IAEA-TECDOC-1025

Assessment and management of ageing of major nuclear power plant components important to safety:

Concrete containment buildings

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FOREWORD

At present, there are over four hundred operational nuclear power plants (NPPs) in IAEA Member States. Operating experience has shown that ineffective control of the ageing degradation of the major NPP components (e.g. caused by unanticipated phenomena and by operating, maintenance, design or manufacturing errors), can jeopardize plant safety and also plant life. Ageing in NPPs must be, therefore, effectively managed to ensure the availability of design functions throughout the plant service life. From the safety perspective, this means controlling within acceptable limits the ageing degradation and wearout of plant components important to safety so that adequate safety margins remain, i.e. integrity and functional capability in excess of normal operating requirements.

This TECDOC is one in a series of reports on the assessment and management of ageing of the major NPP components important to safety. The current practices for the assessment of safety margins (fitness-for-service) and the inspection, monitoring and mitigation of ageing degradation of selected components of Canada deuterium–uranium (CANDU) reactor, boiling water reactor (BWR), pressurized water reactor (PWR), and water moderated, water cooled energy reactor (WWER) plants are documented in the reports. These practices are intended to help all involved directly and indirectly in ensuring the safe operation of NPPs; and also to provide a common technical basis for dialogue between plant operators and regulators when dealing with age-related licensing issues. Since the reports are written from a safety perspective, they do not address life or life-cycle management of the plant components, which involves the integration of ageing management and economic planning. The target audience of the reports consists of technical experts from NPPs and from regulatory, plant design, manufacturing and technical support organizations dealing with specific plant components addressed in the reports.

The component addressed in the present report is concrete containment buildings of nuclear power plants. The report presents results of a Co-ordinated Research Programme (CRP) on Management of Ageing of Concrete Containment Buildings. It is based on a combination of a worldwide survey of NPP owners/operators, knowledge of practice in Member States of the CRP participants, and literature reviews. The report was finalized using a feedback from a May 1997 Technical Committee meeting, which was attended by technical experts from nine Member States.

The contributors to the drafting and review of this TECDOC are identified at the end of this publication. Their work is greatly appreciated. In particular, the contribution of D.J. Naus of the USA is acknowledged. The IAEA officer who directed the preparation of the report was J. Pachner of the Division of Nuclear Installation Safety.

EDITORIAL NOTE

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1. INTRODUCTION

1.1. OBJECTIVE

The objective of this report is to present the results of the Co-ordinated Research Programme (CRP) on the Management of Ageing of Concrete Containment Buildings (CCBs) addressing current practices and techniques for assessing fitness-for-service and the inspection, monitoring and mitigation of ageing degradation of CCBs. This report aims to provide a technical basis for developing and implementing a systematic ageing management programme and also for dialogue between nuclear power plant (NPP) operators and regulators when dealing with age-related licensing issues.

1.2. BACKGROUND

1.2.1. Safety aspects of concrete containment building (CCB)

Concrete containment structures are designed to separate the reactor and other systems and equipment important to safety from the outside environment. External and internal events are considered in the design. 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. The containment is designed to withstand the loadings resulting from the above postulated events and represents the final physical barrier to radioactive material before release to the outside environment. Potential age-related degradation of a concrete containment building, including its various subcomponents, must be effectively controlled to ensure its required leaktightness and structural integrity.

1.2.2. Need for systematic ageing management of CCB

The design life of existing NPPs was often chosen to be 30-40 years. However, current economic pressures on utilities to extend plant service life (60 years total being a quoted target) and decommissioning strategies that involve use of the containment as a "safestore" for periods of up to 100 years, mean that the containment buildings may have to perform safety functions for a time period significantly greater than their initial design life.

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 CCBs, 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, could impair its safety functions and thus increase the risk to public health and safety. Effective ageing management[1] of the CCB is therefore required to ensure its fitness-for-service throughout the plant service life.

A number of existing NPP programmes, such as periodic inspection and testing, and preventive and corrective maintenance, contribute to the ageing management of CCBs. The effectiveness and efficiency of CCB ageing management can be improved by integrating and

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[1]In the Electric Power Research Institute document *Common Ageing Terminology* (1993) ageing management is defined as the engineering, operations and maintenance actions to control within acceptable limits ageing degradation and wear out of systems, structures or components.

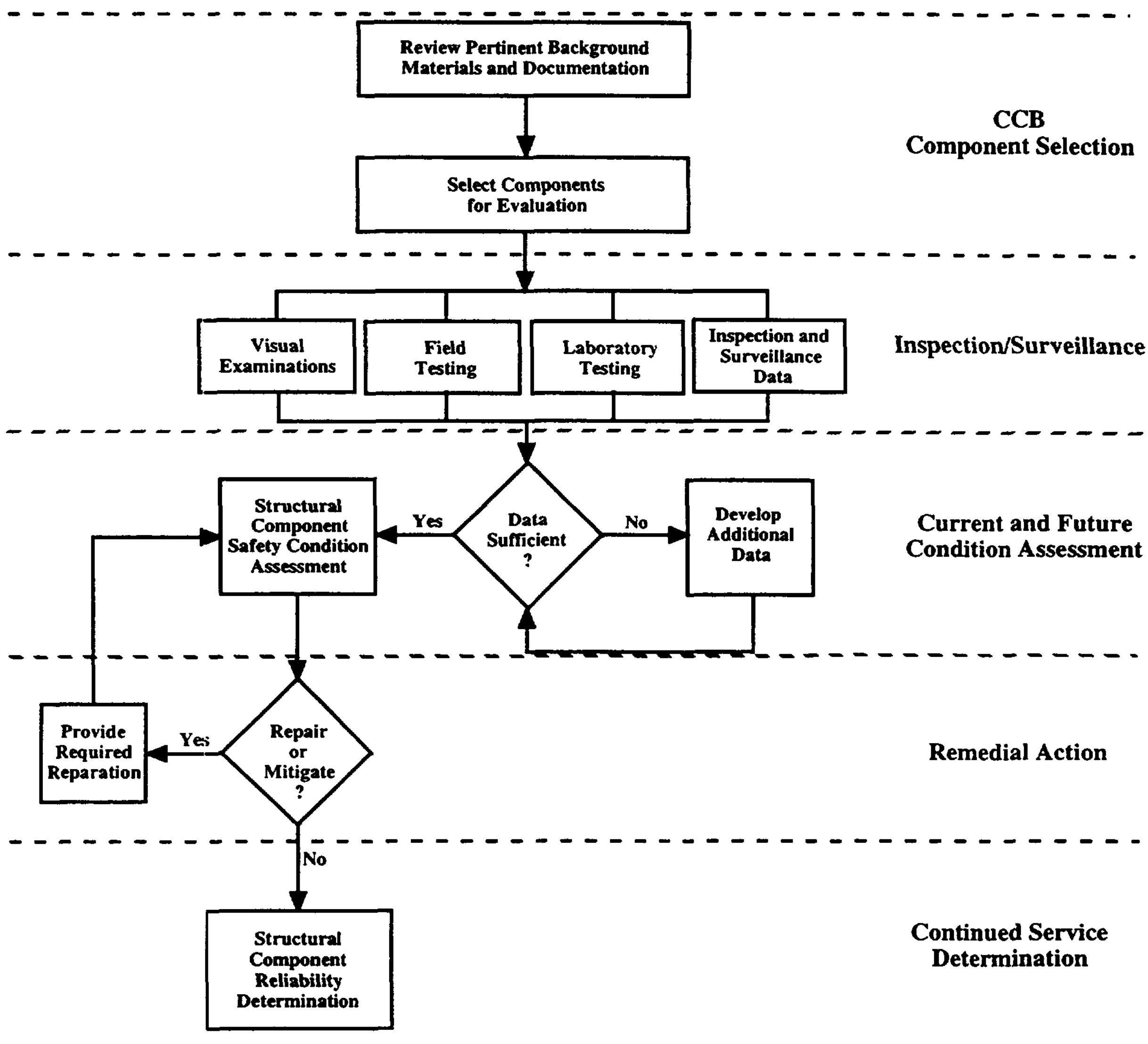


FIG. 1.1. Example of programme for evaluation of current condition and estimation of future performance of concrete containment buildings.

modifying these programmes, as appropriate, within a systematic ageing management programme (AMP), Figure 1.1 presents one approach as to how these various programmes would integrate into a programme for evaluation of the current condition and estimation of the future performance of CCBs. Development of a systematic ageing management programme and the interaction of key elements of such a programme are discussed later in this report.

