY COMMISSION, Advisory Committee on Reactor Safeguards Plant License Renewal Subcommittee, Official Transcript of Proceeding Nuclear Regulatory Commission, Work Order NRC-577, Neal R. Gross and Company Inc., Court Reporters and Transcribers, Washington, DC (2012).

[18] INTERNATIONAL ATOMIC ENERGY AGENCY, Spent Fuel Performance Assessment and Research, IAEA-TECDOC-1343, IAEA, Vienna (2003).

[19] ROSENERGOATOM, The Main Equipment of the Reactor Compartment: Tutorial, Balakovo, Rosenergoatom (2000).

[20] ELECTRIC POWER RESEARCH INSTITUTE, Program on Technology Innovation: Assessment of Needs for Concrete Research in the Energy Industry, EPRI TR-1022373, EPRI, Palo Alto, CA (2010).

[21] ELECTRIC POWER RESEARCH INSTITUTE, Nuclear Maintenance Applications Center: Guideline for Cooling Tower Inspection and Maintenance, Technical Report 1021060, EPRI, Palo Alto, CA (2011).

[22] ELECTRIC POWER RESEARCH INSTITUTE, Class I Structures License Renewal Industry Report, Revision 1, EPRI TR-103842, EPRI, Palo Alto, CA (1994).

[23] AMERICAN WATER WORKS ASSOCIATION, Concrete Pressure Pipe (M9) manual, 3rd edn, AWWA, Denver, CO (2008).

[24] NUCLEAR REGULATORY COMMISSION, Inspection of Structures, Passive Components, and Civil Engineering Features at Nuclear Power Plants, Inspection Procedure 62002, NRC Inspection Manual, NRC, Washington, DC (1996).

[25] MEINHEIT, D., HEIDBRINK, F., Behavior of Drilled-In Expansion Anchors, Concr. Int., 7 4 (1985) 62–66.

[26] HILTI CORPORATION, Hilti Anchoring Solutions for the Nuclear Industry, https://www.us.hilti.com/medias/sys_master/h4e/hfd/9092423155742.pdf?mime=application%2Fpdf&realname=Nuclear+Anchor+Flyer.pdf

[27] DEUTSCHE INDUSTRIE NORMEN, Machine Foundations: Flexible Structures that Support Machines with Rotating Elements, DIN 4024 Part 1, DIN, Berlin (1988).

[28] AMERICAN CONCRETE INSTITUTE COMMITTEE 116, Cement and Concrete Terminology, ACI 116R-85, ACI, Detroit, MI (1985).

[29] INTERNATIONAL ATOMIC ENERGY AGENCY, Design of Reactor Containment Systems for Nuclear Power Plants, IAEA Safety Standards Series No. NS-G-1.10, IAEA, Vienna (2004).

[30] NACE INTERNATIONAL, Cathodic Protection to Control External Corrosion of Concrete Pressure Pipelines and Mortar-Coated Steel Pipelines for Water or Waste Water Service, SP0100-2008 (formerly RP0100), Houston, TX (2008).
[31] AMERCAN CONCRETE INSTITUTE COMMITTEE 221, Guide for Use of Normal Weight and Heavyweight Aggregates in Concrete, ACI 221R-96, American Concrete Institute, Farmington Hills, MI (2001).
[32] AMERICAN CONCRETE INSTITUTE COMMITTEE E-701, Cementitious Materials for Concrete, ACI Education Bulletin E3-01, American Concrete Institute, Farmington Hills, MI (2001).
[33] AMERICAN CONCRETE INSTITUTE COMMITTEE 212, Chemical Admixtures for Concrete, ACI 212.3R0-4, American Concrete Institute, Farmington Hills, MI (2004).
[34] NUCLEAR REGULATORY COMMISSION, Requirements for Renewal of Operating Licenses for Nuclear Power Plants, 10 CFR 54, US Govt Printing Office, Washington, DC (2012).
[35] NUCLEAR REGULATORY COMMISSION, Standard Format and Content for Application to Renew Nuclear Power Plant Operating Licenses, Regulatory Guide 1.188, NRC, Washington, DC (2005).
[36] BRITISH STANDARDS INSTITUTION, The Structural Use of Reinforced Concrete in Buildings, CP 114, Amendment 1, B, BSI, London (1965).
[37] NUCLEAR REGULATORY COMMISSION, NRC Information Notice No. 98-26 Settlement Monitoring and Inspection of Plant Structures Affected by Degradation of Porous Concrete, NRC, Washington, DC (1998).
[38] TROXELL, G., et al., Composition and Properties of Concrete, McGraw-Hill, New York (1968).
[39] INTERNATIONAL ATOMIC ENERGY AGENCY, Construction Technologies for Nuclear Power Plants, IAEA Nuclear Energy Series No. NP-T-2.5, IAEA, Vienna (2012).
[40] AMERICAN CONCRETE INSTITUTE, State-of-the-Art Report on High-Strength Concrete, ACI 363R-92, ACI Committee 363 Report, ACI, Farmington Hills, MI (1992).
[41] MORISHIMA, H., ODAKA, T., Concrete Structure and Nondestructive Testing, JR East Technical Review No. 2 (2003).
[42] INTERNATIONAL FEDERATION FOR STRUCTURAL CONCRETE, Nuclear Containments, State-of-the-Art Report, Bulletin No. 13, FIB, Lausanne (2001).
[43] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete, ASTM A416 / A416M – 12a, ASTM, West Conshohocken, PA (2012).
[44] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete, ASTM A421 / A421M – 10, ASTM, West Conshohocken, PA (2010).
[45] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Specification for Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service, ASTM A516 / A516M – 10, ASTM, West Conshohocken, PA (2010).
[46] REN, W., Gen IV Materials Handbook Functionalities and Operation, ORNL/TM-2009/285, Oak Ridge Natl Lab., TN (2009).
[47] OLAND, C., NAUS, D.J., Structural Material Information Center for Presentation of the Time Variation of Materials Properties, ORNL/TM-90-22, Oak Ridge Natl Lab., TN (1990).
[48] CHO, M.-S., et al., "Structural Life Management System of Nuclear Power Plant in Korea" (Proc. of AMP 2010 Int. Workshop on Ageing Management of Nuclear Power Plants and Waste Disposal Structures), Toronto (2012).
[49] AL-NESHAWY, F., et al., Development of Database for Concrete Facades' Condition Survey, European Symposium on Service Life and Serviceability of Concrete Structures, ESCS-2006, Finland (2006).
[50] SISTONEN, E., PUTTONEN, J., Durability Study of Hot-dip Galvanized Reinforcement Bars in Carbonated and Chloride-contaminated Concrete, 3rd FIB International Congress, Precast/Prestressed Concrete Institute, Washington, DC (2010).
[51] LEIVO, M., et al., Effect of Interacted Deterioration Parameters on Service Life of Concrete Structures in Cold Environments, Laboratory Test Results 2009 – 2011, VTT Research Report, VTT-R-09119-11, Espoo (2011).
[52] KARI, O.-P., PUTTONEN, J., Modelling the Ageing of Concrete Under Nuclear Waste Disposal Conditions (Proc. of 2010 FIB Congress and PCI Convention Bridge Conf.), Precast/Prestressed Concrete Institute, Washington, DC (2010).
[53] UNITED STATES ARMY CORPS OF ENGINEERS, Evaluation and Repair of Concrete Structures, CECW-EG EM-1110-2-2002, Precast/Prestressed Concrete Institute, Washington, DC (1995).
[54] ELECTRIC POWER RESEARCH INSTITUTE, Comparison of Alternate Cooling Technologies for US Power Plants, Economic, Environmental, and Other Tradeoffs, Technical Report 1005358, EPRI, Palo Alto, CA (2004).
[55] UNITED STATES DEPARTMENT OF ENERGY, Use of Treated Municipal Wastewater as Power Plant Cooling System Makeup Water: Tertiary Treatment Versus Expanded Chemical Regimen for Recirculating Water Quality Management, Project No. DE - NT0006550, USDOE, Washington, DC (2009).
[56] UNITED STATES DEPARTMENT OF ENERGY, Water Requirements for Existing and Emerging Thermoelectric Plant Technologies, DOE/NETL-402/080108, August 2008 (April 2009 Revision), USDOE, Washington, DC (2009).
[57] ELECTRIC POWER RESEARCH INSTITUTE, Closed-cycle Cooling System Retrofit Study: Capital and Performance Cost Estimate, Report 1022491, EPRI, Palo Alto, CA (2011).
[58] BURNS, J.M., The Impacts of Retrofitting Cooling Towers at a Large Power Station, EPRI-TR-104867, (Proc. Cooling Tower and Advanced Cooling Systems Conf.), Electric Power Research Institute, Palo Alto, CA (1995).
[59] MAILVAGANAM, N.P., Repair and Protection of Concrete Structures, CRC Press, Boca Raton, FL (1992).

[60] ELECTRIC POWER RESEARCH INSTITUTE, Program on Technology Innovation: Concrete Civil Infrastructure in United States Commercial Nuclear Power Plants, EPRI Product ID 1020932, EPRI, Palo Alto, CA (2010).
[61] ELECTRIC POWER RESEARCH INSTITUTE, General Outline for Conducting Quality Inspections and Tests of Concrete Placement at Nuclear Facilities, Report 3002000520, EPRI, Palo Alto, CA (2013).
[62] NUCLEAR REGULATORY COMMISSION, NRC Information Notice No. 2008-17: Construction Experience with Concrete Placement, NRC, Washington, DC (2008).
[63] ROYAL ACADEMY OF ENGINEERING, Nuclear Construction Lessons Learned Guidance on Best Practice: Concrete, The Royal Academy of Engineering, London (2012).
[64] RADIATION AND NUCLEAR SAFETY AUTHORITY, Management of Safety Requirements in Subcontracting during the Olkiluoto 3 Nuclear Power Plant Construction Phase, Investigation Report 1/06, 2006, Radiation and Nuclear Safety Authority, Helsinki (2006).
[65] RADIATION AND NUCLEAR SAFETY AUTHORITY, Insufficient Guidance of Subcontractors' Work in Olkiluoto 3 Nuclear Power Plant Project, Press Release (2006), http://www.stuk.fi/stuk/tiedotteet/2006/en_GB/news_419/
[66] RADIATION AND NUCLEAR SAFETY AUTHORITY, The Containment Steel Liner Weld of Olkiluoto 3 Will Be Repaired, Press Release (2007), http://www.stuk.fi/stuk/tiedotteet/2007/en_GB/news_468/
[67] EHLEN, M.A., et al., Life-365 Service Life Prediction Model Version 2.0, Concr. Int. American Concrete Institute, Farmington Hills, MI (2009).
[68] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Practice for Measuring Life Cycle Costs of Buildings and Building Systems, ASTM E917-05, ASTM, West Conshohocken, PA (2010).
[69] ACI COMMITTEE 365, Service-Life Prediction, ACI365.1R-00, American Concrete Institute, Farmington Hills, MI (2000).
[70] INTERNATIONAL ATOMIC ENERGY AGENCY, Safe Long Term Operation of Nuclear Power Plants, Safety Reports Series No. 57, IAEA, Vienna (2008).
[71] NUCLEAR REGULATORY COMMISSION, Standard Review Plan for Review of License Renewal Applications for Nuclear Power Plants, Rep. NUREG 1800 Rev. 2, NRC, Washington, DC (2010).
[72] NUCLEAR REGULATORY COMMISSION, Determining Prestressing Forces for Inspection of Prestressed Concrete Containments, USNRC Regulatory Guide 1.35.1, NRC, Washington, DC (1990).
[73] NUCLEAR REGULATORY COMMISSION, Degradation of Prestressing Tendon Systems in Prestressed Concrete Containments, NRC Information Notice No. 99-10, NRC, Washington, DC (1999).
[74] INTERNATIONAL ATOMIC ENERGY AGENCY, TLAA301: Concrete Containment Tendon Pre-stress (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20TLAAs/TLAA301_Concrete_Containment_Tendon_Prestress_final_160413.doc
[75] INTERNATIONAL ATOMIC ENERGY AGENCY, TLAA303: Cumulative Fatigue Damage of Containment Liners and Penetrations (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20TLAAs/TLAA303_Containment_Penetration_final_170413.doc
[76] INTERNATIONAL ATOMIC ENERGY AGENCY, TLAA302: CANDU PHWR Concrete Creep and Shrinkage (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20TLAAs/TLAA302_CANDU_PHWR_Concrete_Creep_and_Shrinkage_final_170413.doc
[77] INTERNATIONAL ATOMIC ENERGY AGENCY, TLAA304: Foundation Settlement due to Soil Movement (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20TLAAs/TLAA304_Ground_Movement_final_260713.doc
[78] NUCLEAR REGULATORY COMMISSION, Safety Evaluation Report Related to the License Renewal of the Donald C. Cook Nuclear Plant, Units 1 and 2, Rep. NUREG-1831, NRC, Washington, DC (2005).
[79] WON, S., LEE, J., Lessons Learned from the Aging Management and Lifetime Evaluation for Continued Operation in Korea, Poster Ref. No. 13, Third Int. Conf. on Nuclear Power Plant Life Manag., Salt Lake City, UT (2012).
[80] NAUS, D., et al., Report on Aging of Nuclear Power Plant Reinforced Concrete Structures, Rep. NUREG/CR-6424 (ORNL/TM-13148), Lockheed Martin Energy Research Corp., Oak Ridge Natl Lab., TN (1996).
[81] NAUS, D., Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants, Rep. NUREG/CR-4652 (ORNL/TM-10059), Martin Marietta Energy Systems, Inc., Oak Ridge Natl Lab., TN (1986).
[82] ELECTRIC POWER RESEARCH INSTITUTE, PWR Containment Structures License Renewal Industry Report: Revision 1, EPRI TR-103835 (NUMARC 90-01), EPRI, Palo Alto, CA (1994).
[83] HOOKHAM, C., SHAH, V., Insights for Aging Management of Major Light Water Reactor Components, Vol. 2, Reinforced and Prestressed Concrete Containments, Rep. NUREG/CR-5314 (EGG-2562), Idaho National Engineering and Environmental Laboratory, Idaho Falls, ID (1994).
[84] KROPP, J., Relations Between Transport Characteristics and Durability, Performance Criteria for Concrete Durability, RILEM Report No.12, RILEM Publications SARL, Bagneux (1995).
[85] BUENFIELD, N., "Measuring and modelling transport phenomena in concrete for life prediction of structures", Prediction of Concrete Durability (GLANVILLE, J., NEVILLE, A., (Eds)), E & FN Spon Publishers, London (1997).

[86] COMITE EURO-INTERNATIONAL DU BETON, Durable Concrete Structures — Design Guide, Thomas Telford Service Publisher, London (1992).
[87] THE CONCRETE SOCIETY, Diagnosis of Deterioration in Concrete Structures: Identification of Defects, Evaluation and Development of Remedial Action, Technical Report No. 54, Century House, Berkshire (2000).
[88] LITZNER, H.U., BAKER, A., Design of Concrete Structures for Durability and Strength to Eurocode 2, Materials and Structures, Vol. 32, RILEM Publications SARL, Bagneux (1999).
[89] THE CONCRETE SOCIETY, Non-Structural Cracks in Concrete, Technical Report No. 22, The Concrete Society, London (1982).
[90] BUCEA, L., KHATRI, R., SIRIVIVATNANON, V., Chemical and Physical Attack of Salts on Concrete, Urban Salt 2005 Conference, Western Sydney Regional Organisation of Councils, Parramatta (2005).
[91] HARRISON, T.A., DEWAR, J.D., BROWN, B.V., Freeze-Thaw Resisting Concrete: Its Achievement in the UK CIRIA C559, CIRIA, London (2001).
[92] MINDESS, S., YOUNG, J., Concrete, Prentice-Hall, Inc., Englewood Cliffs, NJ (1981).
[93] NEVILLE, A.M., Properties of Concrete, 4th edn, John Wiley and Sons, Inc., New York (1997).
[94] ACI COMMITTEE 201, Guide to durable concrete, J. Am. Concr. Inst. 74 12 (1977) 573-609.
[95] CANADIAN STANDARDS ASSOCIATION, Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete, CSA A23.1-09/A23.2-09, CSA, Toronto (2009).
[96] ACI COMMITTEE 318, Building Code Requirements for Structural Concrete and Commentary, ACI 318-11, American Concrete Institute, Farmington Hills, MI (2011).
[97] McDONALD, J.E., An Evaluation of Materials for Repair of Erosion Damage in Hydraulic Structures, High-Performance Materials and Systems Research Program, US Army Corps of Engineers, Vicksburg, MI (1980).
[98] BUREAU OF RECLAMATION, Guide to Concrete Repair, US Department of the Interior, Bureau of Reclamation, Denver, CO (1996).
[99] KHOURY, G. A., Performance of Heated Concrete — Mechanical Properties, Contract NUC/56/3604A with Nuclear Installations Inspectorate, August 1996, Imperial College, London (1996).
[100] BLUNDELL, R., DIAMOND, C., BROWNE, R.G., The Properties of Concrete Subjected to Elevated Temperature, CIRIA Underwater Engineering Group, Technical Note No. 9, The Concrete Society, London (1976).
[101] BRITISH STANDARDS INSTITUTION, Specification of PCPV for Nuclear Reactors, BS4975, BSI, London (1990).
[102] CANADIAN STANDARDS ASSOCIATION, N287.3-14 Design Requirements for Concrete Containment Structures for Nuclear Power Plants, CAN/CSA, Toronto (2014).
[103] NAUS, D.J., The Effect of Elevated Temperature on Concrete Materials and Structures: A Literature Review, Rep. NUREG/CR-6900, NRC, Washington, DC (2006).
[104] THE CONCRETE SOCIETY, Assessment and Repair of Fire-Damaged Concrete Structures, Technical Report No. 33, The Concrete Society, London (1990).
[105] INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION, Reliability of Fire-damaged Structures, Report No. 111, CIB, Rotterdam (1989).
[106] UDDIN, T., CULVER, C., "Effects of elevated temperature on structural materials", J. Str. Div. 101 (1975) 7.
[107] BERTERO, V., POLFVKA, M., "Influence of Thermal Exposure on Mechanical Characteristics of Concrete", Concrete for Nuclear Reactors, American Concrete Institute, Detroit, MI (1972) 505–531
[108] HILSDORF, H., KROPP, J., KOCH, H.J., "The effects of nuclear radiation on the mechanical properties of concrete", ACI SP-55, Douglas McHenry International Symposium on Concrete and Concrete Structures, American Concrete Institute, Farmington Hills, MI (1978).
[109] KAPLAN, M., Concrete Radiation Shielding, Concrete Design & Construction Series, Longman Scientific & Technical, Essex (1989).
[110] AMERICAN NUCLEAR SOCIETY, Guidelines on the Nuclear Analysis and Design of Concrete Radiation Shielding for Nuclear Power Plants, American National Standard, ANSI/ANS-6.4-1985, ANSI/ANS, La Grange Park, IL (1985).
[111] ICHIKAWA, T., KOIZUMI, H., Possibility of radiation-induced degradation of concrete by alkali-silica reaction of aggregates, J. Nucl. Sci. Technol. 39 (2002) 8, 880–884.
[112] MARUYAMA, I., KONTANI, O., ISHIZAWA, B., TAKIZAWA, M., SATO, O., "Development of system for evaluating concrete strength deterioration due to radiation and resultant heat", IAEA-CN-194-096, paper presented at IAEA 3rd Int. Conf. on NPP Life Manag. for Long Term Operations, Vienna (2012).
[113] VODÁK, F., TRTÍK, K., SOPKO, V., KAPICKOVÁ, O., DEMO, P., Effect of γ-irradiation on strength of concrete for nuclear safety-related structures, Cement and Concrete Research, 35 (2005) 1447–1551.
[114] SOO, P., MILIAN, L.M., The Effect of Gamma Radiation on the Strength of Portland Cement Mortar, J. Mater. Sci. Letters 20 (2001) 1345–1348.
[115] ELECTRIC POWER RESEARCH INSTITUTE, Effects of Radiation on Concrete: A Literature Survey and Path Forward, TR-1025584, EPRI, Palo Alto, CA (2012).