1.2.3. IAEA programme on safety aspects of NPP ageing

To assist Member States in understanding ageing of systems, structures and components (SSCs) important to safety and in effective ageing management of these SSCs, the IAEA in 1989 initiated a project on safety aspects of NPP ageing. This project integrates information on the evaluation and management of safety aspects of NPP ageing generated by Member States into a common knowledge base, and derives guidance and assists Member States in the application of this guidance. Results of this work are documented in Refs [1.1-1.7].

The Safety Practices publication *Data Collection and Record Keeping for the Management of Nuclear Power Plant Ageing* [1.2] provides information on data requirements and a system for data collection and record keeping. General data needs, which are grouped into three categories (baseline data, operating history data, and maintenance history data), are illustrated by several examples of component-specific data requirements. Actual record keeping systems, including an advanced system, are also described in the publication.

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 for the management of ageing of NPP components important to safety is documented in Ref. [1.3] and consists of three basic steps:

(1) Selecting, from the safety perspective, plant components in which ageing should be evaluated, by assessing the effects of ageing degradation on the ability of the components to perform their design functions and crediting existing programmes and activities that manage ageing effectively.

(2) Performing ageing management studies for the selected components to determine appropriate ageing management actions. The two-phased method reviews current understanding, monitoring and mitigation of the components' ageing, and identifies or develops effective and practical technology and practices for its monitoring and mitigation.

(3) Using results of the ageing management studies to take appropriate management actions (i.e. improving existing operations and maintenance practices and the design) and to improve relevant codes, standards and regulatory requirements.

Development of the above consensus guidance is beneficial in its own right because it provides opportunities for practitioners from organizations with both direct and indirect responsibility for nuclear safety to address in an interactive environment important issues of common interest and to learn from each other. However, it is the actual application of guidance that has a positive impact on nuclear safety. The IAEA therefore devotes significant effort in assisting Member States in the application of its guidance.

To assist Member States in the application of the above ageing management methodology, the IAEA in 1989 initiated pilot studies on management of ageing of nuclear power plant components, with four safety significant components selected for the studies [1.4]. These four components represented different safety functions and materials susceptible to different types of ageing degradation. The components selected were:

- the primary nozzle of a reactor pressure vessel;
- motor operated valves;
- concrete containment building; and
- 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 co-ordinated research programmes were set up for each of the above four components to implement the Phase 2 pilot studies. The overall objective of each pilot study was to identify the dominant ageing mechanisms and to develop an effective strategy for managing ageing effects caused by these mechanisms.

1.2.4. Implementation of CRP on management of ageing of CCBs

The original objectives for the Co-ordinated Research Programme on CCBs were to:

(1) produce a summary of current ageing management practices and experiences for concrete containment structures;
(2) compile a state-of-the-art report on concrete repair techniques and materials specifically applicable to nuclear containment structures;
(3) develop crack mapping and acceptance/repair guidelines applicable to nuclear containment structures; and
(4) develop a set of practical condition indicators and associated guidelines for monitoring concrete containment ageing.

This CRP investigated CCBs with the underlying objective of making use of the research and engineering expertise of CRP participants to develop the technical basis for a practical ageing management programme for CCBs. The CRP work therefore focused on compiling and evaluating information on age-related degradation of CCBs, as well as 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.

The first step involved a literature survey and preparation of a questionnaire for a worldwide survey of NPP operators in support of the CRP objectives [1.5]. Responses were received representing over 150 NPP units. Information obtained was compiled into a database and then interpreted and evaluated in connection with preparation of the present report. This report primarily addresses CRP objectives (1) to (3). Although information relating to objective (4) is addressed in this report in general terms (e.g. importance of monitoring for concrete cracking, foundation settlement, and loss of prestressing forces), implicit guidelines on interpretation of results obtained is not provided. As a result of limited information obtained from the questionnaire on this topic, and the difficulty in establishing quantitative guidelines on interpretation of results due to the interaction of the form of degradation factor monitored (e.g. cracking), the importance of the structure affected, and the ambient environmental conditions, the CRP participants decided that this subject would best be addressed under a subsequent activity that includes participation of researchers, as well as NPP operators and regulators.

1.3. SCOPE

This report deals with CCBs that are included as part of a nuclear power plant. It addresses potential ageing mechanisms, age-related degradation, and ageing management (i.e. inspection, monitoring, assessment and remedial measures) for the following materials and components for concrete containment buildings:

- concrete;
- reinforcing steel;
- prestressing systems;
- penetrations;
- liner systems;
- waterstops, seals and gaskets; and
- protective coatings.

Structural steel piles and anchorages are also addressed to a limited extent.

The TECDOC does not address life or life-cycle management of CCBs because it is written from the safety perspective and life management includes economic planning.

1.4. STRUCTURE OF THE REPORT

A description of the various CCBs used in NPPs and an overview of the applicable design standards and regulatory requirements for CCBs are given in Chapter 2. Chapter 3 supports understanding of CCB ageing by providing information on the potential degradation mechanisms for each of the primary material systems, and identifying several areas in CCBs where concrete-related materials may experience degradation. Chapter 4 provides an overview of the various techniques that may be used to detect ageing degradation in concrete containments, and the advantages, limitations and primary application of each. Chapter 5 begins by considering the various options that are available to repair or mitigate unacceptable degradation. Details are given on the techniques that are typically used to repair specific forms of degradation, with reference to their application in NPPs. Chapter 6 summarizes operating experience related to performance, inspection, and repair of CCBs. Chapter 7 shows how the key elements of ageing management for CCBs are integrated within a systematic ageing management programme for CCBs. Finally, Chapter 8 summarizes the conclusions of the CRP and recommends follow-up activities that build on the results of this CRP.

REFERENCES TO CHAPTER 1

[1.1] INTERNATIONAL ATOMIC ENERGY AGENCY, Safety Aspects of Nuclear Power Plant Ageing, IAEA-TECDOC-540, IAEA, Vienna (1990).

[1.2] 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).

[1.3] INTERNATIONAL ATOMIC ENERGY AGENCY, Methodology for Ageing Management of Nuclear Power Plant Components Important to Safety, Technical Reports Series No. 338, Vienna (1992).

[1.4] INTERNATIONAL ATOMIC ENERGY AGENCY, Pilot Studies on Management of Ageing of Nuclear Power Plant Components — Results of Phase I, IAEA-TECDOC-670, Vienna (1992).

[1.5] Summary Results of the Survey on Concrete Containment Ageing, internal IAEA report, 1995.

[1.6] Specialists Meeting on the Effectiveness of Methods for the Detection and Monitoring of Age-Related Degradation in Nuclear Power Plants, internal IAEA report, 1996.

[1.7] INTERNATIONAL ATOMIC ENERGY AGENCY, Implementation and Review of Nuclear Power Plant Ageing Management Programme, IAEA, Vienna (in preparation).

2. CONCRETE CONTAINMENT BUILDINGS

2.1. DESCRIPTION

Although some of the early thermal reactors (e.g., gas-cooled and graphite-moderated light-water) were built without containments, current generation reactors are provided with a containment. The primary purpose of a nuclear power plant containment is to prevent the release of fission products to the environment in the case of a loss of coolant accident (LOCA) in the primary system and to provide a shield against radiation under normal operation (i.e., separate the reactor and other safety significant systems and equipment from the outside environment). In addition to being designed to withstand internal pressures and temperatures resulting from design basis events, the containments are designed to provide protection against severe external events (e.g., floods, tornadoes, and missile impact). Associated with the design specifications for these structures is a certain level of durability (e.g., minimum concrete cover requirements or maximum water-to-cement ratios for the concrete to protect the embedded steel reinforcement under different anticipated environmental conditions).

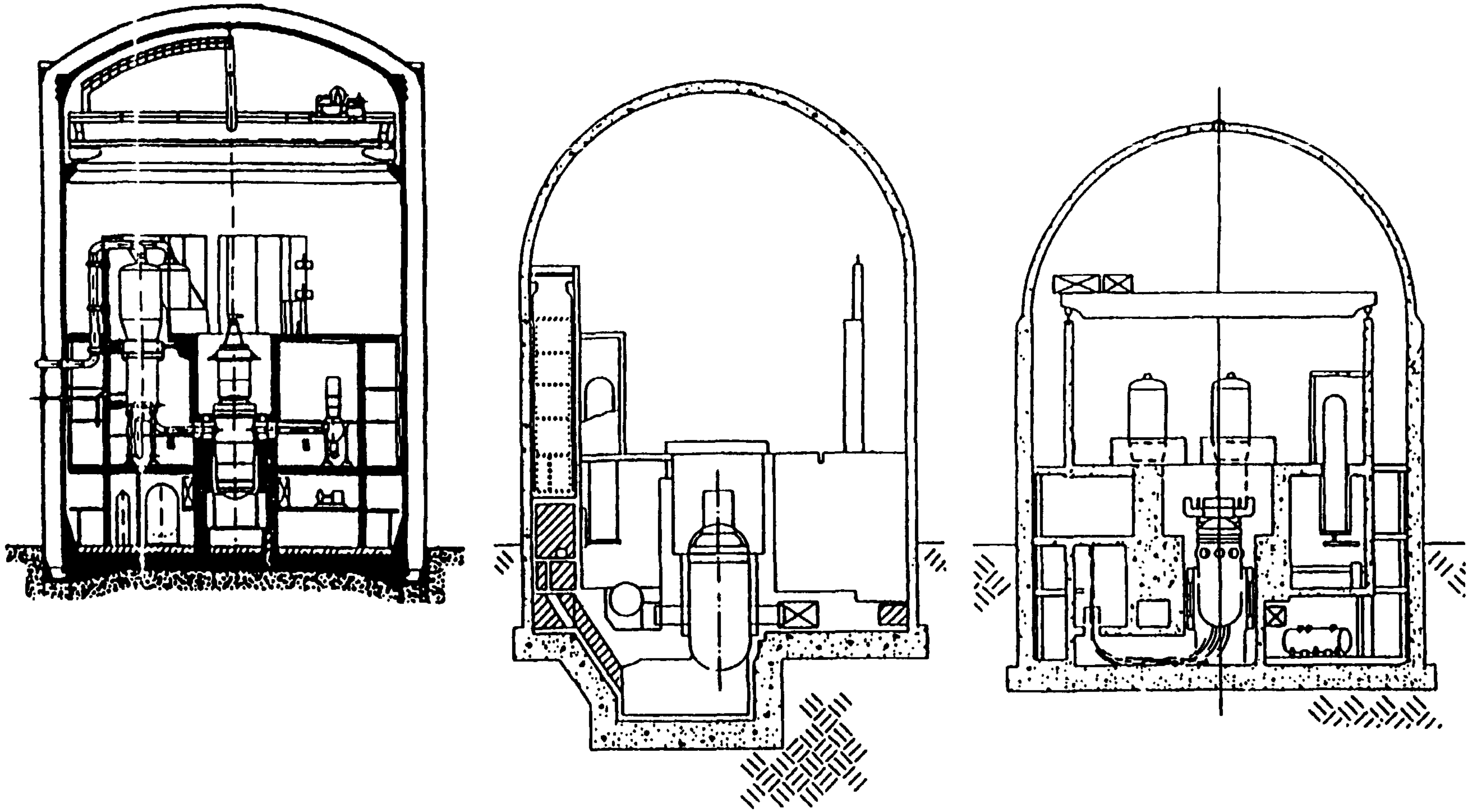
With respect to the containment function, a number of concepts exist depending on the type of reactor system utilized (e.g., pressurized water reactor, boiling water reactor, pressurized heavy water reactor, gas cooled reactor, and water water power reactor). Information on design of concrete containments and performance parameters (e.g., dimensions, capacity, volume, and design pressure) and service conditions is provided in Ref. [2.1]. More detailed information to that provided below is available in Refs [2.2] and [2.3].

2.1.1. Pressurized water reactors (PWRs)

Concrete containments for PWRs may be fabricated from reinforced concrete, prestressed concrete, or a combination of the two. They all enclose the entire primary circuit that includes the reactor pressure vessel, steam generators, and piping. Single- or double-wall construction may be used and the containment may or may not be lined to provide a leaktight barrier. Three general categories of PWR containments exist: large dry, ice condenser, and subatmospheric (Fig. 2.1). The large dry containment is designed to have a capacity to contain the energy of the entire volume of primary coolant fluid in the unlikely event of a LOCA. The ice condenser containments channel the steam resulting from a LOCA through ice beds to reduce the pressure buildup and thus the containment volume and pressure requirements. The subatmospheric containments are fabricated from reinforced concrete and are designed so that a slightly negative pressure is maintained in the containment to reduce the volume requirements. A secondary concrete containment is used as a shield when a steel primary containment is used.

2.1.2. Boiling water reactors (BWRs)

Although the majority of BWR plants utilize a steel containment vessel, a number of units utilize either a prestressed- or reinforced-concrete containment (Fig. 2.2). With only one exception, all BWR plants in the USA that utilize a steel containment have reinforced concrete structures that serve as secondary containments or reactor buildings and provide support and shielding functions for the primary containment. The BWRs are all designed with a pressure suppression system. The containments are divided into two main compartments — wet-well and dry-well. After a LOCA, the air and steam in the dry-well are forced through a number of downcombers to a pool in the wet-well, where the steam condenses. Water spray