[116] NAUS, D., "Activities at ORNL in Support of Continuing the Service of Nuclear Power Plant Concrete Structures" (Proc. 3rd Int. Conf. on NPP Life Manag. for Long Term Operations, Salt Lake City, UT, 2012), IAEA-CN-194-094, International Atomic Energy Agency, Vienna (2012).
[117] ACI COMMITTEE 215, Considerations for Design of Concrete Structures Subjected to Fatigue, ACI 215R-74, American Concrete Institute, Detroit, MI (1974).
[118] AMERICAN CONCRETE INSTITUTE, Fatigue of Concrete Structures, SP-75 (Shah, S.P. Ed), ACI, Farmington Hills, MI (1982).
[119] CURL, J., A Dictionary of Architecture and Landscape Architecture, 2nd edn, Oxford University Press, Oxford (2006).
[120] MEHTA, P., Concrete-Structure, Properties, and Materials, Prentice-Hall Inc., Englewood Cliffs, NJ (1986).
[121] NEVILLE, A.M., Efflorescence: Surface Blemish or Internal Problem? Part 1: The Knowledge, Concr. Int., 24 10 (2002) 85–88.
[122] PORTLAND CEMENT ASSOCIATION, Concrete Slab Surface Defects – Causes, Prevention, Repair, PCA R&D Serial No. 2155, PCA, Skokie, IL (1997).
[123] INDIANA DEPARTMENT OF NATURAL RESOURCES, Division of Water, Dam Safety: Problems in Concrete Materials, Fact Sheet 03-06, Indiana Department of Natural Resources, Indianapolis (2003).
[124] HALVORSEN, U., Corrosion of Steel and Leaching of Lime Near Cracks in Concrete Structures, Bulletin 1, Division of Building Technology, Lund Institute of Technology, Lund, Sweden (1966).
[125] TRINH, C., BUCEA, L., FERGUSON, O., "Sulphate Resistance of Cementitious Materials: Mechanisms, Deterioration Processes, Testing and the Influence of Binder," (Proc. Concrete 97, 1997) Concrete Institute of Australia, Adelaide (1997).
[126] SWENSON, E.G., Concrete in Sulphate Environments, CBD-136, Canadian Build. Digest, National Research Council, Ottawa, Canada (1999).
[127] THAUMASITE EXPERT GROUP, Thaumasite Expert Group One-Year Review, Concr., 34 6 (2000) 51–53.
[128] BUILDING RESEARCH ESTABLISHMENT, 1st Int. Conf. on Thaumasite in Cementitious Materials Conference Proceedings 19–21 June 2002 (2002), http://projects.bre.co.uk/thaumasite/main.htm
[129] WHD MICROANALYSIS CONSULTANTS Ltd, Sulfate attack in concrete and mortar (2005), http://www.understanding-cement.com/sulfate.html
[130] MIELENZ, R.C., MARUSIN, S.L., HIME, W.G., JUGOVIC, Z.T., Investigation of Prestressed Concrete Railway Tie Distress, Concr. Int., 17 12 (1995).
[131] US DEPARTMENT OF TRANSPORT, Federal Highway Administration, Explore Delayed Ettringite Formation in Concrete (2004), http://www.fhwa.dot.gov/publications/transporter/04jul/index.cfm
[132] PORTLAND CEMENT ASSOCIATION, Types and Causes of Concrete Deterioration, Concrete Information IS536, PCA, Skokie, IL (2002).
[133] ACI COMMITTEE 515, A Guide for the Use of Waterproofing Dampproofing, Protective, and Decorative Systems for Concrete, ACI Manual of Concrete Practice-Masonry, Precast Concrete, Special Processes, American Concrete Institute, Detroit, MI (1990).
[134] DEUTSCHE INDUSTRIE NORMEN, Rating the Effect of Aggressive Liquid, Soil, and Gases Upon Concrete (Beurteilung Beton Angreiffender Wasser, Boden, und Gase), DIN 4030, DIN, Berlin (1969).
[135] FARNY, A., Kosmatka, S.H., Diagnosis and Control of Alkali-Aggregate Reactions in Concrete, Concrete Information Series No. IS413.01T, 24, Portland Cement Association, Skokie, IL (1997).
[136] WHD MICROANALYSIS CONSULTANTS LTD, Alkali-silica reaction in concrete (2012), http://www.understanding-cement.com/alkali-silica.html
[137] OZOL, M.A., "Alkali-Carbonate Rock Reactions", Significance of Tests and Properties of Concrete and Concrete-Making Materials STP 169C (Klieger, P., Lamond, J.F., Eds) American Society of Testing and Materials, West Conshohocken, PA (1994) 341–364.
[138] LEMING, M.L., Alkali–silica reactivity: Mechanisms and management, Min. Eng 48 12 (1996) 61–64.
[139] CTLGROUP, Alkali-Silica Reactivity (ASR) Related Services (2012), http://www.ctlgroup.com/OurExpertise/ConsultingServices/Detail/55
[140] KAWAMURA, M., TAKEMOTO, K., ICHISE, M., "Influences of alkali-silica reaction on steel reinforcement in concrete", Proc. of 8th Int. Conf. on Alkali- Aggregate React., Kyoto (1989).
[141] MURAZUMI, Y., et al., Study on the influence of Alkali-Silica reaction on structural behavior of reinforced concrete members (Proc. 18th Int. Conf. Struct. Mech. React. Technol., 2005), Beijing (2005).
[142] TAKAKURA, T., et al., "Structural soundness for turbine-generator foundation affected by alkali-silica reaction and its maintenance plans", Paper No. H03/2, (Proc. 19th Int. Conf. Structural Mechanics React. Technol. (SMiRT)), International Association for Struct. Mech. in React. Technol., Toronto (2007).
[143] NEVILLE, A.M., Neville on Concrete: An Examination of Issues in Concrete Practice, American Concrete Institute, Farmington Hills, MI (2003).
[144] HOBBS, D., Concrete deterioration: Causes, diagnosis, and minimising risk, Int. Mater. Rev., 46 3 (2001) 117–144.
[145] THE CONSTRUCTOR, Concrete in seawater (2012), http://theconstructor.org/concrete/concrete-in-seawater/843/

[146] SUPRENANT, B.A., Designing Concrete for Exposure to Seawater, Low Permeability is Critical, Publication No. C910873, The Aberdeen Group, Boston, MA (1991).
[147] SANCHEZ-SILVA, M., ROSOWSKY, D., Biodeterioration of construction materials: State of the art and future challenges, J. Mater. Civil Eng , 20 (2008) 352–365.
[148] ROGERS, R., Assessment of the Effects of Microbially Influenced Degradation on a Massive Concrete Structure, Final Report to Introduced Materials Task, Yucca Mountain Site Characterization Project, Lawrence Livermore National Laboratory, U.S. Department of Energy, Livermore, CA (1995).
[149] HORNE, J.M., MEIKE, A., Microbial Activity at Yucca Mountain, UCRL-ID-122256, Lawrence Livermore National Laboratory, Livermore, CA (2005).
[150] MORTON, G., Things that go rot in the night: A review of biodegradation, Microbiol. Today 30 21 (2003).
[151] AVIAM, O., BAR-NES, G., ZEIRI, Y., SIVAN, A., Accelerated Biodegradation of Cement by Sulfur- Oxidizing Bacteria as a Bioassay for Evaluating Immobilization of Low-Level Radioactive Waste, Appl. Environ. Microbiol. 70 10 (2004) 6031–6036.
[152] DIERCKS, M., SAND, W., BOCK, E., Microbial corrosion of concrete, Experientia 47 (1991) 514–516.
[153] SAND, W., BOCK, E., Biodeterioration of Ceramic Materials by Biogenic Acid, International Biodeterioration and Biodegeneration (1991).
[154] HOFMANN, F.-J., HORMANN, K., SCHMIDT, M., WAGNER, E., Concrete with Greater Resistance to Acid and to Biogenic Sulfuric Acid Corrosion, Betonwerk+Fertigteil – Technik 4 (1997).
[155] SWIAT, W., et al., State-of-the-Art Report — Corrosion of Steel in Concrete, ORNL/NRC/LTR-93/2, Martin Marietta Energy Systems, Inc., Oak Ridge Natl Lab., TN (1993).
[156] ASM INTERNATIONAL, Metals Handbook, 9th edn, Vol. 13, ASM, Metals Park, OH (1987).
[157] DUNN, D., PULVIRENTI, A., HISER, M.A., Containment Liner Operating Experience Summary, Technical Letter Report, Rev. 1, NRC, Washington, DC (2011).
[158] FIRST ENERGY, BV1 Containment Liner Plate Hole, CR 09-57762, FirstEnergy Corporation, Akron, OH (2009).
[159] MONTEIRO, P., Structure and Properties of Engineering Materials, Course Material for CE60, Department of Civil and Structural Engineering, University of California, Berkeley, CA (2005).
[160] MOHAMMED, T., Durability Related Issues of Concrete Structures Under Marine Exposure, Keikyu Kurihama (2004).
[161] CORROSION-CLUB.COM, Reinforcing Steel Corrosion Gallery (2012), http://www.corrosion-club.com/rebarimages.htm
[162] ACI COMMITTEE 222, Protection of Metals in Concrete Against Corrosion, ACI 222R-01 (Reapproved 2010), American Concrete Institute, Farmington Hills, MI (2001).
[163] ERLIN, B., VERBECK, G.J., Corrosion of Metals in Concrete – Needed Research, Corrosion of Metals in Concrete, SP-49, American Concrete Institute, Farmington Hills, MI (1975).
[164] GONZÁLEZ, J.A., ALONSO, C., ANDRADE, C., “Corrosion rate of reinforcements during accelerated carbonation of mortar made from different types of cement”, Proc. Mtg on Corrosion of Reinforcement in Concrete Construction, London, 1983, Ellis Horwood, New York (1983) 159–174.
[165] ROBERTS, M.H., Carbonation of Concrete Made with Dense Natural Aggregates, Information Paper 6/81, Building Research Establishment, Glasgow (1981).
[166] SIMS, I., The assessment of concrete for carbonation, Concr. 28 (1994) 33–38.
[167] CLIFTON, J., Predicting the Remaining Service Life of Concrete, NISTIR 4712, US Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD (1991).
[168] RILEM RECOMMENDATION CPC-18, Measurement of hardened concrete carbonation depth, Mater. Struct. 24 126 (1988) 453–455.
[169] MEHTA, P.K., GERWICK, J.B.C., Cracking-corrosion interaction in concrete exposed to marine environment, Concr. Int. 4 10 (1982) 45–51.
[170] BRITISH STANDARDS INSTITUTION, Structural Use of Concrete – Code of Practice for Design and Construction, BS 8110, BSI, London (1985).
[171] BRITISH STANDARDS INSTITUTION, Concrete Part 1: Specification, Performance, Production and Conformity, BS ENV 206–1, BSI, London (2000).
[172] HUSSAIN, S.E., AL-GANTANI, A.S., RASHEEDUZZAFAR, Chloride threshold for corrosion of reinforcement in concrete, ACI Mater. J. 93 6 (1996) 535–548.
[173] BUILDING RESEARCH STATION, The durability of steel in concrete: Part 2 – Diagnosis and assessment of corrosion-cracked concrete, Build. Res. Establishment Dig. 26 (1982).
[174] RASHEEDUZZAFAR, AL-SAADOUN, S.S., AL-GAHTANI, A.S., Corrosion cracking in relation to bar diameter, cover, and concrete quality, J. Mater. Civ. Eng 4 4 (1992) 327–341.
[175] SCHNEIDER, U., DIERERICHS, C., EHM, C., Effect of temperature on steel and concrete for PCRV’s, Nucl. Eng Des. 67 (1981) 245–258.
[176] El-NESR, O., “Diagnosis and repair of buildings damaged by fire exposure, extending the lifespan of structures”, a paper presented at IABSE Symp. On Extending the Lifespan of Structures, San Francisco, 1995, Int. Assoc. for Bridge and Structural Eng, Zurich (1995).

[177] SMITH, P., Resistance to High Temperatures, Significance of Tests and Properties of Concrete and Concrete-Making Materials, STP 169B, American Society for Testing and Materials, West Conshohocken, PA (1978).

[178] TAKEUCHI, M., et al., "Material Properties of Concrete and Steel Bars at Elevated Temperatures", Paper H04/4, paper presented at 12th Int. Conf. on Structural Mechanics in Reactor Technology, North-Holland (1993).

[179] DOTREPPE, J.-C., Mechanical Properties of Quenched and Self-Tempered Reinforcing Steel at Elevated Temperatures Compared with Recommendations on Eurocode 1 – Part 1-2, Mater. Struct. 30 (1997) 430–438.

[180] EUROPEAN COMMITTEE FOR STANDARDIZATION, Eurocode 2: Design of Concrete Structures, Part 1-2: Structural Fire Design, ENV 1992- 1-2, CEN, Brussels (1995).

[181] COWEN, A., NICHOLS, R., Effect of Irradiation on Steels Used in Pressure Vessels, Prestressed Concrete Pressure Vessels, Group D, Paper No. 20, a paper presented at Conf. on Prestress. Conc. Press. Vessels, London (1968) 229–235.

[182] SCIENCE APPLICATIONS INC., Study of Radiation Dosage to Structural Components in Nuclear Reactors, EPRI NP-152, Electric Power Research Institute, Palo Alto, CA (1977).

[183] NÜRNBERGER, U., Corrosion induced failures of prestressing steel, Otto-Graf J. Res. Test. 13 (2002).

[184] HARTT, W.H., "A critical evaluation of cathodic protection for prestressing steel in concrete", Corrosion of Reinforcement in Concrete Construction (Proc. Third Int. Symp. Wishaw, Warwickshire, 1990).

[185] WAGNER, J., YOUNG, W., SCHEIRER, S., FAIRER, P., Cathodic Protection Developments for Prestressed Concrete Components – Interim Report, Report No. FHWA-RD-92-056, Federal Highway Administration, Washington, DC (1993).

[186] STRATFULL, R.F., Criteria for the Cathodic Protection of Bridge Decks, Corrosion of Reinforcement in Concrete Construction, Society of Chemical Industry, London (1983).

[187] PODOLNY, W.J., MELVILLE, T., Understanding the Relaxation in Prestressing, Prestressed Concr. Inst. J. 14 4 (1969).

[188] ABRAMS, M., CRUZ, C., The Behavior at High Temperature of Steel Strand for Prestressed Concrete, Research Department Bulletin No. 134, Portland Cement Association, Skokie, IL (1961).

[189] HARMATHY, T.W., Elevated Temperature Tensile and Creep Properties for Some Structural and Prestressing Steels, STP 464, American Society for Testing and Materials, West Conshohocken, PA (1970).

[190] ACI COMMITTEE 216, Guide for Determining the Fire Endurance of Concrete Elements, ACI 216R-81, ACI Manual of Concrete Practice, Part 3, American Concrete Institute, Detroit, MI (1990).

[191] ANDERSON, P., Thirty Years of Measured Prestress at Swedish Nuclear Reactor Containments, Nucl. Eng. Des. 235 (2005).

[192] CAHILL, T., BRANCH, G.D., "Long-Term Relaxation Behavior of Stabilized Prestressing Wires and Strands", Group D, paper 19, paper presented at Conf. on Prestress. Concr. Press. Vessels (UDELL, M. Ed.), The Institute of Civil Engineers, London (1968) 219–228.

[193] COLLINS, M., MITCHELL, D., Prestressed Concrete Structures, Response Publications, Toronto (1997).

[194] LUNDQVIST, P., Nuclear Reactor Containments: Evaluation of Prestress Losses and Prediction Models, 3rd FIB Int. Congress Inc. the PCI Annual Conv. and Bridge Conf., Precast Prestressed Concrete Institute, Washington, DC (2010).

[195] ANDERSON, P., Structural Integrity of Prestressed Nuclear Reactor Containments, PhD Thesis, Lund Univ. (2005).

[196] BATE, S., CORSON, R., "Effect of Temperature on Prestressing Wires", Group D, paper 21, paper presented at Conf. on Prestressed Concrete Pressure Vessels, London (1968).

[197] ACI COMMITTEE 209, ACI 209.1R Report on Factors Affecting Shrink and Creep of Hardened Concrete, American Concrete Institute, Detroit, MI (2005).

[198] LIBBY, J., Modern Prestressed Concrete-Design Principles and Construction Methods, Van Nostrand Reinhold, New York (1971).

[199] AMERICAN CONCRETE INSTITUTE/AMERICAN SOCIETY OF MECHANICAL ENGINEERS, Code for Concrete Reactor Vessels and Containments, Section III, Division 2 of the ASME Boiler and Pressure Vessel Code (ACI Standard 359-92), ACI/ASME, New York (1995).

[200] PODOLNY, W.J., Understanding the Losses in Prestressing, J. Prestress. Concr. Inst. 14 5 (1969).

[201] HANSON, J. A., Prestress Loss as Affected by Type of Curing, Bulletin D75, Portland Cement Association, Skokie, IL (1964).

[202] PCI COMMITTEE ON PRESTRESS LOSSES, Recommendations for Estimating Prestress Losses, Prestress. Concr. Inst. J., 20 5 (1975).

[203] OECD NUCLEAR ENERGY AGENCY, NPP Containment Prestress Loss Summary Statement, NEA/CSNI/R(99)11 OECD, Nuclear Energy Agency, Paris (1999).

[204] PATEL, S.C., GUPCHUP, V., BASU, P., SHYLAMONI, P., "Impact of loss of prestressing force on NPP containment structure" (Proc. 19th Int. Conf. Structural Mechanics React. Technol. (SMiRT)), International Association for Struct. Mech. in React. Technol., Toronto (2007).

[205] GRANGER, L., TORRENTI, J.-M., Lifespan Evaluation of a Nuclear Vessel in Terms of Prestress Loss, Extending the Lifespan of Structures (Int. Symp.) 73 (2), Int. Assoc. for Bridge and Struct. Eng, 1411-16, Zurich (1995).

[206] COMBES, J.-P., et al., "Presentation of works related to the french PWR nuclear power plant lifetime" (Proc. of ASME Pressure Vessel and Piping Conference, San Diego, California, 1991) New York (1991).

[207] BOLVIN, M., CHAUVEL, D., Lifetime Cooling Towers (Proc. ASME Pressure Vessel and Piping Conference, New Orleans, 1992) New York (1992).

[208] ELECTRIC POWER RESEARCH INSTITUTE, Assessment and Repair of Prestressed Concrete Cylinder Pipe (PCCP) (Joint EPRI-EDF Workshop on Inspection and Degradation Management of Concrete Structures in the Nuclear Industry, Materials Ageing Institute, Moret sur Loing 2011) EPRI, Palo Alto, CA (2011).
[209] AMERICAN SOCIETY OF CIVIL ENGINEERS, Guidelines for Structural Condition Assessment of Existing Buildings, ASCE Standard SEI/ASCE 11-99, ASCE, Reston, VA (2000).
[210] ROMANOFF, M., Corrosion of Steel Pilings in Soils, Monograph 58, National Bureau of Standards, Gaithersburg, MD (1962).
[211] PECK, R., HANSON, W., THORNBURN, T., Foundation Engineering, Wiley and Sons, New York (1974).
[212] DONSEZ, D., FAUDEMAY, P., HOMOND, P., A Cooperative Database System Based on Workspace Hierarchy, Industrial Information and Design Issues, Springer, Berlin (1966).
[213] SENI, C., IANKO, L., An International Database on Nuclear Concrete Structures Ageing, Industrial Information and Design Issues, Springer, Berlin (1966).
[214] INTERNATIONAL ATOMIC ENERGY AGENCY, Summary Results of the Survey on Concrete Containment Ageing, Internal IAEA Report, IAEA, Vienna (1995).
[215] GREGOR, F., HOOKHAM, C., Remnant Life Preservation of LWR Plant Structures (Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology held August 15-20, 1993, in Stuttgart, Germany) Paper DH06/2 (1993) 453–459.
[216] ASHAR, H., JENG, D., "Effectiveness of in-service inspection requirements of prestressed concrete containments: US experience" (Proc. Second Int. Conf. on Containment Design and Operation), Canadian Nuclear Society, Toronto (1990).
[217] MacLACHLAN, A., Containment Liners Found Corroded at Some EDF 900-MW Reactors McGraw-Hill, Nucleonics Week 33 19, Platts/McGraw Hill Financial, New York (1992) 5–6.
[218] POCOCK, D., et al., Long-Term Performance of Structures Comprising Nuclear Power Plants, EUR 12758 EN, Commission of European Communities, Luxembourg (1990).
[219] ZAKAIB, G., "Pickering Unit #1 containment leakage characterization" (Proc. Third Int. Conf. on Containment Design and Operation), Canadian Nuclear Society, Toronto (1994).
[220] DAWSON, P., WILSON, M., Surveillance Data for PCPVs at Wylfa, Hartlepool and Heysham 1 Power Stations, Report No. 1302/92/5957, Taywood Engineering Ltd., R&D Division, London (1992).
[221] COPINGER, D., OLAND, C., NAUS, D., A Summary of Aging Effects and Their Management in Reactor Spent Fuel Pools, Refueling Cavities, Tori, and Safety Related Concrete Structures, Rep. NUREG/CR-7111, NRC, Washington, DC (2012).
[222] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 301 In-service inspection for Containment Steel Elements (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP301_ISI%20for%20Containment%20Steel%20Elements_final_130813.doc
[223] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 302 In-service inspection for Concrete Containment (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP302_ISI_for_Concrete_Containment_final_130813.doc
[224] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 303 Safety Class 1, 2 and 3 Piping and Metal Containment Components Supports (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP303_Safety_Class%201_2_3_Piping_and_Metal_Containment_Components_Supports_final_130813.doc
[225] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 304 Containment Leak Rate Testing (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP304_Containment_Leak_Rate_Testing_final_130813.doc
[226] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 305 Masonry Walls (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP305_Masonry_Walls_final_130813.doc
[227] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 306 Structures Monitoring (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP306_Structures_Monitoring%20_final_130813.doc
[228] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 307 Water-control structures (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP307_Water-control_Structures_final_130813.doc
[229] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 308 Protective Coatings Monitoring and Maintenance Programme (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP308_Protective_Coating_final_130813.doc
[230] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 309 Non-metallic Liners (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP309_Non-metallic_Liner_final_130813.doc

[231] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 310 Ground Movement Surveillance (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP310_Ground_Movement_Surveillance_final_130813.doc
[232] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 311 Containment Monitoring System (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP311_Containment_Monitoring_System_final_130813.doc
[233] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 312 Concrete Expansion Detection and Monitoring System (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP311_Containment_Monitoring_System_final_130813.doc
[234] INTERNATIONAL ATOMIC ENERGY AGENCY, AMP 313 Containment Prestressing System (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/Shared%20Documents/IGALL%20DB%20folder/IGALL%20AMPs/AMP313_Containment_Pre-stressing_System_final_130813.doc
[235] KRAUSS, P., Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants, ORNL/NRC/LTR-93/28, Martin Marietta Energy Systems Inc., Oak Ridge Natl Lab., TN (1994).
[236] BRAVERMAN, J., HOFMAYER, C., MORANTE, R., SHTEYNGART, S., BEZLER, P., Assessment of Age-Related Degradation of Structures and Passive Components for U.S. Nuclear Power Plants, Rep. NUREG/CR-6679, BNL-NUREG-52587, NRC, Washington, DC (2000).
[237] DUKE ENERGY, Crystal River Nuclear Plant to be Retired: Company Evaluating Sites for Potential New Gas-fueled Generation, Press Release, Duke Energy, Crystal River, FL (2013).
[238] NUCLEAR REGULATORY COMMISSION, Assessment of In Service Conditions of Safety-Related Nuclear Power Plant Structures, Rep. NUREG-1522, NRC, Washington, DC (1995).
[239] COURTOIS, A., GENEST, Y., VACQUÉ, A., AFONSO, F., In-service Inspection of Cooling Towers (2012), http://www.neimagazine.com/storyprint.asp?sC=2062231
[240] INSTITUTE OF NUCLEAR POWER OPERATIONS, INPO Topical Report TR8-62 Cooling Tower Structure Events, INPO, Atlanta, GA (2008).
[241] ACI COMMITTEE 349, Evaluation of Existing Nuclear Safety Related Concrete Structures, ACI 349.3R-02 (Reapp. 2010), American Concrete Institute, Farmington Hills, MI (2002).
[242] ELECTRIC POWER RESEARCH INSTITUTE, Aging Identification and Assessment Checklist: Civil and Structural Components, Electric Power Research Institute, Palo Alto, CA (2005).
[243] CANADIAN STANDARDS ASSOCIATION, A864-00 (R2005): Guide to the Evaluation and Management of Concrete Structures Affected by Alkali-Aggregate Reaction, CSA, Toronto (2005).
[244] CANADIAN STANDARDS ASSOCIATION (CSA), CSA N287.1-14 General Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, Toronto, CSA (2014).
[245] CANADIAN STANDARDS ASSOCIATION, CSA N287.2-08 Material Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, Toronto (2008).
[246] CANADIAN STANDARDS ASSOCIATION, CSA N287.4-09 Construction, Fabrication, and Installation Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, Toronto (2009).
[247] CANADIAN STANDARDS ASSOCIATION, CSA-N287.5-11 Testing and Examination Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, Toronto (2011).
[248] CANADIAN STANDARDS ASSOCIATION, N287.6-11 Pre-operational Proof and Leakage Rate Testing Requirements for Concrete Containment Structures for Nuclear Power Plants, CSA, Mississauga (2011).
[249] CANADIAN STANDARDS ASSOCIATION, N287.7-08 In-service Examination and Testing Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants, CSA, Toronto (2008).
[250] AMERICAN SOCIETY OF MECHANICAL ENGINEERS, Rules for Inservice Inspection of Nuclear Power Plant Components, Section XI, ASME Boiler and Pressure Vessel Code, ASME, New York (1995).
[251] AMERICAN CONCRETE INSTITUTE, ACI 349-06 Code Requirements for Nuclear Safety Related Concrete Structures, ACI, Farmington Hills, MI (2007).
[252] JAPAN SOCIETY OF MECHANICAL ENGINEERING, JSME S NE1:2011, Codes for Nuclear Power Generation Facilities - Rules on Concrete Containment Vessels for Nuclear Power Plants, JSME, Tokyo (2011).
[253] EUROPEAN UTILITIES REQUIREMENTS ORGANIZATION, European Utility Requirements for LWR Nuclear Power Plants, Vol. 2, Revision C, Villeurbanne (2001).
[254] AREVA NP and EDF, Pre-construction Safety Report UKEPR-0002-035 Issue 03: PCSR: Sub-chapter 3.3: Design of Safety Classified Civil Structures, Paris La Défense Cedex: AREVA NP & EDF (2011).
[255] AFCEN, ETC-C EPR Technical Code for Civil Works, AFCEN, La Défense (2010).
[256] ELECTRICITÉ DE FRANCE, AFCEN, RCC-G : Design and Construction Rules for Civil Works of PWR Nuclear Islands, AFCEN No. 50-1989, EDF, AFCEN, France (1989).
[257] INTERNATIONAL ATOMIC ENERGY AGENCY, Guidebook on Non-destructive Testing of Concrete Structures, IAEA-TCS-17, IAEA, Vienna (2002).

[258] AMERICAN SOCIETY OF MECHANICAL ENGINEERS, Rules for Construction of Nuclear Facility Components, ASME BPVC Section III, ASME Boiler and Pressure Vessel Code, ASME, New York (2013).
[259] NUCLEAR REGULATORY COMMISSION, In-service Inspection of Prestressed Concrete Containment Structures with Grouted Tendons, RG 1.90 Rev 2, NRC, Washington, DC (2012).
[260] CANADIAN NUCLEAR SAFETY COMMISSION, Fitness for Service: Aging Management, CNSC REGDOC-2.6.3, CNSC, Ottawa (2014).
[261] CANADIAN NUCLEAR SAFETY COMMISSION, RD-334: Aging Management for Nuclear Power Plants, CNSC, Ottawa (2011).
[262] CANADIAN STANDARDS ASSOCIATION, Requirements for the Containment System of Nuclear Power Plants, CSA N290.3, CSA, Toronto, Canada (in preparation).
[263] CANADIAN STANDARDS ASSOCIATION, N291-08 (R2013): Requirements for Safety-Related Structures for CANDU Nuclear Power Plants, CSA, Toronto (2008).
[264] CANADIAN STANDARDS ASSOCIATION, CAN/CSA-A23.3-04 (R2010) Design of Concrete Structures, CSA, Toronto (2004).
[265] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1992-1-1 Eurocode 2: Design of Concrete Structures – Part 1-1: General Rules for Buildings, CEN, Brussels (2004).
[266] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-1:2005 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 1: Definitions, CEN, Brussels (2005).
[267] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-2:2004 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 2: Surface Protection Systems for Concrete, CEN, Brussels (2004).
[268] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-3:2005 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 3: Structural and Non-structural Repair, CEN, Brussels (2005).
[269] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-4:2004 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 4: Structural Bonding, CEN, Brussels (2004).
[270] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-5:2004 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 5: Concrete Injection, CEN, Brussels (2004).
[271] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-6:2006 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 6: Anchoring of Reinforcing Steel Bar, CEN, Brussels (2006).
[272] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-7:2006 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 7: Reinforcement Corrosion Protection, CEN, Brussels (2007).
[273] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-8:2004 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 8: Quality Control and Evaluation of Conformity, CEN, Brussels (2004).
[274] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-9:2008 Products and Systems for the Protection and Repair of Concrete Structures: Definitions, Requirements, Quality Control and Evaluation of Conformity Part 9: General Principles for the use of Products and Systems, CEN, Brussels (2008).
[275] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN 1504-10:2003 Products and Systems for the Protection and Repair of Concrete Structures: Definitions - Requirements - Quality Control and Evaluation of Conformity Part 10: Site Application of Products and Systems and Quality Control of the Works, CEN, Brussels (2003).
[276] EUROPEAN COMMITTEE FOR STANDARDIZATION, EN-206-1 Concrete Part 1: Specification, Performance, Production and Conformity, CEN, Brussels (2000).
[277] RADIATION AND NUCLEAR SAFETY AUTHORITY, YVL 1.8 Repairs, Modifications and Preventive Maintenance at Nuclear Facilities, STUK, Helsinki (1986).
[278] RADIATION AND NUCLEAR SAFETY AUTHORITY, YVL A.8, Ageing Management of a Nuclear Facility, STUK, Helsinki (2013).
[279] RADIATION AND NUCLEAR SAFETY AUTHORITY, YVL B.6 Containment of a Nuclear Power Plant, STUK, Helsinki (2013).
[280] FINNISH STANDARDS ASSOCIATION, SFS-EN 13670: Execution of Concrete Structures, Finnish Standards Association, Helsinki (2010).
[281] AMERICAN SOCIETY OF MECHANICAL ENGINEERS, Rules for Construction of Nuclear Facility Components: Division 2 Code for Concrete Containments, BPVC Section III Division 2, ASME, New York (2013).

[282] INTERNATIONAL STANDARDS ORGANIZATION, ISO 16311-1 Maintenance and Repair of Concrete Structures — Part 1: General Principles, ISO, Geneva (2014).
[283] INTERNATIONAL STANDARDS ORGANIZATION, ISO 16311-2:2014 Maintenance and Repair of Concrete Structures — Part 2: Assessment of Existing concrete structures, ISO, Geneva (2014).
[284] INTERNATIONAL STANDARDS ORGANIZATION, ISO 16311-3:2014 Maintenance and Repair of Concrete Structures — Part 3: Design of Repairs and Prevention, ISO, Geneva (2014).
[285] INTERNATIONAL STANDARDS ORGANIZATION, ISO 16311-4:2014 Maintenance and Repair of Concrete Structures — Part 4: Execution of Repairs and Prevention, ISO, Geneva (2014).
[286] INTERNATIONAL STANDARDS ORGANIZATION (ISO), ISO 12696:2012 Cathodic protection of steel in concrete, ISO, Geneva (2012).
[287] JAPAN SOCIETY OF CIVIL ENGINEERS, JSCE Guidelines for Concrete No. 15 — Standard Specifications for Concrete Structures — 2007 "Design", JSCE, Tokyo (2007).
[288] SWISS SOCIETY OF ENGINEERS AND ARCHITECTS, Concrete: Part 1: Specification, Performances, Production and Conformity, SN EN 206-1:2000, SIA, Zurich (2000).
[289] SWISS SOCIETY OF ENGINEERS AND ARCHITECTS, Concrete Structures, SIA 162, SIA, Zurich (1993).
[290] HEALTH AND SAFETY EXECUTIVE, The Tolerability of Risk from Nuclear Power Stations, HSE, London (1992).
[291] HEALTH AND SAFETY EXECUTIVE, Safety Assessment Principle for Nuclear Plants, HSE, London (1992).
[292] UK OFFICE FOR NUCLEAR REGULATION, Licence Condition Handbook, United Kingdom Office for Nuclear Regulation, Bootle (2011).
[293] HEALTH AND SAFETY EXECUTIVE, The Pressure Systems and Transportable Gas Containers Regulations 1989, HSE, London (1989).
[294] BRITISH STANDARDS INSTITUTION, The Structural Use of Reinforced Concrete in Buildings, CP 114 (withdrawn), BSI, London (1969).
[295] BRITISH STANDARDS INSTITUTION, The Structural Use of Prestressed Concrete in Buildings, CP 115:1969 (withdrawn), BSI, London (1969).
[296] NUCLEAR REGULATORY COMMISSION, Domestic Licensing of Production and Utilization Facilities, 10 CFR 50, US Govt Printing Office, Washington, DC (2007).
[297] NUCLEAR REGULATORY COMMISSION, Requirements for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants, 10 CFR 50, US Govt Printing Office, Washington, DC (2007).
[298] NUCLEAR REGULATORY COMMISSION, Appendix A: General Design Criteria for Nuclear Power Plants, 10 CFR 50, US Govt Printing Office, Washington, DC (2007).
[299] NUCLEAR REGULATORY COMMISSION, Appendix J: Primary Reactor Containment Leakage Testing for Water-Cooled Reactors, 10 CFR 50, US Govt Printing Office, Washington, DC (1995) 748–753.
[300] ACI COMMITTEE 201, Guide for Making a Condition Survey of Concrete in Service, ACI 201.1R-92, American Concrete Institute, Farmington Hills, MI (1997).
[301] NUCLEAR REGULATORY COMMISSION, Qualification for Cement Grouting for Prestressing Tendons in Containment Structures, USNRC REGULATORY GUIDE 1.107 Rev. 2, NRC, Washington, DC (2011).
[302] NUCLEAR REGULATORY COMMISSION, Structural Acceptance Test for Concrete Primary Reactor Containments Rev. 1, RG 1.18, NRC, Washington, DC (1972).
[303] NUCLEAR REGULATORY COMMISSION, Inservice Inspections of Ungrouted Tendons in Prestressed Concrete Containment Structures, Regulatory Guide 1.35, Rev. 3, Office of Standards Development, NRC, Washington, DC (1990).
[304] NUCLEAR REGULATORY COMMISSION, Design Limits, Loading Combinations, Materials, Construction and Testing of Concrete Containment, RG 1.136, Rev 3, NRC, Washington, DC (2007).
[305] NUCLEAR REGULATORY COMMISSION, Inspection of Water-Control Structures Associated with Nuclear Power Plants, Regulatory Guide 1.127, Rev. 1, NRC, Washington, DC (1978).
[306] NUCLEAR REGULATORY COMMISSION, Generic Aging Lessons Learned (GALL) Report, Rep. NUREG-1801, Rev. 2, Division of Regulatory Improvement Programs, NRC, Washington, DC (2010).
[307] NUCLEAR REGULATORY COMMISSION, Light-Water Reactor Inspection Program-Operations Phase, Inspection Manual Chapter IMC 2515, NRC, Washington, DC (2013).
[308] NUCLEAR REGULATORY COMMISSION, Policy and Guidance for the License Renewal Inspection Program, Inspection Manual Chapter IMC 2516, USNRC, Washington, DC (2013).
[309] NUCLEAR REGULATORY COMMISSION, License Renewal Inspection, Inspection Procedure 71002, NRC Inspection Manual, NRC, Washington, DC (2005).
[310] NUCLEAR REGULATORY COMMISSION, Post-approval Site Inspection for License Renewal, Inspection Procedure 71003, NRC Inspection Manual, NRC, Washington, DC (2008).
[311] NUCLEAR ENERGY INSTITUTE, Revision 6 Industry Guideline for Implementing the Requirements of 10 CFR Part 54, The License Renewal Rule, NEI 95-10, NEI, Washington, DC (2005).