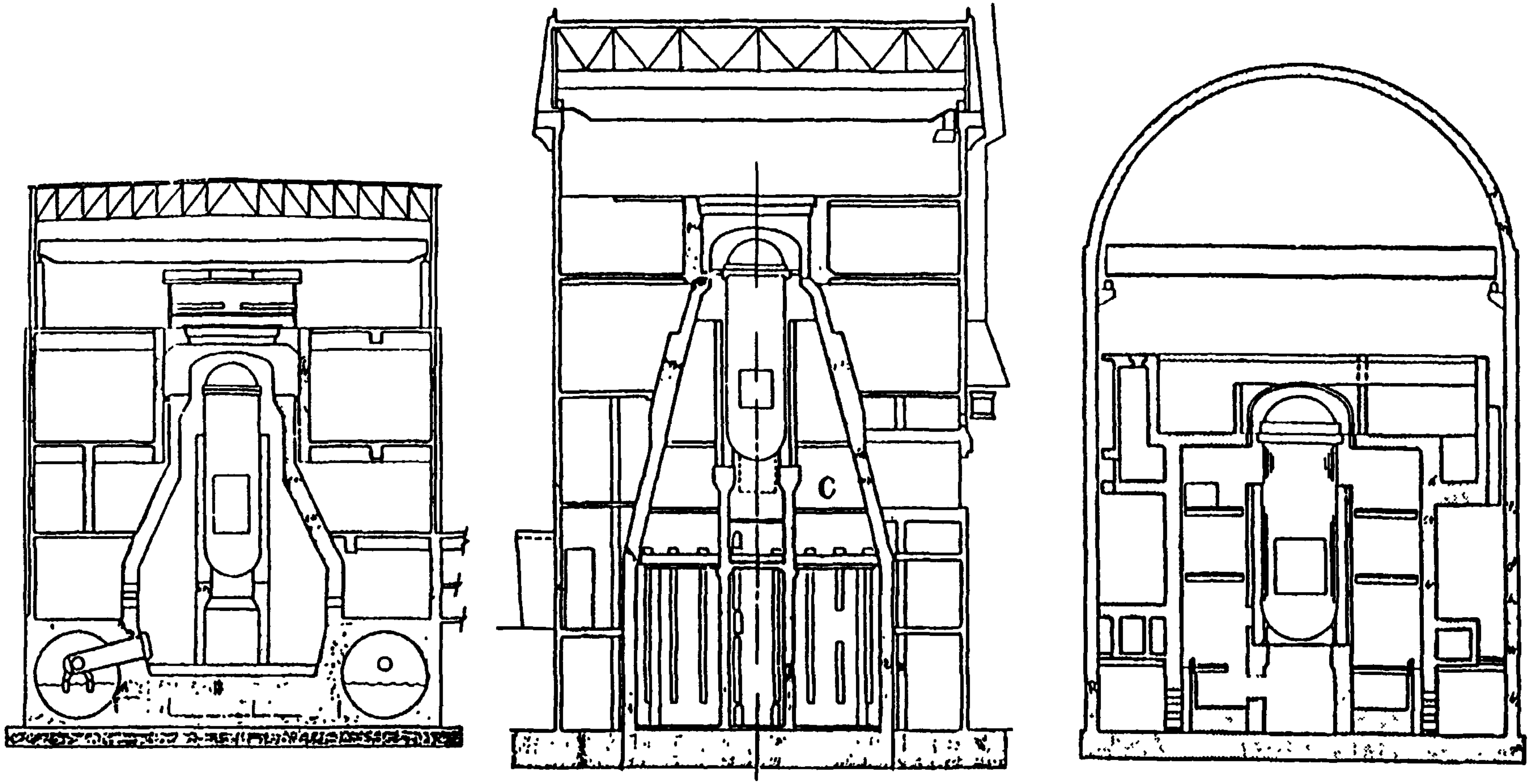


a. PWR large-dry double barrier

b. PWR ice-condenser

c. PWR sub-atmospheric

FIG. 2.1. Examples of PWR concrete containment cross-sections.



a. BWR Mark I

b. BWR Mark II

c. BWR Mark III

FIG. 2 2. Examples of BWR concrete containment cross-sections systems are provided to reduce the containment volume requirements, and the auxiliary systems are generally housed in the secondary containment.

2.1.3. Pressurized heavy water reactors (PHWRs)

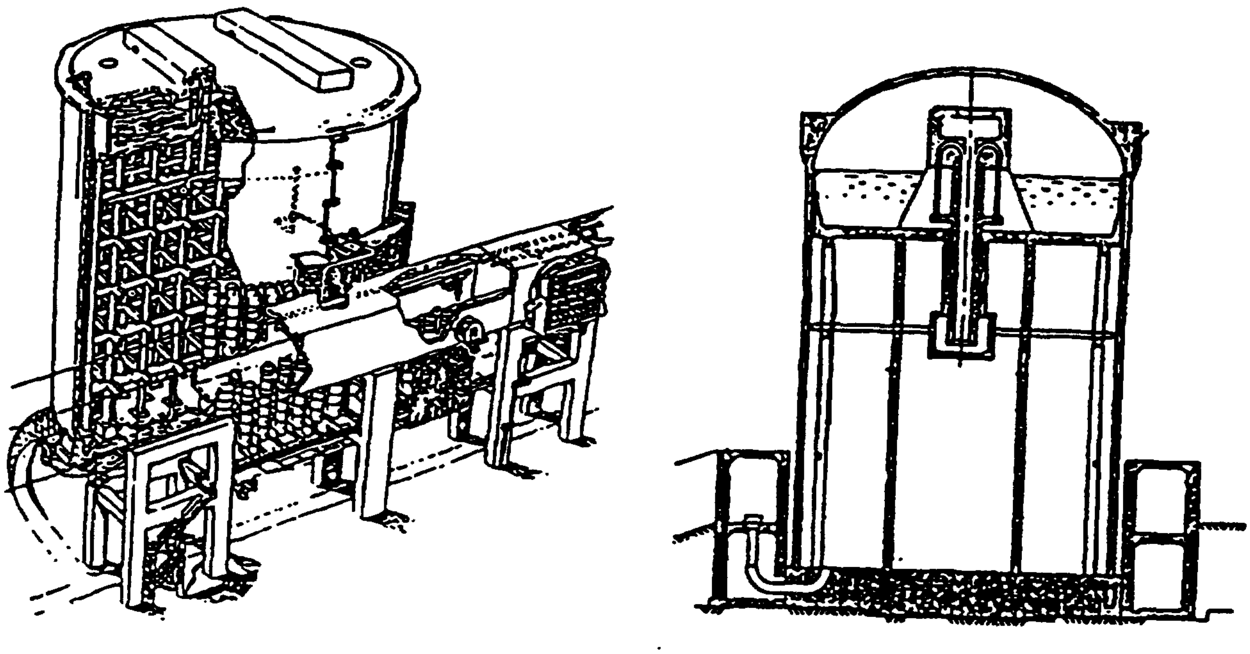
Of the heavy water reactor designs, that for the Canadian Deuterium Uranium (CANDU) reactors is the most prevalent. These plants are constructed in single or multi-unit configurations. CANDU multi unit stations employ a common vacuum building and pressure relief duct as the main components of the containment system. Connected to the pressure relief duct are up to eight reactor buildings that form the balance of the system. During normal operation, the vacuum building and pressure relief duct are isolated from the reactor buildings, however, with sufficient increase in pressure due to an accident in any of the reactor buildings, panels will rupture and hot gases and steam that caused the pressure rise will fill and pressurize the pressure relief duct. Pressure relief valves will then open to release the hot gases and steam into the vacuum building where a pressure-actuated water dousing system will condense the steam. Reactor buildings for CANDU multi unit stations come in two basic designs: (1) domed cylindrical structures with unlined single exterior wall, independent internal frame, and calandria vault; and (2) thick-walled cube shaped structures with steel liner and post-tensioned roof beams. Single unit reactor buildings are similar to design (1) above except they have post-tensioned wall construction, a double-walled dome to contain the dousing water reservoir (contained in the vacuum building in the multi unit configuration), and an organic liner. The vacuum building and pressure relief duct of the multi unit stations are reinforced concrete structures. A post-tensioned ring girder is used to stiffen the top of the vacuum building cylinder perimeter wall. The perimeter wall also provides shielding. Recent requirements to contain higher positive pressures in the newer vacuum building have resulted in some design changes. Most notable are the monolithic connection of the roof dome to the peripheral ring girder, circumferential and vertical prestressing of the wall and ring girder, prestressing of the roof dome, and lining the floor slab and lower six meters of the perimeter wall with carbon steel. Figure 2.3 provides examples of CANDU vacuum and reactor building cross-sections.