[312] ELECTRIC POWER RESEARCH INSTITUTE, BWR Containments License Renewal Industry Report: Revision 1 Final Report, EPRI TR-103840 (NUMARC 90-10), EPRI, Palo Alto, CA (1990).
[313] INTERNATIONAL STANDARDS ORGANIZATION, Performance and Assessment Requirements for Design Standards on Structural Concrete, ISO 19338:2007, ISO, Geneva (2007).
[314] INTERNATIONAL ATOMIC ENERGY AGENCY, Data Collection and Record Keeping for the Management of Nuclear Power Plant Ageing, Safety Series No. 50-P-3, IAEA, Vienna (1991).
[315] VESIKARI, E., Service Life Management System of Concrete Structures in Nuclear Power Plants, VTT Publications 648, VTT Technical Research Centre of Finland, Espoo (2007).
[316] INTERNATIONAL ATOMIC ENERGY AGENCY, Periodic Safety Review of Nuclear Power Plants, IAEA Safety Standards Series No. NS-G-2.10, IAEA, Vienna (2003).
[317] HOOKHAM, C., Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants, ORNL/NRC/LTR-90/17, Martin Marietta Energy Systems, Inc., Oak Ridge Natl Lab., TN (1991).
[318] HOOKHAM, C., In-service Inspection Guidelines for Concrete Structures in Nuclear Power Plants, ORNL/NRC/LTR-95/14, Lockheed Martin Energy Systems, Inc., Oak Ridge Natl Lab., TN (1995).
[319] JUDGE, R., Classification of Structural Component Degradation Mechanisms for Containment Systems (Proc. Third Int.Conf. on Containment Design and Operation) Toronto (1994).
[320] TCHERNER, J., AZIZ, T., Candu Energy experience in condition assessment of nuclear safety related concrete structures, SMiRT 22, San Francisco, CA (2013).
[321] AMERICAN SOCIETY OF MECHANICAL ENGINEERS, Requirements for Class CC Concrete Components of Light-Water Cooled Plants, Section XI, Subsection IWL in American Society of Mechanical Engineers Boiler and Pressure Vessel Code, 1992 edn, ASME, New York (1995).
[322] ELLINGWOOD, B., Aging Effects on Probabilistic Risk Assessment, Rep. NUREG/CR-6425 (ORNLATM-13149), Lockheed Martin Energy Research Corp., Oak Ridge Natl Lab., TN (1996).
[323] INTERNATIONAL ATOMIC ENERGY AGENCY, Assessing and Managing Cable Ageing in Nuclear Power Plants, IAEA Nuclear Energy Series No. NP-T-3.6, IAEA, Vienna (2012).
[324] BROWNE, R., Durability of Reinforced Concrete Structures, Parts 1 and 2, New Zealand Concrete Research Association, New Zealand (1989).
[325] GANNON, E., CADY, P., Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion: State-of-the-Art of Existing Methods, SHRP-S-323, Vol. 1, Strategic Highway Research Programme, National Research Council, Washington, DC (1993).
[326] INTERNATIONAL ATOMIC ENERGY AGENCY, Recruitment, Qualification and Training of Personnel for Nuclear Power Plants, IAEA Safety Standards Series No. NS-G-2.8, IAEA, Vienna (2002).
[327] PRICE, W.F., et al., Review of European Repair Practice for Corrosion Damaged Reinforced Concrete, Report No. 1303/91/5823, Taywood Engineering, London (1993).
[328] MALHOTRA, V., CARINO, N.J., Handbook of Nondestructive Testing of Concrete, CRC Press, Boca Raton, FL (1991).
[329] MATHEY, R., CLIFTON, J., Review of Nondestructive Evaluation Methods Applicable to Construction Materials and Structures, NBS Technical Note 1247, National Bureau of Standards, US Department of Commerce, Gaithersburg, MD (1988).
[330] MALHOTRA, V., In Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, MI (1984).
[331] AMERICAN CONCRETE INSTITUTE, Nondestructive Testing, SP-112 (LEW, H., Ed.) ACI, Farmington Hills, MI (1988).
[332] DUNNICLIFF, J., Geotechnical Instrumentation for Monitoring Field Performance, John Wiley and Sons, Toronto (1988).
[333] AMERICAN CONCRETE INSTITUTE, ACI Manual of Concrete Inspection SP-2(07), ACI, Farmington Hills, MI (2007).
[334] RAO, B., Introduction to Non-Destructive Testing (NDT), http://geocities.ws/raobpc/NDT.html
[335] ACI COMMITTEE 228, Report on Nondestructive Test Methods for Evaluation of Concrete Structures, ACI 228.2R-13, American Concrete Institute, Farmington Hills, MI (2013).
[336] RAO, B., Visual Techniques in NDT, Encyclopaedia of Materials: Science and Technology, Elsevier Science (2001) 6043–6046.
[337] LASUDRY, R., Instrumentation and Monitoring of Belgian Cooling Towers, NEA/CSNI/R(2000)15, (Workshop on Instrumentation and Monitoring of Concrete Structures) Nuclear Energy Agency, Issy-les-Molineaux (2001).
[338] DE SCHUTTE, G., Advanced Monitoring of Cracked Structures Using Video Microscope and Automated Image Analysis, NDT&E Int. 35 4 (2002) 209–212.
[339] BAM FEDERAL INSTITUTE FOR MATERIAL RESEARCH AND TESTING, Building Survey System Photograph (2012), http://www.bam.de/en/kompetenzen/fachabteilungen/abteilung_8/fg82/fachgruppe_82a.htm
[340] WOODSON, R.D., Concrete Structures Protection, Repair and Rehabilitation, Elsevier, Oxford (2009).
[341] ACI COMMITTEE 207, Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions, ACI 207.3R-94, American Concrete Institute, Farmington Hills, MI (1994).
[342] ACI COMMITTEE 224, Causes, Evaluation, and Repair of Cracks in Concrete Structures, ACI 224.1R-07, American Concrete Institute, Farmington Hills, MI (2007).
[343] ACI COMMITTEE 311, Guide for Concrete Inspection, ACI 311.4R-00, American Concrete Institute, Farmington Hills, MI (2000).

[344] AMERICAN SOCIETY FOR TESTING AND MATERIALS, “Standard test method for pulse velocity through concrete”, ASTM C 857-04, Annual Book of ASTM Standards, American Society for Testing and Materials, West Conshohocken, PA (2004).
[345] NDT JAMES INSTRUMENTS, V-Meter MK IV Ultrasonic Pulse Velocity System for Finding Voids and Cracks, and Determining Other Material Properties (2012), http://www.ndtjames.com/V-Meter-MK-IV-p/v-c-4.htm
[346] DRINKWATER, B., WILCOX, P., Ultrasonic Arrays for Non-Destructive Evaluation: A Review, NDT&E Int. 39 7 (2006) 525–541.
[347] LANGENBERG, K., et al., Ultrasonic Phased Array and Synthetic Aperture Imaging in Concrete, 16th World Conf. on NDT (2004), http://www.ndt.net/article/wcndt2004/pdf/array_transducers/261_mayer.pdf
[348] SCHICKERT, M., “Progress in Ultrasonic SAFT-Imaging of Concrete”, NDT-CE-2003, paper presented Int. Symp. on Non-destr. Test. in Civil Eng 2003, Berlin (2003).
[349] KRAUSE, M., MILMANN, B., STREICHER, D., MÜLLER, W., Ultrasonic Imaging of Concrete Elements: State-of-the-Art Using 2D Synthetic Aperture (Int. Symp. on Non-destr. Testing in Civil Eng 2003) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).
[350] KOVALEV, A., Ultrasonic Testing of Concrete Structures (2002), http://www.ndt.net/article/ecndt02/220/220.htm
[351] MAIERHOFER, C., KRAUSE, M., NIEDERLEITHINGER, E., WIGGENHAUSER, H., Non-destructive Testing Methods at BAM for Damage Assessment and Quality Assurance in Civil Engineering, (Int. Symp. Non-destr. Test. Civil Eng 2003) Federal Institute for Materials Research and Testing, Berlin (2003).
[352] NORDIC INNOVATION CENTER, A Practical Guide to Non-destructive Examination of Concrete, Nordic Innovation Center Report, Force Technology, Helsingborg (2004).
[353] SCHURR, D., Monitoring Damage in Concrete Using Diffuse Ultrasonic Coda Wave Interferometry, Georgia Institute of Technology, Atlanta, GA (2010).
[354] CLAYTON, D., HILEMAN, M., Light Water Reactor Sustainability Nondestructive Evaluation for Concrete Research and Development, ORNL/TM-2012/360, Oak Ridge Natl Lab., TN (2012).
[355] WOODHAM, D., SCHULLER, M., Tomographic Imaging for Investigation of Concrete Structures (Int. Conf. Life Predict. Aging Manag. Concr. Struct, EXPERTCENTRUM, Bratislava (1999).
[356] BOND, L.J., KEPLER, W.F., FRANGOPOL, D.M., Improved assessment of mass concrete dams using acoustic travel time tomography. Part I: Theory, construction and building materials 14 (2000) 147–156
[357] KEPLER, W., BOND, L., FRANGOPOL, D.M., Improved assessment of mass concrete dams using acoustic travel time tomography Part II: Application, construction and building materials 14 (2000) 147–156.
[358] RHAZI, J., Evaluation of Concrete Structures by the Acoustic Tomography Technique, Structural Health Monitoring 5 4 (2006) 333–342.
[359] SANSALONE, M., CARINO, N., Stress Wave Propagation Methods, CRC Handbook on Nondestructive Testing of Concrete (MALHOTRA, V., CARINO, N., Eds), CRC Press, Boca Raton, FL (1991) 275–304.
[360] OLSON INSTRUMENTS, Impact Echo System, http://www.olsoninstruments.com/pdf_downloads/catOI_ie.pdf
[361] GEOVISION, Spectral Analysis of Surface Waves Method (2010), http://www.geovision.com/sasw.php
[362] DAVIS, A., PETERSEN, C., Nondestructive Evaluation of Prestressed Concrete Bridges Using Impulse Response (Int. Symp. Non-Destr. Test. Civil Eng) NDT-CE-2003, Berlin (2003).
[363] DAVIS, A.G., LIM, M., PETERSEN, C., Rapid and economical evaluation of concrete tunnel linings with impulse response and impulse radar non-destructive methods, NDT&E Int. 38 3 (2005) 181–186.
[364] HOLA, J., SADOWSKI, L., SCHABOWICZ, K., Nondestructive identification of delaminations in concrete floor toppings with acoustic methods, Automation in Construction, 20 7 (2011) 799–807.
[365] EWINS, D., Modal Testing: Theory, Practice and Application, Research Studies Press Limited, Hertfordshire (2000).
[366] HE, J., FU, Z.-F., Modal Analysis, Butterworth-Heinemann, Woburn, MA (2001).
[367] PLACHY, T., POLAK, M., Nondestructive damage detection based on modal analysis, Acta Polytechnica 44 5 (2004) 5–6.
[368] OWEN, R., Portable Linear Accelerators for X-Ray and Electron-Beam Applications in Civil Engineering, NDT&E Int. 31 6 (1998) 401–409.
[369] REDMER, B., WEISE, F., EWERT, U., LIKHATCHEV, A., Location of Reinforcement in Structures by Different Methods of Gamma-Radiography (Int. Symp. Non-Destr. Test. Civil Eng) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).
[370] PLA-RUCKI, G., EBERHARD, M., Imaging of Reinforced Concrete: State-Of-The-Art Review, J. Infrastruct. Syst. (1995).
[371] BROWN, K., LEGER, J.S., Use of the MegascanTM Imaging Process in Inspection Systems for Post-Tensioning Bridges and Other Major Structures (Int. Symp. Non-Destr. Test. Civil Eng) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).
[372] HAMASAKI, T., et al., Identification of Reinforced in Concrete by Electro-Magnetic Method (Int. Symp. Non-destr. Test. Civil Eng) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).
[373] PROCEQ USA, Profometer 5+ (2007), http://www.proceq.com/fileadmin/documents/proceq/products/Concrete/Profometer_5_/English/Proceq_Brochure_Profometer_5__E.pdf

[374] PROCEQ SA, Profometer PM-6 Advanced Cover Meters, Brochure 810 392 01E ver 08 2014, Proceq SA, Schwerzenbach (2014).
[375] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography, ASTM D 4788-03, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2003).
[376] WERITZ, F., et al., Investigation of Concrete Structures with Pulse Phase Thermography (Int. Symp. Non-Destr. Test. Civil Eng 2003) NDT-CE-2003, Bundesanstalt, Berlin (2003).
[377] MAIERHOFER, C., BRINK, A., RÖLLIG, M., WIGGENHOUSER, H., Quantitative impulse-thermography as non-destructive testing method in civil engineering – Experimental results and numerical simulations, Construction and Building Materials 19 10 (2005) 731–737.
[378] ZHAO, Y., WU, J., WANG, J. WAN, M., Ground Penetrating Radar Technique and Its Application in Non-Destructive Testing of Reinforced Concrete (10th Asian Pacific NDT Conf., 2001) Brisbane (2001).
[379] BUNGEY, J., Sub-Surface Radar Testing of Concrete: A Review, Construction and Building Materials 18 1 (2004) 1–8.
[380] FORCE TECHNOLOGY INC., Non-destructive Testing (NDT) and Examination (NDE), 2048-3-en, Force Technology Inc., Brøndby (2007).
[381] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding, ASTM D 4580-03, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2003).
[382] COSTLEY, R., BOUDREAUX, G., Finding Delaminations in Concrete Bridge Decks, 146th Acoustical Society of America Meeting, Austin, TX (2003).
[383] KROPP, J., HILSDORF, H.K., Eds, Performance Criteria for Concrete Durability, E&FN Spon, London (1995).
[384] BASHEER, P., 'Clam' Tests for Assessing the Durability of Concrete, PhD thesis, Queen's University, Belfast (1991).
[385] STEINERT, J., Zerstörung Ermittlung der Wassereindringtiefe in Kiesbeton am Bauwerk, Forschungsbeiträge für die Baupraxis, Wilhelm Ernst & Sohn, Berlin (1979) 151–162.
[386] TESTCONSULT, Structures Data Sheet 7 – Initial Surface Absorption Test (2012), http://www.testconsult.co.uk/datasheets.aspx?dept=all#
[387] ELE INTERNATIONAL, ELE Product Catalogue 04_06_09, 35 Concrete: Non-destructive Testing of Hardened Concrete (2009), http://www.ele.com/Product/gwt-permeability-kit-0-6-bar-range-supplied-in-a-carrying-case-
[388] BUNGEY, J., MILLARD, S., GRANTHAM, M., Testing of Concrete in Structures, 4th edn, Taylor & Francis Group, Abingdon (2006).
[389] WOLTER, B., KOHL, F., SURKOWA, N., DOBMANN, G., Practical Applications of NMR in Civil Engineering (International Symposium on Non-Destructive Testing in Civil Engineering) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).
[390] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Test for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete, ASTM C 42-04, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2004).
[391] CENTER OF MATERIAL SCIENCE AND LIFETIME MANAGEMENT, Calculation-Experimental Substantiation the Possibility of Extending the Lifetime of Building Constructions BNS-2 and 2RDES-1, 2, 3 Unit No. 2 Balakovo NPP, Final Technical Report No. 2, TO-001 / 01-12, CMSLM, Moscow (2012).
[392] SUURONEN, J.-P., et al., Analysis of Short Fibres Orientation in Steel Fibre-reinforced Concrete (SFRC) by X-ray Tomography, J. Mater. Sci. (2012).
[393] SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING, Nanyang Technological University, Non Destructive Test Equipment (2012), http://www.ntu.edu.sg/home/cchtui/Webequip.htm#break
[394] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Standard Test Method for Penetration Resistance of Hardened Concrete", ASTM C 803/803M-03, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2004).
[395] PCA NORTHEAST CEMENT SHIPPERS ASSOCIATION, Windsor Probe Technique (2005), http://www.necementshippers.com/displaypic.asp?category=21&sortorder=10&total=21&catname=Testing
[396] PUCINOTTI, R., In situ concrete strength assessment: Influence of the aggregate hardness on the Windsor probe test results, J. Build. Appraisal 5 (2009) 75–85.
[397] PETERSEN, C., LOK-TEST and CAPO-TEST, Twenty Years Experience (Non-Destr. Test. Civil Eng Conf.) Liverpool (1997).
[398] HAMMOND CONCRETE TESTING & SERVICES, Hammond Concrete Testing & Services (2010), http://www.hammond-concrete.co.uk/concrete/concrete1.html
[399] TESTCONSULT SPECIALIST TESTING SERVICES, Internal Fracture Test Datasheet, http://www.testconsult.co.uk/datasheets/Service%20Datasheet%20-%20Internal%20Fracture%20Test.pdf
[400] GERMANN INSTRUMENTS INC., LOK-Test, http://germann.org/wp-content/uploads/2014/04/LOK-test.pdf
[401] GERMANN INSTRUMENTS INC., CAPO-Test, http://germann.org/products/category-3/compressive-strength/capo-test
[402] US DEPARTMENT OF TRANSPORTATION, Mobile Concrete Laboratory Project Reports 9904 SD Tensile Bond Strength of a High Performance Concrete Bridge Deck Overlay (1999), http://www.fhwa.dot.gov/pavement/concrete/mcl9904.cfm
[403] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Standard Test Method for Water-Soluble Chloride in Mortar and Concrete", ASTM C 1218/C 1218M-99, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2006).