2.1.4. Gas cooled reactors (GCRs)

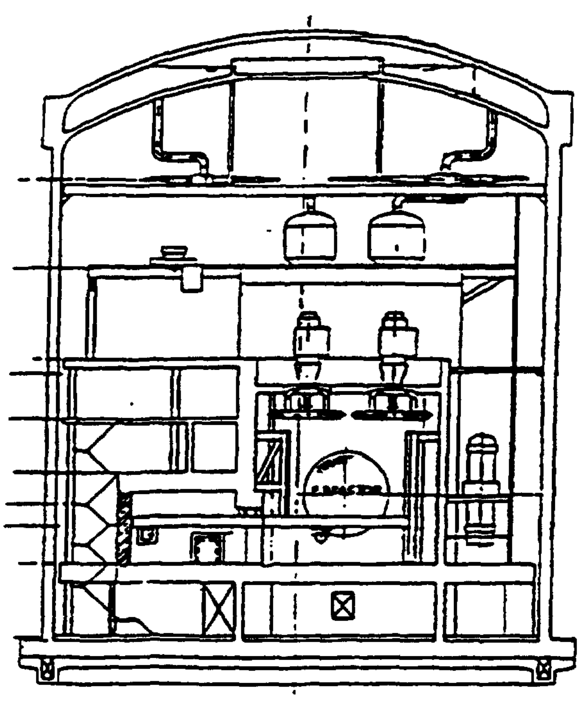
Primary containment for many gas-cooled reactors (e.g., Magnox Stations, Advanced Gas-Cooled Reactors, and the High-Temperature Gas Cooled Reactors) is provided by a prestressed concrete pressure vessel (PCPV) (Fig. 2.4). In most cases, the concept of an "integral design" has been adopted, with the pressure circuit, reactor core, and boiler all contained within the PCPV. The principal structural functions of the PCPV are to support the reactor and boiler, and to provide support to the steel liner that creates the pressure enclosure for the gas circuit transferring heat from the reactor core to the boiler. The steel liner is provided with insulation, and cooled by recirculated water, thus also providing a thermal barrier between the hot gas and the concrete surface. Mass concrete temperatures are generally kept below 50°C. with some isolated hotter areas existing. Although the PCPV also provides biological shielding, thicknesses of the vessel wall and end caps (typically 4-5 m) are determined by the primary function of support for the pressure boundary.

2.1.5. Water-water power reactors (WWERs)

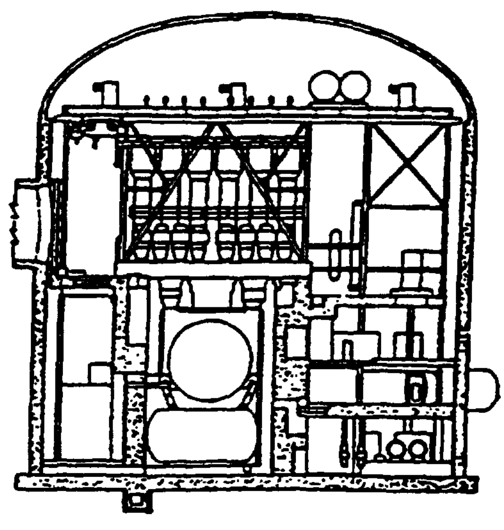
The water-water power reactors are basically PWRs of Soviet design (Fig. 2.5). The WWERs all feature some level of containment as a provision against release of radioactivity