[404] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete", ASTM C 1152/C 1152M-04, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2006).

[405] SOHANGHPURWALA, A., Manual on Service Life of Corrosion-Damaged Reinforced Concrete Bridge Superstructure Elements, NCHRP Report 558, National Cooperative Research Program, Transportation Research Board, Washington, DC (2006).

[406] CLEMEÑA, G., APUSEN, C.M., Final Report: An Alternative Potentiometric Method for Determining Chloride Ion Content in Concrete Samples from Reinforced-Concrete Bridges, VTRC 02-R18, Virginia Transportation Research Council, Charlottesville, VA (2002).

[407] ORLOVA, N., WESTALL, J., The Study of Chloride Ion Migration in Reinforced Concrete Under Cathodic Protection, SPR 357, Oregon Department of Transportation, Salem, OR (1999).

[408] ARAI, J., MIZOBUCHI, T., SUDA, K., Study on Measurement of Chloride Content Using Electromagnetic Wave in Reinforced Concrete Structures (Int. Symp. Non-Destr. Test. Civil Eng) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).

[409] WILSCH, G., WERITZ, F., SCHAURICH, D., WIGGENHAUSE, H., Determination of chloride content in concrete structures using laser-induced breakdown spectroscopy, Construction and Building Materials 19 10 (2005) 724–730.

[410] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Standard Practice for Petrographic Examination of Hardened Concrete", ASTM C 856-04, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2006).

[411] PETERSEN, C., POULSEN, E., In-situ NDT Methods for Concrete With Particular Reference to Strength, Permeability, Chloride Content and Disintegration (Proc. of 1st Int. Conf. Deterior. Repair Reinforc. Concr. Arabian Gulf), Construction Industry Research Association, London (1985).

[412] THE CONCRETE SOCIETY, Analysis of Hardened Concrete: A Guide to Tests, Performance and Interpretation of Results, Technical Report 32, The Concrete Society, Surrey (1989).

[413] WALKER, H., LANE, D., STUTZMAN, P., Petrographic Methods of Examining Hardened Concrete: A Petrographic Manual. FHWA-HRT-04-150, US Department of Transportation, Federal Highway Administration, McLean, VA (2006).

[414] US DEPARTMENT OF TRANSPORTATION, Protocol for Selecting Alkali-Silica Reaction (ASR)-Affected Structures for Lithium Treatment, Publication No. FHWA-HRT-06-071, Federal Highway Administration, US Department of Transportation, McLean, VA (2006).

[415] STARK, D., Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures, Report No. SHRP-C-/FR-91-101, Strategic Highway Research Program, National Research Council, Washington, DC (1990).

[416] GUTHRIE, G., CAREY, J., A simple, environmentally friendly, and chemically specific method for the identification and evaluation of the alkali-silica reaction, Cement and Concrete Res. 27 9 (1997) 1407–1417.

[417] WIGUM, B., BJÖRNSDÓTTIR, V., ÓLAFSSON, H., IVERSEN, K., Alkali Aggregate Reaction in Iceland-New Test Methods, Report VH 2007-036, VGK HÖNNUM Consulting Engineers, Reykjavik (2007).

[418] POOLE, A., THOMAS, A., A staining technique the identification of sulfates in aggregates and concrete, Mineralogical Magazine 40 (1975) 315–316.

[419] WERITZ, F., SCHAURICH, D., WILSCH, G., WÖSTMANN, J., WIGGENHAUSER, H., Laser Induced Breakdown Spectroscopy as a Tool for the Characterization and Sorting of Concrete Waste Material in View of High-Order Re-Use, in International Symposium on Non-Destructive Testing in Civil Engineering, NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).

[420] TAFFE, A., SCHAURICH, D., WERITZ, F., WILSCH, G., Laser-Induced Breakdown Spectroscopy (Libs) in Civil Engineering (2006), http://www.ndt.net/article/ecndt2006/doc/P15.pdf

[421] WIGGENHAUSER, H., WILSCH, G., SCHAURICH, D., WÖSTMANN, J., VII.3 - Division of Building Diagnostic; Non-Destructive Testing in Civil Engineering, Federal Institute for Materials Research and Testing, Berlin (2006).

[422] NUCLEAR ENERGY AGENCY, Electrochemical Techniques to Detect Corrosion in Concrete Structures in Nuclear Installations, NEA/CSNI/R(2002)21, Nuclear Energy Agency, Committee on Safety of Nuclear Installation, Paris (2002).

[423] BROOMFIELD, J., Corrosion of Steel in Concrete: Understanding, Investigation and Repair, E&FN Spon, London (1997).

[424] BROOMFIELD, J.P., DAVIES, K., HLADKY, K., The Use of Permanent Corrosion Monitoring in New and Existing Reinforced Concrete Structures, Cement and Concrete Composites 24 1 (2002).

[425] SONG, H.-W., SARASWATHY, V. Corrosion Monitoring of Reinforced Concrete Structures – A Review, Int. J. Electrochemical Sci. 2 (2007) 1–28.

[426] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete", ASTM C 876-91, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2004).

[427] GOWERS, K., MILLARD, S., Measurement of Concrete Resistivity for Assessment of Corrosion Severity of Steel Using Wenner Technique, Title No. 96-M66, ACI Materials Journal (1999).

[428] TULLMIN, M., HANSSON, C., ROBERGE, P.R., "Electrochemical Techniques for Measuring Reinforcing Steel Corrosion", InterCorr/96, paper presented at 1st Global Internet Corrosion Conference (1996).

[429] US DEPARTMENT OF TRANSPORTATION, Direct Measurement Methods (2012), http://www.cflhd.gov/resources/agm/engApplications/BridgeSystemSubstructure/231directMeasurementMethods.cfm

[430] INGVARSSON, H., Non-destructive Condition Assessment of Concrete, Monitoring of Large Structures and Assessment of their Safety, International Association for Bridge and Structural Engineering, Zurich (1987) 65–81.
[431] POLDER, P., et al., Reinforcement Corrosion and Concrete Resistivity: State of the Art Laboratory and Field Results (Int. Conf. Corrosion and Corrosion Protection of Steel in Concr., University of Sheffield, Sheffield Academic Press 1994).
[432] FLIS, J., et al., Electrochemical measurements of bridges for evaluation of reinforcement corrosion rates, Corrosion 49 7 (1993) 601–613.
[433] BERKE, N., HICKS, M., "Estimating the life cycle of reinforced concrete decks and marine piles using laboratory diffusion and corrosion data", Corrosion Forms and Control for Infrastructure ASTM STP 1137 (CHAKER, V. Ed.) ASTM, West Conshohocken, PA (1992) 207–231.
[434] FELIU, S., ANDRADE, C., GONZALEZ, J., ALONSO, A., A New method for in-situ measurement of electrical resistivity of reinforced concrete, Materials and Structures 29 6 (1996) 362–365.
[435] RILEM TC 154-EMC, Electrochemical techniques for measuring metallic corrosion, Materials and Structures 37 (2004) 623–643.
[436] ANDRADE, C., MARTINEZ, I., Electrochemical Corrosion Rate Measurement Using Modulated Confinement of the Current: Calibration of this Method by Gravimetrics Losses (2003), http://www.ndt.net/article/ndtce03/papers/v048/v048.htm.
[437] ELSENER, B., WOJTAS, H., BOHNI, H., Galvanostatic Pulse Measurements — Rapid On Site Corrosion Monitoring (Proc. Int. Conf. Univ. of Sheffield, 1994) Sheffield Academic Press, Sheffield (1994).
[438] NEWTON, C., SYKES, J., A galvanic pulse technique for investigation of steel corrosion in concrete, Corrosion Science 28 (1988) 1051–1073.
[439] KLINGHOFFER, O., In-situ monitoring of reinforcement corrosion by means of electro-chemical methods, Nordic Concrete Research 95 1 (1995).
[440] ELSENER, B., BOHNI, H., "Potential mapping and corrosion of steel in concrete", ASTM STP 1056, Corrosion Rates of Steel in Concrete (BERKE, N., CHAKER, V., WHITING, D., Eds.) American Society for Testing and Materials, West Conshohocken, PA (1990) 143–156.
[441] JOLIVET, D., BONEN, D., SHAH, S., The corrosion resistance of coated steel dowels determined by impedance spectroscopy, Cement and Concrete Res. 37 (2007) 1134–1143.
[442] SHI, M., CHEN, Z., SUN, J., Determination of Chloride Diffusivity in Concrete by AC Impedance Spectroscopy, Cement and Concrete Res. 29 (1999) 1111-1115.
[443] MacDONALD, D., El-TANTAWY, Y., ROCHA-FILHO, R., URQUIDE-MacDONALD, M., Evaluation of Electrochemical Impedance Technology for Detecting Corrosion of Rebar in Reinforced Concrete, SHRP-ID/UFR-91-524, Vol. 1, Strategic Highway Research Program, National Research Council, Washington, DC (1991).
[444] MEHTA, P., MONTEIRO, P., Concrete: Microstructure, Properties, and Materials, 3rd Edition, McGraw Hill Companies, Inc., New York (2006).
[445] NUCLEAR REGULATORY COMMISSION, Issuance of Final Amendment to 10 CFR 50.55a to Incorporate by Reference the ASME Boiler and Pressure Vessel Code (ASME Code), Section XI, Subsection IWE and Subsection IWL, SECY-96-080, NRC, Washington, DC (1996).
[446] DOUGAN, J., Evaluation of In-Service Inspections of Greased Prestressing Tendons, Rep. NUREG/CR-2719 (ORNL/TM-8278), NRC, Washington, DC (1982).
[447] CIOLKO, A., TABATABAI, H., Nondestructive Methods for Condition Evaluation of Prestressing Steel Strands in Concrete Bridges, NCHRP Project 10-53, National Research Council, Skokie, IL (1999).
[448] AZIZINAMINI, A., GULL, J., Improved Inspection Techniques for Steel Prestressing/Post-tensioning Strand, FDOT Contract No. BDK80 977-13, Florida Department of Transportation, Miami, FL (2012).
[449] NAUS, D., Inspection of Nuclear Power Plant Structures – Overview of Methods and Related Applications, ORNL/TM-2007/191, Oak Ridge Natl Lab., TN (2009).
[450] WORMAN, J., Liquid Penetrant Examination, The National Board of Boiler and Pressure Vessel Inspectors (2011), http://www.nationalboard.org/index.aspx?pageID=164&ID=374.
[451] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Test Methods for Measuring Adhesion by Tape Test, ASTM D3359 - 09e2, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2009).
[452] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Test Method for Adhesion-in-Peel of Elastomeric Joint Sealants, ASTM C794 - 10, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2010).
[453] BAMU, P., ZINGONI, A., Damage, deterioration and the long-term structural performance of cooling-tower shells: A survey of developments over the past 50 years, Engineering Structures, 27 (2005) 1794–1800.
[454] ELECTRIC POWER RESEARCH INSTITUTE, Facilities Maintenance Guide, EPRI, Palo Alto, CA (2004).
[455] ELECTRIC POWER RESEARCH INSTITUTE, Nondestructive Evaluation: Buried Pipe NDE-Revision 2, Report 1025220, EPRI, Palo Alto, CA (2012).
[456] JOINT DEPARTMENTS OF ARMY, NAVY AND AIR FORCE, "Nondestructive evaluation methods", Masonry Structural Design for Buildings, TM 5-809-3/NAVFAC DM-2.9/AFM 88-3, Joint Departments of Army, Navy and Air Force, Washington, DC (1992).

[457] AMERICAN SOCIETY FOR TESTING AND MATERIALS, “Standard test method for in situ measurement of masonry mortar joint shear strength index”, ASTM C 1197-04, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2004).

[458] AMERICAN SOCIETY FOR TESTING AND MATERIALS, “Standard test methods for in situ measurement of masonry deformability properties using flatjack method”, ASTM C 1531-03, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2004).

[459] AMERICAN SOCIETY FOR TESTING AND MATERIALS, “Standard test method for in situ compressive stress within solid unit masonry using flatjack method”, ASTM C 1196-04, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2004).

[460] ESPOSITO, E., COPPARONI, S., NATICCHIA, B., Recent Progress in Diagnostics of Civil Structures by Laser Vibrometry (2004), http://www.ndt.net/abstract/wcndt2004/713.htm

[461] MAIERHOFER, C., ZIEBOLZ, A., KÖPP, C., ONSITEFORMASONRY - A European Research Project: On-site investigation techniques for the structural evaluation of historic masonry (2003), http://www.ndt.net/article/ndtce03/papers/v070/v070.htm

[462] MAIERHOFER, C., KRAUSE, M., WIGGENHAUSER, H., Non-destructive Investigation of Sluices Using Radar and Ultrasonic Impulse Echo (Proc. 7th Int. Conf. Structural Faults and Repair, Edinburgh, 1997) Engineering Technics Press (1997).

[463] COLLA, C., FORDE, M., DAS, P., Radar Imaging in Composite Masonry Structures (Proc. 7th Int. Conf. Structural Faults and Repair, Edinburgh, 1997) Engineering Technics Press (1997).

[464] WENDRICH, A., et al., Location of Voids in Masonry Structures by Using Radar and Ultrasonic Traveltime Tomography (2006), http://www.ndt.net/article/ecndt2006/doc/Tu.3.2.5.pdf

[465] PIERACCINI, M., PISANESCHI, M., NOFERINI, L., ATZENI, C., Polarimetric Radar Signature of Masonry Walls, NDT&E Int. 40 (2007) 271–271.

[466] TAFFE, A., MAIERHOFER, C., Guidelines for NDT Methods in Civil Engineering (Int. Symp. Non-Destr. Test. Civil Eng) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).

[467] MAIERHOFER, C., LEIPOLD, S., WIGGENHAUSER, H., Investigation of the Influence of Moisture and Salt Content on the Dielectric Properties of Brick Materials Using Radar (Proc. 7th Int. Conf. Ground Penetrating Radar (GPR), Lawrence, Kansas, 1998).

[468] SCHULLER, M., BERRA, M., ATKINSON, R., BINDA, L., Acoustic Tomography for Evaluation of Unreinforced Masonry, Constr. Build. Mater. 11 3 (1997) 199–204.

[469] HERTLEIN, B., DAVIS, A., Nondestructive Testing of Deep Foundations, John Wiley & Sons, Chichester (2006).

[470] RAUSCHE, F., Non-Destructive Evaluation of Deep Foundations (Proc. Fifth Int. Conf. Case Histories in Geotechnical Eng, 2004) New York (2004).

[471] FINNO, R., GASSMAN, S.L., OSBORN, P.W., Non-destructive Evaluation of a Deep Foundation Test Section at the Northwestern University National Geotechnical Experimentation Site, Federal Highway Administration, McLean, VA (1997).

[472] NIEDERLEITHINGER, E., TAFFE, A., Early Stage elastic wave velocity of concrete piles, Cement and Concrete Comp. 28 4 (2006) 317–320.

[473] BRITISH RESEARCH ESTABLISHMENT, Proc. Int. Conf. Reuse of Foundations for Urban Sites, Bracknell (2006).

[474] NIEDERLEITHINGER, E., TAFFE, A., Concept for Reference Pile Testing Sites for the Development and Improvement of NDT-CE (Int. Symp. Non-Dest. Test. Civil Eng 2003) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).

[475] NUCLEAR REGULATORY COMMISSION, Foundations, Standard Review Plan, Rep. NUREG-0800, Revision 2, Section 3.8.5, NRC, Washington, DC (2007).

[476] NUCLEAR REGULATORY COMMISSION, Groundwater, Standard Review Plan, Rep. NUREG-0800, Revision 2, Section 2.4.12, NRC, Washington, DC (2007).

[477] BUILDING RESEARCH ESTABLISHMENT, Concrete in Aggressive Ground, BRE Special Digest 1, Building Research Establishment, Garston (2005).

[478] Bowley, M.J., Sulfate and Acid Attack on Concrete in Ground: Recommended Procedures for Soil Analysis, BRE Report 279, Building Research Establishment, Garston (1995).

[479] INTERNATIONAL ATOMIC ENERGY AGENCY, Maintenance, Surveillance and In-service Inspection in Nuclear Power Plants, IAEA Safety Standards Series No. NS-G-2.6, IAEA, Vienna (2002).

[480] INTERNATIONAL ATOMIC ENERGY AGENCY, Review of the Methods used for Leak Rate Measurements for WWER-440/230 Confinements and WWER-440/213 Containments, IAEA-EBP-WWER-10, IAEA, Vienna (1998).

[481] HOHMANN, B.P., BRUCK, P., ESSELMAN, T.C., SCHMIDT, T., Digital Image Correlation (DIC): An Advanced Nondestructive Testing Method for Nuclear Power Plants, IAEA-CN-194-016, a paper presented at the Third Int. Conf. Nuclear Power Plant Life Manag. (PLiM) , Salt Lake City, UT (2012).

[482] TCHERNER, J., Aging Management of CANDU NPP Concrete Structures, presentation made at IAEA Consultancy Meeting, Assessment and Ageing of Concrete Structures May 29 to June 1, 2012, Candu Energy, Vienna (2012).