a. Vacuum buildings



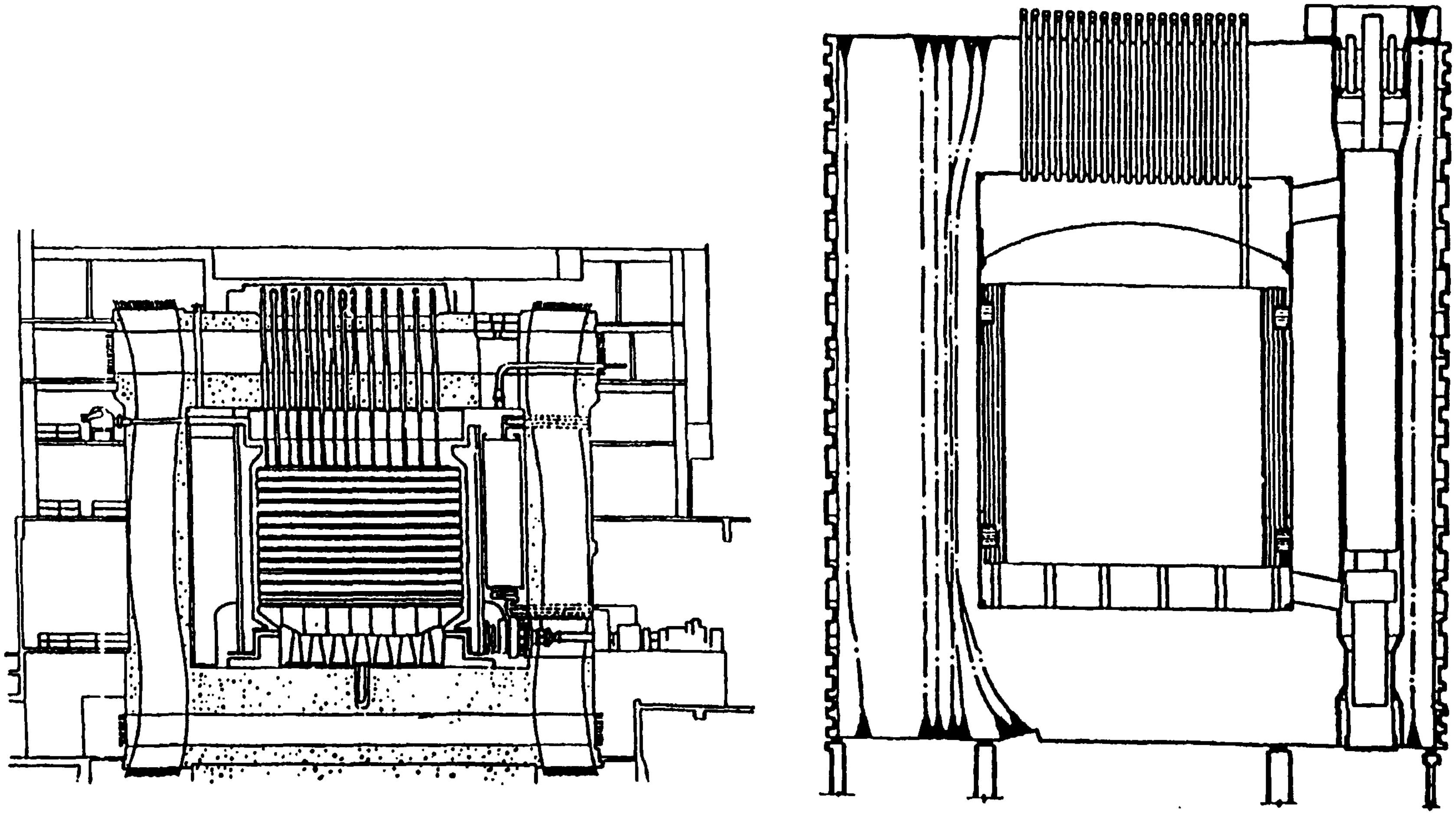
Single Unit



Multi-Unit

b. Reactor building

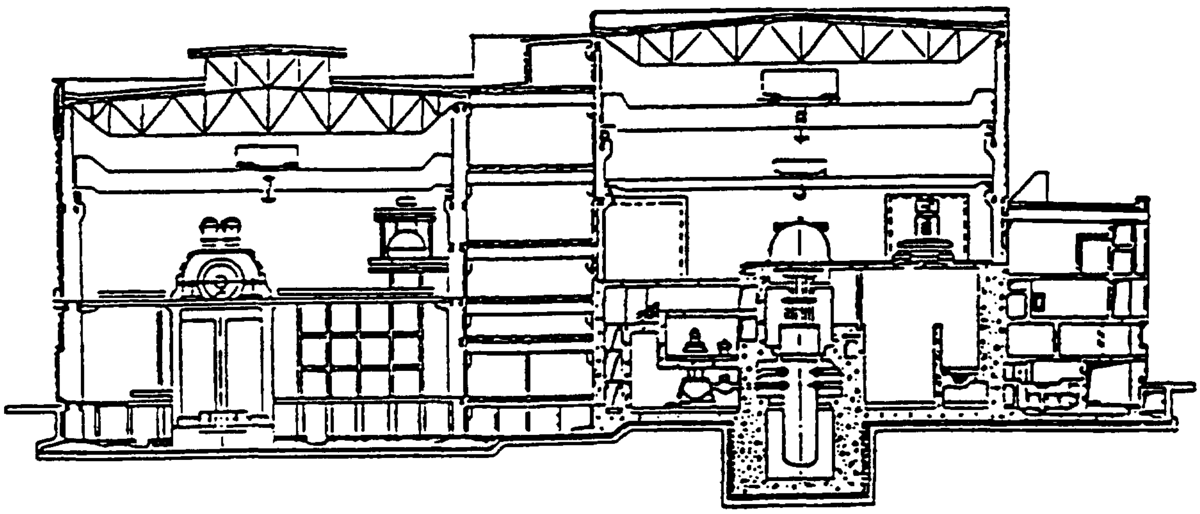
FIG. 2.3. Examples of CANDU vacuum and reactor building cross-sections.



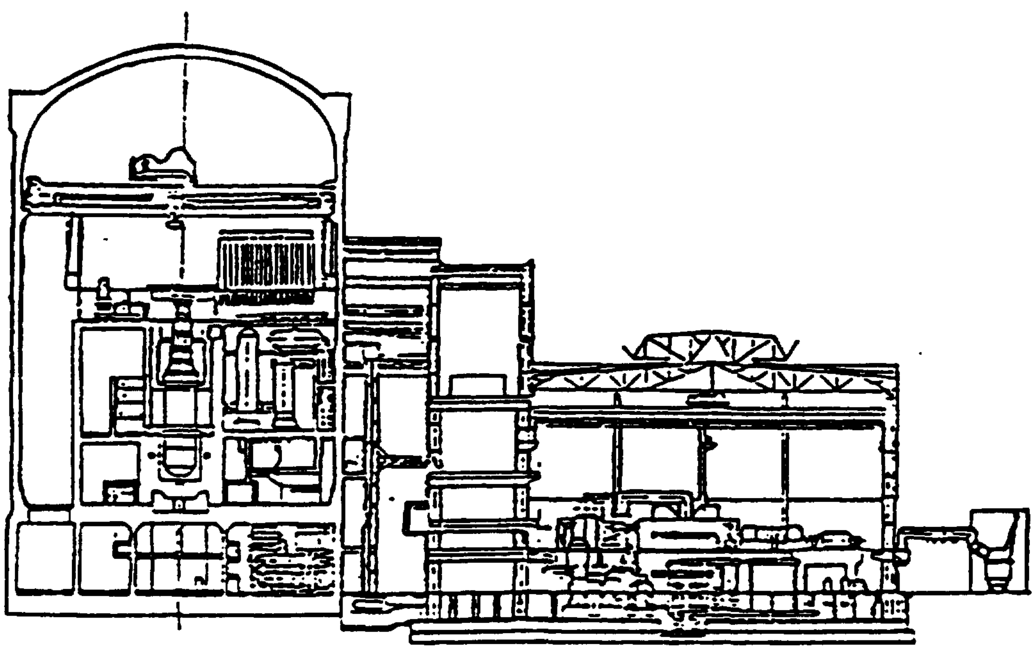
a. Gas-Cooled
(Magnox)

b. Advanced Gas-Cooled

FIG. 2.4. Examples of gas cooled reactor prestressed concrete pressure vessel sections.



a. VVER-440, Version V-230



b. VVER-1000, Version V-187

FIG. 2.5. Examples of water–water power reactor cross-sections.

following the design basis accident, but the degree of provision varies widely between older and newer designs. The 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 base structure. The base structure forms a sealed set of linked compartments that are designed to confine the radioactive material in the event of an accident. Cold water spray systems are provided to minimize overpressures in the event of a LOCA. The newer WWER-1000 type reactors utilize a single barrier large-dry prestressed concrete containment of similar design to that used by western PWR plants. Nongrouted 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.

2.2. MATERIALS OF CONSTRUCTION

Nuclear power plant concrete containment structures are composed of several constituents that. in concert, perform multiple functions (i.e., load carrying capacity, radiation shielding, and leaktightness). Primarily, these constituents include concrete, conventional steel reinforcement, prestressing steel, and steel or non-metallic liner materials. Concretes used in the fabrication of these structures typically consist of a moderate heat of hydration cement, fine aggregates (sand), water, various chemical and mineral admixtures for improving properties or performance of the concrete, and either normal-weight or heavy-weight coarse aggregate. Both the water and aggregate materials normally are obtained from local sources and subjected to material characterisation prior to use. Mild steel reinforcement is used primarily to provide tensile and shear load resistance/transfer and consists of plain carbon steel bar stock with deformations (lugs or protrusions) on the surface. Minimum tensile yield strengths generally range from 270 to 415 MPa. The prestressing system consists of tendons that are installed. tensioned, and then anchored to the hardened concrete forming the structure. The prestressing tendons counteract tensile loadings to prevent or minimize concrete cracking and also improve resistance to shear forces such as could develop during earthquake loadings. Both partial and full prestressing concepts have been utilized. Three major categories of prestressing system exist depending on the type of tendon utilized — wire, bar, or strand. Minimum ultimate strengths of prestressing tendons generally range from 2000 to 10,000 kN. The tendons are installed within preplaced ducts in the containment structure and are post-tensioned from one or both ends after the concrete has achieved sufficient strength. After tensioning, the tendons are anchored by buttonheads, nuts, or wedges, depending on the tendon type. Corrosion protection is provided by filling the ducts with wax or corrosion-inhibiting grease (unbonded), or Portland-cement grout (bonded). Some PCPVs have been circumferentially prestressed by wrapping wire or strand under tension around the vessel circumference. Metallic or nonmetallic liners are provided on the inside surface of many of the single-wall containments to provide a barrier against the leakage of gas. A typical metallic liner is composed of carbon steel plate stock up to 25-mm-thick, joined by welding, and anchored to the concrete by studs, structural steel shapes, or other steel products. Typical nonmetallic liners use epoxy or polyurethane paints applied in thicknesses up to one millimeter. The PWR containments and dry-well portions of BWR containments are typically lined with carbon steel plate. The liner of the wet-well of BWR containments, as well as the light-water reactor (LWR) fuel pool structures, typically consists of stainless steel plates. Certain LWR facilities have used carbon steel plates clad with stainless steel for the liner. Although the liner's primary function is to provide a leaktight barrier, it also can act as part of the formwork during concrete placement.

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TABLE 2.1. CRP MEMBER COUNTRY PRIMARY REGULATIONS, CODES, AND STANDARDS RELATED TO DESIGN, CONSTRUCTION, AND INSPECTION OF CONCRETE CONTAINMENT BUILDINGS

Country	Document	Commentary Relative to Ageing Management Considerations
Belgium	American Society of Mechanical Engineers	
	Section III, "Code for Concrete Reactor Vessels and Containments"	Stresses in reinforcing steel are limited
	Subsection IWE, "Requirements for Class MC and Metallic Liners of Class CC Components of Light-Water Cooled Plants"	Requires visual inspection of liner
	Subsection IWL, "Requirements for Class CC Concrete Components of Light-Water Cooled Plants"	Requires visual inspection of concrete and examination of post-tensioning systems
	US NRC Regulatory Guide 1.90, "Inservice Inspection of Prestressed Concrete Containments with Grouted Tendons"	Defines the instrumentation for monitoring prestress level
Canada	CAN/CSA-N287.2, "Material Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants," Canadian Standards Association, 1991	Ageing addressed through considerations for obtaining durable concrete
	CAN/CSA-N287.3, "Design Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants," Canadian Standards Association, 1993	Ageing addressed through provision of requirements for concrete cover and water-to-cement ratios to protect steel reinforcement from corrosion
	CAN/CSA-N287.4, "Construction, Fabrication and Installation Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants," Canadian Standards Association, 1992	Ageing addressed through considerations for obtaining durable concrete

TABLE 2.1. (cont.)

Country	Document	Commentary Relative to Ageing Management Considerations
Canada (cont.)	CAN/CSA-N287.5, "Testing and Examination Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants," Canadian Standards Association, 1993	Ageing addressed through specification of tests during construction to ensure concrete quality
	CAN/CSA-N287.6, "Pre-Operational Proof and Leakage Rate Testing Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants," Canadian Standards Association, 1994	Ageing not addressed
	CAN/CSA-N287.7, "In-Service Examination and Testing Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants," Canadian Standards Association, 1994	Ageing addressed through inservice inspection/ monitoring/testing for service life management (concrete leaktightness, prestressing cable relaxation, liner inspection/testing)
	CAN/CSA-A.23.1, "Concrete Materials and Methods of Concrete Construction," Canadian Standards Association, 1990	Ageing addressed through considerations for concrete durability
	CAN/CSA-A.23.2, "Methods of Test for Concrete," Canadian Standards Association, 1990	Ageing addressed through specification of tests/ acceptance criteria on concrete durability (e.g., chloride content, freeze/thaw, sulfate attack, seawater attack, and air pollution)
	CAN/CSA-A.23.3, "Design of Concrete Structures for Buildings," Canadian Standards Association, 1984	Ageing addressed through provision of design requirements for crack control, and concrete cover and water-to-cement ratios to protect steel reinforcement from corrosion.

TABLE 2.1. (cont.)

Country	Document	Commentary Relative to Ageing Management Considerations
India	"Design and Construction of Nuclear Power Plant Containment Structures (to be issued)," Atomic Energy Regulatory Board, Bombay	Long-term strain measurements using embedded gauges, determination of tendon forces and changes in force, inservice surveillance to identify any ageing-related degradation
	"Rules for Construction of Nuclear Power Plant Components", Section III, Division 2, "Code for Concrete Reactor Vessels and Containments," Subsection CC, American Society of Mechanical Engineers, 1992	See commentary under entry for United States
	CAN/CSA-N287 Series, Canadian Standards Association, 1991-1994	See commentary under Canada
	French Code RCC-G, "Design and Construction Rules for Civil Works of PWR Nuclear Islands," Electricité de France, 1988 Edition	Instrumentation for long term structural monitoring, requirements for structural integrity and leakage rate tests
Switzerland	SIA 160, "Effects on Load-Bearing Parts" Swiss Society of Engineers and Architects	Fatigue strength of load-bearing structures
	SIA 161, "Steel Structures," Swiss Society of Engineers and Architects	Construction rules of steel structures that reference fatigue or vibration-induced cracking; quality assurance rules for steel structures (SIA 161/1)
	SIA 162, "Concrete Structures," Swiss Society of Engineers and Architects	Rules for construction of concrete structures including prestressed structures, covers relaxation of prestressing tendons

TABLE 2.1. (cont.)

Country	Document	Commentary Relative to Ageing Management Considerations
Switzerland (cont.)	SIA 162/1	Material testing, sample testing of existing structures, material properties versus time
	SIA 162/2	Chloride in concrete, determination of chloride content in concrete samples to estimate potential risk for corrosion of embedded steel reinforcement
	SIA 162/3	Carbonation in concrete structures, determination of depth of carbonation in concrete structures to estimate potential risk for depassivation of embedded steel reinforcement
	SIA 169, "Maintenance of Civil Engineering Structures," Swiss Society of Engineers and Architects	Inspection programmes including main inspections and intermediate inspection documentation and record keeping
United Kingdom	British Standard 4975, "Prestressed Concrete Pressure Vessels for Nuclear Engineering," British Standards Institution, 1990	Guidance to assure 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, 1992	Provides guidelines on the tolerable levels of individual and social risk to workers and the public from nuclear power stations

TABLE 2.1. (cont.)

Country	Document	Commentary Relative to Ageing Management Considerations
United Kingdom (cont.)	"Safety Assessment Principles for Nuclear Plants" The Health and Safety Executive, 1992	Provides framework against which technical judgements 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
	"Nuclear Site Licence Standard Conditions," The Health and Safety Executive	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 (LC 28)
	"Pressurized Systems and Transportable Gas Container Regulations," The Health and Safety Executive, 1989	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," CP 114, and "The Structural Use of Prestressed Concrete in Buildings," CP 115, British Standards Institution, 1957 and 1959	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," Subsection CC (modified), American Society of Mechanical Engineers, 1995	Design and construction rules for PWR containment with revisions to definitions and specifying an ultimate load requirement to make consistent with BS4975

TABLE 2.1. (cont.)

Country	Document	Commentary Relative to Ageing Management Considerations
United Kingdom (cont.)	"Rules for the Inservice Inspection of Nuclear Power Plant Components," Section XI, "Code for Concrete Reactor Vessels and Containments," American Society of Mechanical Engineers, 1995	Provides basis for inspection procedures adopted for Sizewell "B" PWR prestressed concrete containment vessel
United States of America	Code of Federal Regulations, Title 10, "Energy,"	
	Part 50 - "Domestic Licensing of Production and Utilization Facilities"	
	§ 50.65 - Requirements for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants	Intended to provide basis that an effective programme to manage and to mitigate the effects of ageing is being implemented
	Appendix A - General Design Criteria for Nuclear Power Plants	Ageing not addressed directly, but requires containment to be designed to permit periodic inspection of important areas and an appropriate surveillance programme
	Appendix J - Primary Reactor Containment Leakage Testing for Water Cooled Power Reactors	Requires general visual inspection and conduction of leak-rate tests at specified intervals, results can be used for trending
	Part 54 - "Requirements for Renewal of Operating Licenses for Nuclear Power Plants"	Provides methodology for determination of need for an ageing management review of long-lived structures

TABLE 2.1. (cont.)