[483] TCHERNER, J., AZIZ, T.S., PANESAR, D.K., DEMERS, M., NEALE, K.W., Performance Surveillance of Gentilly-1 Reactor Building GFRP Repair Using Fiber Optic Sensors and Strain Gauges, SMiRT 20 paper 1848, paper presented at the 20th Int. Conf. Structural Mechanics in Reactor Technology, Espoo (2009).
[484] STONE & WEBSTER ENGINEERING CORPORATION, Containment Integrated Leak-Rate Testing Improvements, EPRI NP-2726, Electric Power Research Corporation, Palo Alto, CA (1982).
[485] POSTEMA, F.J., VAN BEEK, A., NDT Used in the Netherlands from a Principal Point of View (Int. Symp. Non-Destructive Test. Civil Eng) NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).
[486] AL-NESHAWY, F., PIIRONEN, J., PELTOLA, S., SISTONEN, E., PUTTONEN, J., Network system for assessing the moisture and thermal behaviour of repaired concrete building façades, Electr. J. Inf. Technol. Constr. 16 (2011) 601–616.
[487] BRAMESHUBER, W., RAUPACH, M., SCHRÖDER, S., DAUBERSCHMIDT, A.C., Non-Destructive Determination of the Water Content in the Concrete Cover Using the Multiring Electrode, Int. Symp. Non-Destructive Test. Civil Eng, NDT-CE-2003, Federal Institute for Materials Research and Testing, Berlin (2003).
[488] KRAUSE, H.-J., DANG, C., KLEIN, N., DUMAT, F., RATH, E., “Dielectric microwave resonator for non-destructive evaluation of moisture and salinity”, paper presented at the Eur. Conf. NDT, Berlin (2006).
[489] CESCOR SRL, Ti MMO Electrode, http://www.cescor.net/uploads/prodotti/ti-mmo-eng.pdf
[490] GERMANN INSTRUMENTS, ERE-Probe, http://www.germann.org/TestSystems/ERE-Probe/ERE%20Probe.pdf
[491] HTLINNOVATIV, Nondestructive measurement of the corrosion of reinforced and prestressed concrete structures (2003), http://www.htl-innovativ.at/index.php?lang=eng&modul=detail&id=1348
[492] FORCE TECHNOLOGY, CorroWatch Multisensor, http://www.forcetechnology.com/en/Menu/Products/Concrete-monitoring/Concrete-monitoring-probes/corrowatchmultisensor.htm
[493] SENSORTEC, Corrosion Sensor ANODE LADDER (2010), http://www.sensortec.de/images/stories/pdf/Data_Sheet_Anode_Ladder_eng.pdf
[494] BJEGOVIC, D., MIKULIC, D., SEKULIC, D., “Non-destructive corrosion rate monitoring for reinforced concrete structures”, paper in the 15th World Conf. on Non-Destructive Testing, Rome (2000).
[495] SENSORTECH GMBH, Specification of the Anode Ladder Corrosion Sensor (2009), http://www.sensortec.de/images/pdf/Installation_manual_AL_eng.pdf
[496] RAUPACH, M., SCHIESSL, P., Macrocell sensor for monitoring of the corrosion risk of the reinforcement in concrete structures, NDT&E Int. 34 6 (2001) 435–442.
[497] VIRGINIA TECHNOLOGIES INC, Embedded Corrosion Instrument: For Non-Destructive Evaluation of Steel-Reinforced Concrete, Virginia Technologies, Inc., Charlottesville, VA (2005).
[498] DENG, L., CAI, C., Applications of fiber optic sensors in civil engineering, Struct. Eng Mech. 25 5 (2007) 577–596.
[499] YE, X.W., SU, Y.H., HAN, J.P., Structural Health Monitoring of Civil Infrastructure Using Optical Fiber Sensing Technology: A Comprehensive Review, Article ID 652329, The Scientific World J. (2014) 11.
[500] NDT RESOURCE CENTER, Introduction to Acoustic Emission Testing, NDT Resource Center, http://www.ndt-ed.org/EducationResources/CommunityCollege/Other%20Methods/AE/AE_Intro.htm
[501] HAMSTAD, M.A., What is Acoustic Emission Technology? (2000), http://www.engr.du.edu/profile/Marvin.htm
[502] MURAVIN, G., Inspection, Diagnostics and Monitoring of Construction Materials and Structures by Acoustic Emission Method, Minerva Press, London (2000).
[503] GROSSE, C., KRÜGER, M., GLASER, S., “Wireless acoustic emission sensor networks for structural health monitoring in civil engineering”, paper presented at the Eur. Conf. on Non-Destructive Testing, Berlin (2006).
[504] PAULSON, P., “The use of acoustic monitoring to manage concrete structures” NDT-CE-2003, paper presented at the Int. Symp. On Non-Destructive Testing Civil Eng, Berlin (2003).
[505] PAULSON, P.O., TOZSER, O., YOUDAN, D., The Use of Acoustic Monitoring to Manage Concrete Structures in the Nuclear Industry, Instrumentation and Monitoring of Concrete Structures (Proc. Workshop Inst. Monitor. Concr. Struct., Brussels, 2001) NEA/CSNI/R(2000)15, Committee on Safety of Nuclear Installations, Isy-les-Moulineaux (2001).
[506] VOGEL, T., SCHECHINGER, B., FRICKER, S., “Acoustic emission analysis as a monitoring method for prestressed concrete structures”, paper presented at the Eur. Conf. on Non-Destructive Testing, Berlin, 2006.
[507] BERÉNYI, A., LOVAS, T., Remote Sensing, MSc University Textbook (2011), http://www.fmt.bme.hu/fmt/oktatas/feltoltesek/BMEEOFTMK08/taverzekeles_msc_jegyzet.pdf
[508] ALBA, M., FREGONESE, L., PRANDI, F., SCAIONI, M., VALGOI, P., Structural Monitoring of a Large Dam by Terrestrial Laser Scanning (2012), http://www.isprs.org/proceedings/XXXVI/part5/paper/1271_Dresden06.pdf, 05.10.2012
[509] BERÉNYI, A., LOVAS, T., Laser Scanning in Deformation Measurements Acquiring 3D Spatial Data of Bridges, GIM Int. March Vol. (2009) 17–21.
[510] COURTOIS, A., VACQUE, A., GENEST, Y., “In service inspection and monitoring of Reinforced Concrete Draught Cooling Towers”, EPRI-EDF-MAI Workshop, Joint Electr. Power Res. Inst./EDF/Mater. Ageing Inst. Workshop Insp. Degrad. Manag. Concr. Struct. Nucl. Ind., Moret-sur-Loing, France (2011) 16.
[511] NACE INTERNATIONAL, NACE Standard Practice NACE SP0169-2013 Control of External Corrosion on Underground or Submerged Metallic Piping Systems, NACE, Houston, TX (2013).

[512] INTERNATIONAL STANDARDS ORGANIZATION, Petroleum and Natural Gas Industries: Cathodic Protection of Pipeline Transportation Systems Part 1: On-land Pipelines, ISO 15589-1:2003, ISO, Geneva (2003).
[513] EUROPEAN COMMITTEE FOR STANDARDIZATION, Cathodic Protection of Buried or Immersed Metallic Structures – General Principles and Application for Pipelines, EN 12954:2001, CEN, Brussels (2001).
[514] AUSTRALASIAN CORROSION ASSOCIATION, Cathodic Protection of Metals: Part 2: Compact buried structures, AS 2832.2, Standards Australia, ACA, Victoria (2003).
[515] CANADIAN GAS ASSOCIATION, Recommended Practice OCC-1-2013 Control of External Corrosion on Buried or Submerged Metallic Piping Systems, Canadian Gas Association, Ottawa (2013).
[516] GOSSTANDART OF RUSSIA, GOST R 51164-98 Steel Main Pipelines - General Requirements for Protection Against Corrosion, Gosstandart of Russia, Moscow (1998).
[517] WADDELL, J., Basic Steps of a Concrete Repair Programme, Concr. Int., 2 9 (1980) 30–33.
[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.
[519] INTERNATIONAL FEDERATION FOR STRUCTURAL CONCRETE, Structural Concrete – Textbook on Behaviour, Design and Performance, 2nd Edn, Vol. 5, FIB, Lausanne (2012).
[520] PFEIFER, D. S. M., "Concrete Sealers for Protection of Bridge Structures", NCHRP Report No. 244, National Cooperative Highway Research Programme, Washington, DC (1981).
[521] ACI COMMITTEE 201, 201.1R-08 Guide for Conducting a Visual Inspection of Concrete in Service, ACI, Detroit, MI (2008).
[522] ACI COMMITTEE 546, 546.3R-06 Guide for the Selection of Materials for the Repair of Concrete, ACI, Farmington Hills, MI (2006).
[523] ACI COMMITTEE 546, ACI 546.2R-10 Guide to Underwater Repair of Concrete, ACI, Farmington Hills, MI (2010).
[524] AMERICAN CONCRETE INSTITUTE, Concrete Repair Manual, ACI, Farmington Hills, MI (2008).
[525] BUREAU OF RECLAMATION, Standard Specification for the Repair of Concrete, M-47, M0470000.390, US Department of the Interior, Denver, CO (1990).
[526] UNITED STATES ARMY CORPS OF ENGINEERS, The REMR Notebook, USACE, Washington, DC (1992).
[527] NUCLEAR ENERGY STANDARDS COORDINATION COLLABORATIVE, Concrete Codes and Standards for the Repair of Nuclear Power Plant Structures: Recommendations for Future Developments, Summer 2013, NIST/ANSI, Gaithersburg, MD (2013).
[528] GERMAN COMMITTEE ON REINFORCED CONCRETE, DAfStb-Richtlinie - Schutz und Instandsetzung von Betonbauteilen (Instandsetzungs-Richtlinie) - Teil 1: Allgemeine Regelungen und Planungsgrundsätze; Teil 2: Bauprodukte und Anwendung; Teil 3: Anforderungen an die Betriebe und Überwachung der Ausführung, Deutscher Ausschuss für Stahlbeton e. V., Berlin (2001).
[529] THE CONCRETE SOCIETY, Repair of Concrete Damaged by Reinforcement Corrosion, Technical Report 26, The Concrete Society, London (1984).
[530] PULLAR-STRECKER, P., Corrosion Damaged Concrete Assessment and Repair, Butterworths, London (1987).
[531] DEPARTMENT OF TRANSPORT, Code of Practice for Routine Maintenance, Department of Transport, London (1985).
[532] DEPARTMENT OF TRANSPORT, Materials for the Repair of Concrete Highway Structures, Department of Transport, London (1986).
[533] COMITE EURO-INTERNATIONAL DU BETON, Behavior in Service, Maintenance, and Repair, Bulletin 163, Comite Euro International du Beton, Paris (1983).
[534] FEDERATION INTERNATIONALE DE LA PRECONTRAINTE, Inspection and Maintenance of Reinforced and Prestressed Concrete Structures, FIP Guide to Good Practice, Federation Internationale de la Precontrainte, Thomas Telford, London (1986).
[535] AUSTRIAN STANDARDS INSTITUTE, Austrian Standard ÖNORM Betonbau; Instandsetzung, Umbau und Verstärkung B 4200-6 (Conf. on Repair and Maint. of Concr. and Reinforced Concr. Struct., Innsbruck, 1988) ASI, Vienna (1988).
[536] AUSTRIAN STANDARDS INSTITUTE, ÖNORM B 4706 Instandsetzung, Umbau und Verstärkung von Betonbauten: Allgemeine Regeln und nationale Umsetzung der ÖNORM EN 1504 (Berichtigung), ASI, Vienna (2009).
[537] THE CONCRETE SOCIETY, Repair of concrete structures with reference to BS EN 1504, Technical Report TR69, The Concrete Society, Camberley (2009).
[538] BRITISH STANDARDS INSTITUTION, Products and Systems for the Protection and Repair of Concrete Structures, Definitions, Requirements, Quality Control and Evaluation of Conformity, Structural and Non-structural Repair, BS EN 1504-3:2005, BSI, London (2005).
[539] McDONALD, J., CAMPBELL, R.S., The Condition of the Corps of Engineers Civil Works Concrete Structures, US Army Corps of Engineers Waterways Experiment Station, US Army Corps of Engineers, Vicksburg, MS (1985).
[540] DARWIN, D., Debate: Crack width, cover, and corrosion, Concrete Int. 7 5 (1985) 29–35.
[541] CAMPBELL-ALLEN, D.L.B., Cracks in Concrete Bridge Decks, Civil Engineering Research Report R313, University of Sydney, Sydney (1978).

[542] BEEBY, A., The prediction of crack width in hardened concrete, Structural Eng 57A 1 (1979) 9–17.
[543] ACI COMMITTEE 224, Causes, evaluation, and repair of cracks in concrete structures, Report No. 224R-84, J. Am. Concr. Inst. 81 3 (1984) 211–230.
[544] AMERICAN CONCRETE INSTITUTE/AMERICAN SOCIETY OF MECHANICAL ENGINEERS, Code for Concrete Reactor Vessels and Containments, Section HI, Division 2, ASME Boiler and Pressure Vessel Code (ACI Standard 359-92), ASME, New York (1985).
[545] ACI COMMITTEE 546, Guide for Repair of Concrete Bridge Superstructures, ACI 546.1R-80, Manual of Concrete Practice, Part 2, American Concrete Institute, Detroit, MI (1988).
[546] UNITED STATES ARMY CORPS OF ENGINEERS, Chemical Grouting, Engineer Manual EM 1110-2-3504, Office of Chief of Engineers, USACE, Washington, DC (1973).
[547] STARK, D., MORGAN, B., OKAMOTO, P., Eliminating or Minimizing Alkali-Silica Reactivity, SHRP-C-343, Strategic Highway Research Programme, National Research Council, Washington, DC (1993).
[548] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Standard test method for determination of length change of concrete due to alkali-silica reaction", ASTM C1293, Annual Book of ASTM Standards, Vol. 04.02, ASTM, West Conshohocken, PA (2008).
[549] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Standard test method for potential alkali reactivity of carbonate rocks for concrete aggregate (rock cylinder method)", ASTM C 586, Annual Book of ASTM Standards, Vol. 04.02, ASTM, West Conshohocken, PA (1991).
[550] HELMUTH, R., et. al., Alkali-Silica Reactivity: An Overview of Research, SHRP-C-342, Strategic Highway Research Programme, National Research Council, Washington, DC (1993).
[551] HOBBS, D., Alkali-Silica Reaction in Concrete, Thomas Telford Ltd, London (1988).
[552] HOPPE, G., "Rehabilitation of an Arch Bridge, Concrete Alkali-Aggregate Reactions" (Proc. 7th Int. Conf. Ottawa, 1986) Noyes Publications, Park Ridge, NJ (1987).
[553] CAVALCANTI, A., "Alkali-Aggregate Reaction at Moxoto Dam, Brazil, Concrete Alkali-Aggregate Reactions" (Proc. 7th Int. Conf. Ottawa, 1986) Noyes Publications, Park Ridge, NJ (1987).
[554] ACI COMMITTEE 222, Corrosion of Metals in Concrete, ACI 222R-89, American Concrete Institute, Detroit, MI (1989).
[555] TAROU, K., BOSHART, S., Hyperbolic Cooling Tower Column and Lintel Beam Protection, Structure (2007) 15–19.
[556] VSL INTERNATIONAL, External Post Tensioning, Design Considerations, VSL External Tendons Examples from Practice, VSL International Ltd., Lyssach (1992).
[557] OECD NUCLEAR ENERGY AGENCY, Report of the Task Group Reviewing International Activities in the Area of Ageing of Nuclear Power Plant Concrete Structures, NEA/CSNI/R(95)19, OECD-NEA, Issy-les-Moulineaux (1995).
[558] SHAH, V.N., SMITH, S., SINHA, U., Insights for Aging Management of Light Water Reactor Components: Metal Containments, Rep. NUREG/CR-5324 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, ID (1994).
[559] ANSI/AWS, D3.6M:2010, Underwater Welding Code, American Welding Society, Doral, FL (2010).
[560] 560 ELECTRIC POWER RESEARCH INSTITUTE, Welding and Repair Technology Center: Mitigation of Nuclear Fuel Pool Leaks, EPRI, Palo Alto, CA (2012).
[561] DAWSON, D., Cured-in-place Pipe: Trenchless Trends (2012), http://www.compositesworld.com/articles/cured-in-place-pipe-trenchless-trends
[562] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Pulled in Place Installation of Glass Reinforced Plastic (GRP) Cured-in-Place Thermosetting Resin Pipe (CIPP), ASTM F2019-11, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2011).
[563] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-impregnated Tube, ASTM F1216-09, Annualm Book of ASTM Standards, ASTM, West Conshohocken, PA (2009).
[564] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Standard Practice for Rehabilitation of Existing Pipelines and Conduits by Pulled-in-place Installation of Cured-in-place Thermosetting Resin Pipe (CIPP), ASTM F1743-08, Annual Book of ASTM Standards, ASTM, West Conshohocken, PA (2008).
[565] ALKHRDAJI, T., ROCCA, S., THOMAS, J., "The development of a novel steel reinforced composite (SRC) liner for the rehabilitation of deteriorated PCCP", Pipelines 2012: Innovations in Design, Construction, Operations, and Maintenance, Doing More with Less, (Proc. Conf. Miami Beach, 2012), America Society of Civil Engineers, Florida (2012).
[566] TOWNSEND, F.C., ANDERSON, J.B., A Compendium of Ground Modification Techniques, Florida Department of Transport Report BC 354, RPWO No. 64, Florida Department of Transport, Talsahassee, FL (2004).
[567] SOUTH CAROLINA DEPARTMENT OF TRANSPORTATION, Geotechnical Design Manual, Version 1.1, South Carolina Department of Transportation, Columbia, SC (2010) 89.
[568] FULLER, A., WESTLAND, J., BLAKITA, P., TZAMBAZIS, V., Shoring Up, Civil Eng 77 3 (2007) 56–61.
[569] NUCLEAR STREET NEWS, Underwater Construction Corporation and TFT Nuclear Coating System Benefits a Nuclear Plant Life Extension Program (2012), http://nuclearstreet.com/nuclear_power_industry_news/b/nuclear_power_news/

archive/2012/08/17/underwater-construction-corporation-and-tft-nuclear-coating-system-benefits-a-nuclear-plant-life-extension-program-08171.aspx
[570] UNDERWATER CONSTRUCTION CORPORATION, Decommissioning of Nuclear Storage Basin (2008), http://www.uccdive.com/decommissioning-of-nuclear-storage-basin/
[571] BUREAU OF INDIAN STANDARDS, Plain and Reinforced Concrete — Code of Practice, IS 456, 4th Revision, BIS, New Delhi (2000).
[572] INTERNATIONAL ATOMIC ENERGY AGENCY, Safety Aspects of Nuclear Power Plant Ageing, IAEA-TECDOC-540, Vienna, IAEA, Vienna (1990).
[573] INTERNATIONAL ATOMIC ENERGY AGENCY, Methodology for the Management of Ageing Nuclear Power Plant Components Important to Safety, Technical Report Series No. 338, IAEA, Vienna (1992).
[574] INTERNATIONAL ATOMIC ENERGY AGENCY, Specialists Meeting on the Effectiveness of Methods for the Detection and Monitoring of Age-Related Degradation in Nuclear Power Plants, Internal Report, IAEA, Vienna (1996).
[575] INTERNATIONAL ATOMIC ENERGY AGENCY, AMAT Guidelines: Reference Document for IAEA Ageing Management Assessment Teams (AMATs), IAEA Services Series No. 4, Vienna (1999).
[576] INTERNATIONAL ATOMIC ENERGY AGENCY, Assessment and Management of Ageing of Ageing of Major Nuclear Power Plant Components Important to Safety: Metal Components of BWR Containment Systems, IAEA-TECDOC-1181, IAEA, Vienna (2000).
[577] INTERNATIONAL ATOMIC ENERGY AGENCY, IAEA Guidance on Ageing Management for Nuclear Power Plants — Version 1, 2002, IAEA, Vienna (2002).
[578] INTERNATIONAL ATOMIC ENERGY AGENCY, Plant Life Management for Long Term Operation of Light Water Reactors, Technical Report Series No. 448, IAEA, Vienna (2005).
[579] INTERNATIONAL ATOMIC ENERGY AGENCY, Safety Aspects of Long Term Operation of Water Moderated Reactors, IAEA, Vienna (2007).
[580] INTERNATIONAL ATOMIC ENERGY AGENCY, SALTO Guidelines, IAEA Services Series No. 17, IAEA, Vienna (2008).
[581] INTERNATIONAL ATOMIC ENERGY AGENCY, SALTO Peer Review Guidelines, IAEA Services Series No. 26, IAEA, Vienna (2014).
[582] INTERNATIONAL ATOMIC ENERGY AGENCY, Nuclear Power Plant Life Management Processes: Guidelines and Practices for Heavy Water Reactors, IAEA-TECDOC-1503, IAEA, Vienna (2006).
[583] INTERNATIONAL ATOMIC ENERGY AGENCY, Proactive Management of Ageing for Nuclear Power Plants, Safety Report Series No. 62, IAEA, Vienna (2009).
[584] INTERNATIONAL ATOMIC ENERGY AGENCY, Welcome to IGALL Database (2013), http://gnssn.iaea.org/sites/auth/NSNI/ExtPRJ/IGALL/DB/SitePages/Home.aspx
[585] INTERNATIONAL ATOMIC ENERGY AGENCY, Approaches to Ageing Management for Nuclear Power Plants: International Generic Ageing Lessons Learned (IGALL) Final Report, IAEA-TECDOC-1736, Vienna, Austria (2014).
[586] ATOMIC ENERGY REGULATORY BOARD, Containment System Design for Pressurized Heavy Water Reactors, AERB Safety Guide No. AERB/NPP-PHWR/SG/D-21, AERB, Mumbai (2007).
[587] Training Program NSRTS, Technological Systems MFCC, course, Composition, Purpose. Operation Modes, Desnogorsk (2001).
[588] DEMENTEV, B., Nuclear Power Reactors, Energoatomizdat, Moscow (1990).
[589] NAUS, D.J., OAK RIDGE NATIONAL LABORATORY, Concrete Structures in Existing and New Reactor Systems — Description and Review of Thermal Loading Tests in Support of Their Design, Oakridge, TN, unpublished data.
[590] WESTINGHOUSE ELECTRIC COMPANY LLC., AP1000, Simple, Safe Innovative, Brochure 85065A, Rev.2 0110 2M, Westinghouse Electric Company LLC., Cranberry Township, PA (2010).
[591] FRAMATOME ANP, EPR Design, Framatome ANP Inc., Lynchburg, VA (2005).
[592] SWEET, W., Advanced Reactor Development Rebounding, IEEE Spectrum 34 (1997) 41–48.
[593] SHAPIRO, H., KRISHNAN, V., SANTAMAURA, P., LEKAKH, B., BLAHNIK, C., "Design features of ACR in severe accident mitigation", Evaluation of Uncertainties in Relation to Severe Accidents and Level-2 Probabilistic Safety Analysis (Proc. Workshop, Aix-en-Provence, 2005), NEA, Aix-en-Provence (2005).
[594] INTERNATIONAL ATOMIC ENERGY AGENCY, Status of Advanced Light Water Cooled Reactor Designs — 1996, IAEA-TECDOC-968, IAEA, Vienna (1996) 313–342.
[595] NUCLEAR REGULATORY COMMISSION, Final Safety Evaluation Report Related to the Certification of the System 80+ Design, Rep. NUREG-1462, Office of Nuclear Reactor Regulation, Washington, DC 1994.
[596] GE NUCLEAR ENERGY, ABWR Design Control Document, Rev. 4, GE Nuclear Energy (1997).
[597] HIRAMA, T., et al., Seismic Proof Test of a Reinforced Concrete Containment Vessel (RCCV), Part 1: Test Model and Pressure Test, Nucl. Eng Des. 235 (2005) 1335–1348.
[598] GE HITACHI NUCLEAR ENERGY, ABWR Design Control Document Tier 2, Revision 5 (2010), http://pbadupws.nrc.gov/docs/ML1100/ML110040261.pdf

[599] GE HITACHI NUCLEAR ENERGY, ESBWR Design Control Document Tier 2, GE Hitachi Nuclear Energy (2007), http://adamswebsearch2.nrc.gov/idmws/ViewDocByAccession.asp?AccessionNumber=ML052450245
[600] INTERNATIONAL ATOMIC ENERGY AGENCY, Status of Advanced Light Water Reactor Designs 2004, IAEA-TECDOC-1391, IAEA, Vienna (2004).
[601] BABCOCK & WILCOX NUCLEAR ENERGY, B&W mPower Reactor Design Overview, Technical Report 08-00000341-000(NP), Babcock & Wilcox Nuclear Energy, Charlotte, NC (2010).
[602] ADAMS, R., Nuclear Power for Galena, Alaska (2005), http://atomicinsights.com/2005/03/nuclear-power-for-galena-alaska.html
[603] BURNS AND ROE, Safety Features of the Containment of a 4S Reactor-based Power Generation Facility, Rev 01 (2007), http://www.uxc.com/smr/Library%5CDesign%20Specific/4S/Papers/2007%20-%20Safety%20Features%20of%20the%20Containment%20of%20a%204S%20Reactor-Based%20Power%20Generation%20Facility.pdf
[604] UNITED STATES DEPARTMENT OF ENERGY, Galena Electric Power — a Situational Analysis, USDOE, National Energy Technology Laboratory (2004), http://www.uxc.com/smr/Library%5CDesign%20Specific/4S/Papers/2004%20-%20Galena%20Electric%20Power%20-%20A%20Situational%20Analysis.pdf
[605] JAVOR, T., "Instrumentation systems for monitoring the performance of new and existing nuclear power plants", Considerations for Use in Managing the Aging of Nuclear Power Plant Concrete Structures — State-of-the-Art Report of RILEM TC 160-MLN, RILEM, Bagneux (1999) 39–62.
[606] CRAGG, C., Overview of Aging Management Programmes for Concrete Containment Structures at Ontario Hydro (IAEA Co-ordinated Research Programme Meeting in Vienna, 1993) IAEA, Vienna (1993).
[607] BACOT, J., "Maintenance of Major EdF Components", SMiRT-13, Transactions of 13th Int. Conf. on Structural Mechanics in Reactor Technology, Vol. 1 (Proc. 13th Int. Conf. Porto Alegre, Brazil, 1995), IASMiRT, Raleigh, NC (1995) 231–235.
[608] JEAN, C., CHANTRON, J., HEINFLING, G., BISCH, P., "Realistic model of a double-walled containment vessel for the assessment of the areas to be repaired", Proc. 12th Int. Conf. on Nuclear Engineering, Arlington, VA, 2004, Vol. 2, ASME (2004) 381–387.
[609] BISHNOI, L., BASU, P., "Durability and ageing management of nuclear safety related concrete structures" (Proc. Int. Conf. on Maintenance and Durability of Concrete Structures, JNT University, 1997) Hyderabad (1997).
[610] ATOMIC ENERGY REGULATORY BOARD, Civil Engineering Structures Important to Safety of Nuclear Facilities, AERB Safety Standard No. AERB/SS/CSE, AERB, Mumbai (1998).
[611] ATOMIC ENERGY REGULATORY BOARD, Renewal of Authorisation for Operation of Nuclear Power Plants, AERB/SG/O-12, AERB, Mumbai (2000).
[612] ATOMIC ENERGY REGULATORY BOARD, Life Management of Nuclear Power Plants, AERB/SG/O-14, AERB, Mumbai (2005).
[613] ATOMIC ENERGY REGULATORY BOARD, Design of Pressurised Heavy Water Reactor Based Nuclear Power Plants, AERB/NPP-PHWR/SC/D (Rev. 1), AERB, Mumbai (2009).
[614] ATOMIC ENERGY REGULATORY BOARD, Design of Concrete Structures important to Safety of Nuclear Facilities, AERB/SS/CSE-1, AERB, Mumbai (2001).
[615] ATOMIC ENERGY REGULATORY BOARD, Quality Assurance during Construction of Civil Engineering Structures Important to Safety of Nuclear Facilities, AERB/SM/CSE-2, AERB, Mumbai (2011).
[616] BASU, P.C., GUPCUP, V.N., Safety Evaluation of rehabilitation of delaminated containment dome" Nuclear Engineering and Design 228 (2004) 195–205.
[617] BASU, P.C., MITTAL, A., High Performance Concrete for Nuclear Power Plants, SMiRT-15, Transactions of 15th Int. Conf. on Structural Mechanics in Reactor Technology, Seoul, paper No. H07/7, SMiRT, Raleigh, NC (1999).
[618] MITTAL, A., KAMATH, R.R., Properties of HPC for PC dome of NPP, Kaiga, Indian Concr. J., 73 9 (1999) 561–568.
[619] ATOMIC ENERGY REGULATORY BOARD, Nuclear Power Plant Operation, AERB/NPP/SC/O (Rev. 1), AERB, Mumbai (2008).
[620] ATOMIC ENERGY REGULATORY BOARD, Maintenance of Civil Engineering Structure Important to Safety of Nuclear Power Plants, AERB/SM/CSE-1, AERB, Mumbai (2002).
[621] ATOMIC ENERGY REGULATORY BOARD, In-Service Inspection of Civil Engineering Structures Important to Safety of Nuclear Power Plants, AERB/SM/CSE-2, AERB, Mumbai (2004).
[622] MINISTRY OF INTERNATIONAL TRADE AND INDUSTRY, Basic Concept for Nuclear Power Station Ageing Management, Japanese Government, Tokyo (1996) (in Japanese).
[623] JAPAN NUCLEAR ENERGY SAFETY ORGANIZATION, Review Manual for Age-related Technical Assessment Reduction in Strength and Reduction in Radiation Shielding Capability of Concrete (2009), https://translate.googleusercontent.com/translate_c?depth=1&hl=en&ie=UTF8&prev=_t&rurl=translate.google.com&sl=auto&tl=en&u=http://www.nsr.go.jp/archive/jnes/plec/doc/VLM/ss0512-04.pdf&usg=ALkJrhhdervxFIqgy_b82ig1VQqeSWQeVA
[624] ZWICKY, P., KLUGE, D., Aging management of safety related concrete structures in Switzerland, SMiRT-12, Transactions of the 12th Int. Conf. on Structural Mechanics in Reactor Technology, Paper No. DH06/1, Stuttgart, 1993, Elsevier, Amsterdam (1993).

[625] HSK, BASLER & HOFMANN, Alterung der Baustrukturen von Kernkraftwerken, Technical Report B 1712-2, internal report, Zurich (1992).
[626] UK DEPARTMENT OF ENERGY & CLIMATE CHANGE, Electricity Generation and Supply Figures for Scotland, Wales, Northern Ireland and England, 2008 to 2011, Department of Energy & Climate Change, London (2012).
[627] GOVERNMENT OF UNITED KINGDOM, UK Nuclear Installations Act 1965, Government of United Kingdom, London (1965).
[628] UK OFFICE FOR NUCLEAR REGULATION, Licensing Nuclear Installations, United Kingdom Office for Nuclear Regulation, Bootle (2012).
[629] BRADFORD, P., McNAIR, I., "A Regulatory View of Nuclear Containment on UK Licensed Sites", Int. Conf. Nucl. Cont., Cambridge (1996).
[630] SMITTON, C., Magnox 30 Years On, Institution of Mechanical Engineers Evening Discussion Meeting on Past Operation of the Nuclear Electric plc Power Stations, London (1992).
[631] McFARLANE, J., SMITH, L., DAVIES, D., McCLUSKEY, D., "In-service monitoring of AGR and PWR nuclear safety related structures", Int. Conf. Nucl. Cont., Cambridge (1996).
[632] UK OFFICE FOR NUCLEAR REGULATION, Safety Assessment Principles for Nuclear Facilities, Revision 1, United Kingdom Office for Nuclear Regulation, Bootle (2008).
[633] UK OFFICE FOR NUCLEAR REGULATION, Technical Assessment Guides, United Kingdom Office for Nuclear Regulation, Bootle (2013).
[634] THE AMERICAN SOCIETY OF MECHANICAL ENGINEERS, ASME Boiler and Pressure Vessel Code, ASME, New York (2010).
[635] NUCLEAR REGULATORY COMMISSION, Masonry Wall Design, Inspection Enforcement Bulletin No. 80-11, NRC, Washington, DC (1980).
[636] NUCLEAR REGULATORY COMMISSION, Inspection Notification 87-67: Lessons Learned from Regional Inspections of Licensee Actions in Response to IE Bulletin 80-11, NRC, Washington, DC (1987).
[637] AULUCK, R., Containment Degradation Issues at U.S. Nuclear Power Plants — Lessons Learned, SMiRT-21, paper No. 614 (Division 8), Transactions of 21st Int. Conf. on Structural Mechanics in Reactor Technology, New Delhi (2011).
[638] NUCLEAR REGULATORY COMMISSION, Reactor Site Criteria, 10 CFR 100, US Govt Printing Office, Washington, DC (2012).
[639] NUCLEAR REGULATORY COMMISSION, Assessment of Inservice Conditions of Safety-Related Nuclear Plant Structures, Rep. NUREG-1522, Office of Standards Development, Washington, DC (1995).
[640] de MARNEFEE, L., SEUNIER, D., Ageing of Concrete Containment Buildings, BELGATOM, Brussels, Belgium, Letter to D J Naus, Oak Ridge Natl Lab., TN (1997).
[641] OECD NUCLEAR ENERGY AGENCY, Repair Criteria & Methods of Repair for Concrete Structures on Nuclear Power Plants, NEA/CSNI/R(2002)7, Vol. 1, OECD-NEA Workshop on the Evaluation of Defects, OECD-NEA, Berlin (2002).
[642] INTERNATIONAL ATOMIC ENERGY AGENCY, Partial Flooding of the Plant, due to Sudden Clogging of the Outlet of the Cooling Tower, Induced By Concrete Beams Rupture, Incident Report 7531, 2002-11-01, Tihange-3, Belgium (2003), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=3303
[643] DEANS, J.J., CHAN, P., GOMME, R., Pickering NGS a Reactor Building 1 Dome Refurbishment Long-Term Monitoring of Coating (Seventh CNS Int. Conf. on CANDU Maintenance Toronto, Ontario, Canada, 2005) Toronto (2005).
[644] INTERNATIONAL ATOMIC ENERGY AGENCY, Vacuum Building Roof Seal Impairment, Incident Report 113 1981-09-18, Canada Bruce (1984), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=199
[645] DEMERS, M., POPOVIC, A., Neale, K. W., Rizkalla, S., Tadros, G., FRP Retrofit of the Ring-Beam of a Nuclear Reactor Containment Structure, SP-215-18, SP-215: Field Applications of FRP Reinforcement: Case Studies, Canada, American Concrete Institute, Farmington Hills, MI (2003).
[646] SENI, C., Overview of AECL Research Related to Reducing Leakage Through Concrete Containment Walls (IAEA Co-ordinated Research Programme Meeting in Vienna, 1993) Vienna (1993).
[647] INTERNATIONAL ATOMIC ENERGY AGENCY, Failure of the Prestressing Cable of the Containment Dome, Incident Report 7982, 2007-04-16, Temelin, Czech Republic (2007), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=10508
[648] NUCLEAR REGULATORY COMMISSION, DSIN Worried that Two New Detects may be Present in Many French Units, Inside NRC (1996).
[649] INTERNATIONAL ATOMIC ENERGY AGENCY, Incident Report 1404 , 1999-01-28, Generic, France (1999), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=2523
[650] INTERNATIONAL ATOMIC ENERGY AGENCY, Incident Report 1120, 1990-08-21, Generic Follow-up, France (1996), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=2442
[651] INTERNATIONAL ATOMIC ENERGY AGENCY, Incident Report 772, 1987-09-07, Gravelines-1, France (1999), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=1073

[652] INTERNATIONAL ATOMIC ENERGY AGENCY, Incident Report 7849: Potential Common Cause Vulnerabilities in Essential Service Water Systems: NRC Information Notice 2007-06 (2007), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=10120
[653] NUCLEAR REGULATORY COMMISSION, NRC Information Notice 2007-06: Potential Common Cause Vulnerabilities in Essential Service Water Systems, NRC, Washington, DC (2007).
[654] VALFORT, J.-L., Electricité de France, Villeurbanne, personal communication, 2013.
[655] INTERNATIONAL ATOMIC ENERGY AGENCY, Incident Report 1201, 1991-01-01 Obrigehim, Germany, Corrosion of Containment Shells in the Area of Transition to Concrete (1991), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=1562
[656] BISHNOI, L.R., Outline of Concrete Containment Structures of Indian NPPs, Technical Committee Meeting on Safety Aspects of Ageing Management for BWR Internals and Concrete Containment Buildings, IAEA, Vienna (1997).
[657] INTERNATIONAL ATOMIC ENERGY AGENCY, Incident Report 7285 , 1998-12-11, Ulchin1, Republic of Korea (1998), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=3021
[658] INTERNATIONAL ATOMIC ENERGY AGENCY, Ageing Related Degradation of Cooling Water Lines, Incident Report 8320, http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=10976
[659] INTERNATIONAL ATOMIC ENERGY AGENCY, Circumferential Break of Essential Service Water (ESW) Pipe at Vandellós 2 NPP While Operating at Rated Power, Incident Report 7663, http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=10012
[660] INTERNATIONAL ATOMIC ENERGY AGENCY, Barsebeck 2, Sweden 1993-10-15, Corrosion in the Containment Liner, Incident Report 1390 (1993), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=2535
[661] INTERNATIONAL ATOMIC ENERGY AGENCY, Ringhals, Sweden, Water Leakage from Containment Due to Degradation of the Metal Liner in Ringhals 2 NPP, Incident Report 7765, 2004-07-16 (2004), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=3560
[662] HEEP, W., Letter from Mr. W. Heep (Nordostschwezerische Kraftwerke) to Mr D J Naus (Oak Ridge National Laboratory) (1996).
[663] NUCLEAR REGULATORY COMMISSION, NRC Information Notice No. 2010-12 Containment Liner Corrosion, NRC, Washington, DC (2010).
[664] NUCLEAR REGULATORY COMMISSION, NRC Information Notice No. 85-10: Post Tensioned Containment Tendon Anchor Head Failure, NRC, Washington, DC (1985).
[665] NUCLEAR REGULATORY COMMISSION, NRC Information Notice No. 2003-08 Potential Flooding Through Unsealed Concrete Floor Cracks, NRC, Washington, DC (2003).
[666] NUCLEAR REGULATORY COMMISSION, Safety Evaluation Report With Open Items Related to the License Renewal of Crystal River Unit 3 Nuclear Plant, Docket 50-302, Florida Power Corporation, NRC, Washington, DC (2010).
[667] NUCLEAR REGULATORY COMMISSION, Special Inspection Report 05000302/2009007 Crystal River Containment Building, NRC, Washington, DC (2010).
[668] NUCLEAR REGULATORY COMMISSION, Davis-Besse Nuclear Power Station: Inspection to Evaluate the Root Cause and Corrective Actions for Cracking in the Reinforced Concrete Shield Building of the Containment System, Inspection Report IR 05000346/2012-009(DRS), NRC, Washington, DC (2012).
[669] NUCLEAR REGULATORY COMMISSION, NRC Information Notice No. 2013-04: Shield Building Concrete Subsurface Laminar Cracking Caused By Moisture Intrusion and Freezing, NRC, Washington, DC (2013).
[670] NUCLEAR REGULATORY COMMISSION, Resolution of Generic Safety Issues: Issue 202: Spent Fuel Pool Leakage Impacts, Rep. NUREG-0933, Main Report with Supplements 1–34, NRC, Washington, DC (2011).
[671] NUCLEAR REGULATORY COMMISSION, Safety Evaluation Report, Related to the License Renewal of Indian Point Nuclear Generating Unit Nos. 2 and 3, Docket Nos 50-247 and 50-286, Entergy Nuclear Operations, NRC, Washington, DC (2009).
[672] NUCLEAR REGULATORY COMMISSION, NRC Information Notice No. 97-11: Cement Erosion from Containment Subfoundations at Nuclear Power Plants, NRC, Washington, DC (1997).
[673] INTERNATIONAL ATOMIC ENERGY AGENCY, Cement Erosion from Containment Sub-Foundations at Nuclear Power Plants, Incident Report 7124, NRC Information Notice 97-11, IAEA, Vienna (1998).
[674] NUCLEAR REGULATORY COMMISSION, NRC Information Notice No. 2004-05 Spent Fuel Pool Leakage to Onsite Groundwater, NRC, Washington, DC (2004).
[675] NUCLEAR REGULATORY COMMISSION, NRC News No. I-12-025: NRC Issues Confirmatory Action Letter on Commitments to Address Seabrook Nuclear Plant Concrete Degradation, NRC, Washington, DC (2012).
[676] SOUTHERN NUCLEAR OPERATING COMPANY, Southern Nuclear Operating Company Vogtle Electric Generating Plant Unit 3 Preliminary Amendment Request (PAR): Basemat Concrete/Rebar Details (PAR-12-007), Southern Nuclear Operating Company, Birmingham, AL (2012).
[677] PAVEY, R., Stronger Concrete Might Resolve Deficient Vogtle Rebar (2012), http://chronicle.augusta.com/news/metro/2012-08-06/stronger-concrete-might-resolve-deficient-vogtle-rebar

[678] IRVING, J., SMITH, J., EADIE, D.M., HORNBY, I.W., Experience of Inservice Surveillance and Monitoring of Prestressed Concrete Pressure Vessels for Nuclear Reactors (Proc. Int. Conf. on Experience in the Design and Construction and Operation of Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors, York, 1975) The Institution of Mechanical Engineers, London (1975).
[679] JONES, W.C., TAYLOR, S.J., Construction of the PCPVs at Hinkley Point B and Hunsterston B (Proc. Int. Conf. on Experience in the Design and Construction and Operation of Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors, York, 1975) The Institution of Mechanical Engineers, London (1975).
[680] BROWN, V., BLAND, A., The Operator's View of the First Seven Years of Service of the Concrete Pressure Vessels at Oldbury on-the-Severn Power Station (Proc. Int. Conf. on Experience in the Design and Construction and Operation of Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors, York, 1975) The Institution of Mechanical Engineers, London (1975).
[681] HOUGHTON-BROWN, A., DARTON, A.J., The Oldbury Vessels (Proc. Conf. on Prestressed Concrete Pressure Vessels), The Institution of Civil Engineers, London (1967).
[682] INTERNATIONAL ATOMIC ENERGY AGENCY, Incident Report 8461, Reactor Building Sump Liner Degradation (2014), http://irs.iaea.org/IncidentReport.aspx?ReportType=0&ReportNumber=11126
[683] STOWE, L., THORNTON, H.J., Engineering Condition Survey of Concrete in Service, Technical Rep. REMR-CS-1, US Army Corps of Engineers, Vicksburg, MS (1984).
[684] CANADIAN DAM ASSOCIATION, Dam Safety Guidelines 2007, CDA, Toronto (2007).
[685] WILLIAMS, M., Review of International Acceptance Criteria Used when Inspecting Concrete Structures, AEA-ESD-0167, AEA Technology Engineering Services, Risley, Warrington (1995).
[686] SMALL, R., "Offshore and marine structures", The Design Life of Structures, (SORNERVILLE, G. Ed.) Blackie and Son, London (1992) 94–104.
[687] BROWNE, R., DOYLE, V., PAPWORTH, F., Inspection of concrete offshore structures, J. Pet. Tech. 33 11 (1981) 2243–2251.
[688] AMERICAN CONCRETE INSTITUTE, Guide for the Design and Construction of Fixed Offshore Concrete Structures, Rep. ACI 357R-84, ACI, Detroit, MI (1984).
[689] SLETTEN, R., FJELD, S., ROLAND, B., "In-service inspection of North Sea Structures" (Proc. 9th Annual Offshore Technology Conference, Offshore Technology), American Inst. of Mining, Metallurgical and Petroleum Eng, Houston (1977).
[690] LIU, P., Damages to concrete structures in a marine environment, Mat. Struct. 24 4 (1991) 302–307.
[691] ROSTAM, S., "The European approach to design life", The Design Life of Structures, (SORNERVILLE, G., Ed.) Blackie and Son, London (1992) 132–143.
[692] JOHNSON, P.M., COUTURE, A., NICOLET, R., Commission of Inquiry into the Collapse of a Portion of the De La Concorde Overpass", Gouvernement du Québec, Montreal (2007).
[693] ORGANISATION FOR ECONOMIC CO-OPERATION AND DEVELOPMENT, Bridge Management, OECD, Washington, DC (1992).
[694] DEPARTMENT FOR TRANSPORT, Inspection and Repair of Concrete Highway Structures, BA35/90, Design Manual for Roads and Bridges, Vol. 3, Department for Transport, London (1990).
[695] DEPARTMENT FOR TRANSPORT, Trunk Road and Motorway Structures — Records and Inspection, TRMM 2/88, Trunk Road Maintenance and Management Notice, Department for Transport, London (1988).
[696] WS ATKINS CONSULTANTS, Bridge Conditions Indicators, Volume 1, Commission Report, Issue 3, CSS Bridges Group, Lincoln (2002).
[697] WS ATKINS CONSULTANTS, Bridge Conditions Indicators, Volume 2, Guidance Note on Bridge Inspection Reporting, CSS Bridges Group, Lincoln (2002).
[698] WS ATKINS CONSULTANTS, Bridge Conditions Indicators, Volume 3, Guidance Note on Evaluation of Bridge Condition Indicators, CSS Bridges Group, Lincoln (2002).
[699] WS ATKINS CONSULTANTS, Bridge Conditions Indicators Project: Structures Condition Survey of Borough Principal Road Network, Six Year Report — April 03 to March 09, London Bridges Engineering Group, Lincoln (2010).
[700] SABNIS, G., SOROKO, R., DOSHI, K., Numerical System for Rating of Load Carrying Members, Concr. Int. 12 1 (1990) 63–65.
[701] SHIROLE, A., WINKLER, W., HILL, J., Bridge Management Systems — State of the Art, Transportation Research Records No. 1290, Transportation Research Board, Washington, DC (1991).
[702] GOLABI, K., THOMPSON, P., HYMAN, W., Pontis Technical Manual, A Network Optimization System for Bridge Improvements and Maintenance, Optima, Cambridge Systems, The Urban Institute, Washington, DC (1992).
[703] LAUER, K., "Comments on concrete damage classification system", Proc. 6th Meeting of RILEM Technical Committee 104-DCC, Haifa, 1992.
[704] STRATEGIC HIGHWAY RESEARCH PROGRAM, Distress Identification Manual for Long-Term Pavement Performance Studies, Rep. No. SHRP-LTPP/FR-90-001, National Research Council, Washington, DC (1990).
[705] STRATEGIC HIGHWAY RESEARCH PROGRAM, The Handbook for the Identification of Alkalii-Silica Reactivity in Highway Structures, Rep. No. SHRP-C/FR-91-101, National Research Council, Washington, DC (1991).

[706] AMERICAN SOCIETY OF CIVIL ENGINEERS, Instrumentation system maps nation's highways, Civil Eng. 61 8 (1991).
[707] HUNGARIAN ROAD SOCIETY, Maintenance Technologies of PCC Pavements, Road Instruction No. ÚT 2-2.125:2007, Hungarian Road Society, Budapest (2007).
[708] W.D.M., SCRIM/SCRIMTEX, W.D.M. Limited, http://www.wdm.co.uk/equipment/3-equipment/3-scrimr-scrimtex
[709] ROE, P.G., SINHAL, R., Recent Developments to the SCRIM Measurement Technique in the UK, Paper No. PA/INF/4521/05, TRL Limited, Wokingham (2005).
[710] DEPARTMENT FOR TRANSPORT, Design Manual for Roads and Bridges, Volume 7, Section 3, Department for Transport, London (2008).
[711] DIXON, J., "Railway structures", Durability of Concrete Structures, (MAYS, G. Ed.) E.&F.N. Spon, London (1992).
[712] SMITH, B., "An application of structural monitoring", Proc. Structural Congress of ASCE, Orlando, FL, 1987, American Society of Civil Engineers, New York (1987).
[713] GERWICK, B., SUDHAKAR, A., YAO, S., OVADIA, D., BEMER, D., "Repair of corrosion damages and protection of cooling water intake/circulation structures", Extending the Lifespan of Structures, IABSE Symposium, San Francisco, 1995, IABSE Rep. No. 73, IABSE, Zurich (1995) 481–486.
[714] JÓZSEF, D.A., OROSZ, D.Á., Repairing of a group of 100 m tall ventilation chimneys, Vasbetonépítés 2000 1 (2000) 3–12.

ABBREVIATIONS

AASHTO	American Association of State and Highway Transportation Officials
AAR	alkali-aggregate reaction
ABWR	advanced boiling water reactor
AC	alternating current
ACI	American Concrete Institute
ACR	Advanced CANDU Reactor
AECL	Atomic Energy of Canada Limited
AERB	Atomic Energy Regulatory Board (of India)
AFCR	advanced fuel CANDU reactor
AFNOR	Association Française de Normalisation
AGR	Advanced Gas Cooled Reactor
AMP	ageing management plan/programme
ANSI	American National Standards Institute
ARDM	ageing related degradation mechanism
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials (former name; now known as ASTM International)
BS	British Standard
BWR	boiling water reactor
CA	condition assessment
CANDU	Canada deuterium-uranium
CCB	concrete containment building
CCS	concrete containment structure
CEN	Comité Européen de Normalisation (European Committee for Standardization)
CFR	Code of Federal Regulations (USA)
CNSC	Canadian Nuclear Safety Commission
COG	CANDU Owners Group
CSA	Canadian Standards Association
CSN	Czech Standards Institute
DIN	Deutsches Institut für Normung e.V. (German Institute for Standardization)
EC6	Enhanced CANDU 6
EDF	Electricité de France
EN	Européenne Norme (European standard)
EN-ISO	ISO standard written for European Union (regional standard)
EPR	European Pressurized Reactor/Evolutionary Power Reactor
EPRI	Electric Power Research Institute
ESBWR	economic simplified boiling water reactor
ETC-C	Code Technique pour les Travaux de génie Civil EPR (EPR Technical Code for Civil Works)
FSTM	Florida State Test Method
GPR	ground penetrating radar
HSK	Hauptabteilung für die Sicherheit der Kernanlagen (former Swiss Federal Nuclear Safety Inspectorate)
I&C	instrumentation and control
IGALL	International Generic Ageing Lessons Learned
INPO	Institute of Nuclear Power Operators
ISO	International Organization for Standardization
JNES	Japan Nuclear Energy Safety Organization
JSCE	Japan Society of Civil Engineers
KTA	Kerntechnischer Ausschuss (German Nuclear Safety Standards Commission)

LOCA	loss of coolant accident
LTSR	long term safety review
MR&R	maintenance, repair and rehabilitation
NBI	national bridge inventory (USA)
NDT	non-destructive testing
NISA	Nuclear and Industrial Safety Agency (Japan)
NKe	Normenausschuss Kerntechnik (German Nuclear Standards Committee)
NPP	nuclear power plant
NRA	Nuclear Regulatory Authority (Japan)
OECD	Organisation for Economic Co-operation and Development
OPEX	operating experience
OPG	Ontario Power Generation
ORNL	Oak Ridge National Laboratory
PCCP	prestressed concrete cylindrical pipe
PHWR	pressurized heavy water reactor
RB	reactor building
RCC-G	Règles de Conception et de Construction du Génie Civil (French standard)
RCCV	reinforced concrete containment vessel
Rebar	reinforcement bar, steel
RSK	Reaktor-Sicherheitskommission (German Reactor Safety Commission)
SCRIM	Sideway-force Coefficient Routine Investigation Machine
SD	secure digital (memory card)
SFP	spent fuel pool
SHRP	Strategic Highway Research Program
SIA	Swiss Society of Engineers and Architects
SI&A	structural integrity and appraisal
SLMS	Structural Life Management System
SMIC	Structural Materials Information Center (USA)
SSC	systems, structures and components
TLAA	time limited ageing analysis
USNRC	United States Nuclear Regulatory Commission
WANO	World Association of Nuclear Operators
WWER	water cooled water moderated power reactor

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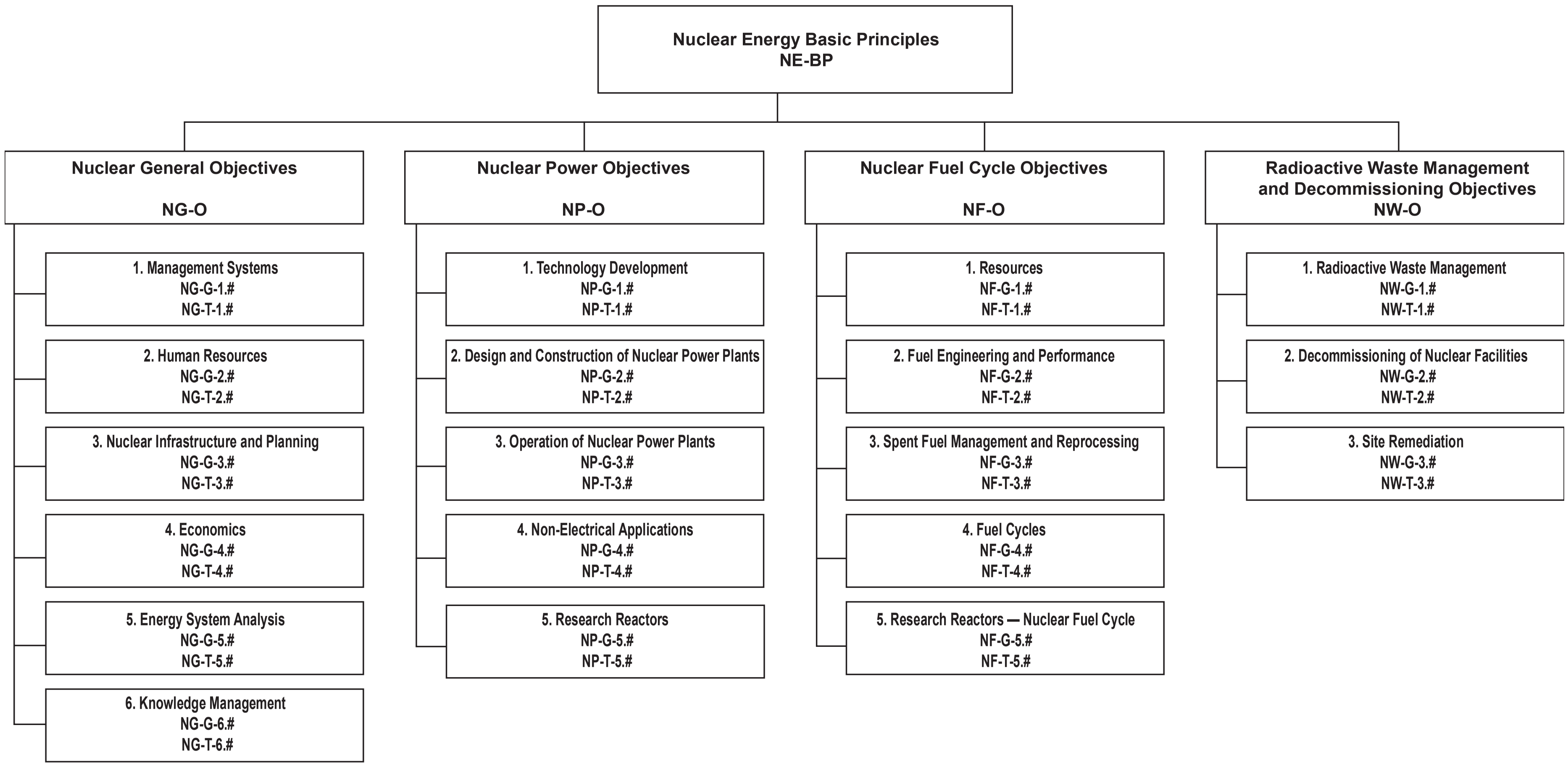

Key

BP: Basic Principles
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ISSN 1995–7807