Country	Document	Commentary Relative to Ageing Management Considerations
United States of America (cont.)	American Concrete Institute	
	ACI 318 - "Building Code Requirements for Reinforced Concrete"	Ageing addressed through provision of requirements for concrete cover and water-to-cement ratios to protect steel reinforcement from corrosion
	American Society of Mechanical Engineers	
	Section III - "Rules for Construction of Nuclear Power Plant Components", Division 2, "Code for Concrete Reactor Vessels and Containments," 1995	Ageing approach similar to that of ACI 318
	Section XI - "Rules for Inservice Inspection of Nuclear Power Plant Components," 1995	Requires visual and post-tensioning system inspections at prescribed intervals, general performance of post-tensioning system with time
	US Nuclear Regulatory Commission (NRC) Regulatory Guides	
	1.18 - "Structural Acceptance Test for Concrete Primary Reactor Containments"	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)

TABLE 2.1. (cont.)

Country	Document		Commentary Relative to Ageing Management Considerations
United States of America (cont.)	US Nuclear Regulatory Commission (NRC) Regulatory Guides (cont.)		
	1.35 -	"Inservice Inspection of Ungrouted Tendons in Prestressed Concrete Containments"	Provides a procedure acceptable to NRC for inspection of ungrouted tendons
	1.35.1 -	"Determining Prestressing Forces for Inspection of Concrete Containments"	Provides a means for determining prestressing force levels to be used in inservice inspections
	1.90 -	"Inservice Inspection of Prestressed Concrete Containments with Grouted Tendons"	Provides a procedure acceptable to NRC for inspection of grouted tendons
	1.136 -	"Materials, Construction and Testing of Concrete Containment"	Ageing not directly addressed; provides information on NRC staff's position on acceptability for licensing actions of Article CC-2000, "Material," of ASME Code, Section III, Division 2 (related to assurances of material control and control of special processes such as welding)

2.3. APPLICABLE CODES AND STANDARDS

Reinforced concrete containment structures at nuclear power plants have been designed, constructed, operated, and inspected in accordance with national consensus codes and standards (e.g., Refs [2.4-2.9]). The rules in these documents were developed over the years by experienced people and are based on the knowledge that was acquired in testing laboratories and supplemented by field experience. Although differences in the codes and standards occur from country to country due to different approaches to ensuring continuing plant safety, the codes and standards tend to be conservative. Table 2.1 provides a listing of several national codes and standards for the design, construction, and inspection of concrete containment buildings, and how they relate to ageing management. In general these codes and standards address ageing management indirectly.

REFERENCES TO CHAPTER 2

[2.1] INTERNATIONAL ATOMIC ENERGY AGENCY, Pilot Studies on Management of Ageing of Nuclear Power Plant Components – Results of Phase I, IAEA-TECDOC-670, Vienna (1992).

[2.2] FEDERATION INTERNATIONALE DE LA PRECONTRAINTE, FIP State of Art Report: Concrete Containments, FIP/3/4, London (1978).

[2.3] 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, London (1982).

[2.4] 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), New York (1995).

[2.5] AMERICAN SOCIETY OF MECHANICAL ENGINEERS, "Rules for Inservice Inspection of Nuclear Power Plant Components", Section XI, ASME Boiler and Pressure Vessel Code, New York (1995).

[2.6] AMERICAN CONCRETE INSTITUTE, Code Requirements for Nuclear Safety Related Concrete Structures, ACI 349-85, Detroit, MI (1985).

[2.7] BRITISH STANDARDS INSTITUTION, Specification of PCPV for Nuclear Reactors, BS4975, London (1990) 42 pp.

[2.8] CANADIAN STANDARDS ASSOCIATION, N287 Series, CAN/CSA, Toronto (1991-1994).

[2.9] SWISS SOCIETY OF ENGINEERS AND ARCHITECTS (SIA), Concrete Structures, SIA 162, Zürich.

3. UNDERSTANDING AGEING

3.1. AGEING

As concrete ages, changes in its properties will occur as a result of continuing microstructural changes (e.g., cement hydration, crystallisation of amorphous constituents, and reactions between cement paste and aggregates), as well as environmental influences. These changes do not have to be detrimental to the point that reinforced concrete contained as part of a containment building will not be able to meet its functional and performance requirements. Concrete, however, can suffer undesirable degrees of change with time because of improper design or construction specifications, a violation of specifications, or unanticipated environmental conditions.

3.2. AGEING MECHANISMS AND EFFECTS

The longevity, or long-term performance of reinforced concrete containment buildings (as well as other safety-related concrete structures) is primarily a function of the durability or propensity of these structures to withstand the potential effects of degradation. Table 3.1 presents a summary of the degradation factors that potentially can impact the performance of the basic constituents that comprise reinforced concrete containment buildings (i.e., concrete, mild steel reinforcement, post-tensioning system, and liner systems). Also contained in the table is a listing of primary manifestations of each degradation factor. A listing of several areas in NPPs where concrete-related materials potentially may experience degradation is provided in this table. More detailed information to that summarised below is available elsewhere [3.1-3.5].

3.2.1. Concrete

The durability of concrete materials can be limited as a result of either adverse performance of its cement-paste matrix or aggregate constituents under either chemical or physical attack. In practice, these processes may occur concurrently to reinforce each other. In nearly all chemical and physical processes influencing the durability of concrete structures, dominant factors involved include transport mechanisms within the pores and cracks,[1] and the presence of water.

3.2.1.1. Chemical attack

Chemical attack involves the alteration of concrete through chemical reaction with either the cement paste or coarse aggregate, steel reinforcement,[2] or other embedded components. Generally, the attack occurs on the exposed surface region of the concrete (cover concrete), but with the presence of cracks or prolonged exposure, chemical attack can affect entire structural cross-sections. Chemical causes of deterioration can be grouped into three

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[1]Cracking occurs in virtually all concrete structures and, because of concrete's inherently low tensile strength, can never be totally eliminated. Cracks are significant from the standpoint that they can indicate major structural problems such as differential settlement (active cracks), provide an avenue for ingress of hostile environments (active or dormant cracks), and may inhibit a component from meeting its performance requirements (active or dormant cracks) (e g , diminished leaktightness or shielding capacity)

[2]Corrosion of steel reinforcement due to carbonation of the concrete or the action of chloride ions is covered under the section addressing mild steel reinforcement

TABLE 3.1. DEGRADATION FACTORS THAT CAN IMPACT THE PERFORMANCE OF SAFETY-RELATED CONCRETE STRUCTURES

a. Concrete

Ageing stressors/ service conditions	Ageing mechanism	Ageing effect	Potential degradation sites	Remarks (e.g., significance)
Percolation of fluid through concrete due to moisture gradient	Leaching and efflorescence	Increased porosity and permeability; lowers strength	Near cracks; Areas of high moisture percolation	Makes concrete more vulnerable to hostile environments; may indicate other changes to cement paste; unlikely to be an issue for high quality, low-permeability concretes
Exposure to alkali and magnesium sulfates present in soils, seawater or groundwater	Sulphate attack	Expansion and irregular cracking	Subgrade structures and foundations	Sulphate-resistant cements or partial replacement of cements used to minimise potential occurrence
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	Acid rain not an issue for containments
Combination of reactive aggregate, high moisture levels, and alkalis	Alkali-aggregate reactions leading to swelling	Cracking; gel exudation; aggregate pop-out	Areas where moisture levels are high and improper materials utilized	Eliminate potentially reactive materials; use low alkali-content cements or partial cement replacement
Cyclic loads/vibrations	Fatigue	Cracking; strength loss	Equipment/piping supports	Localized damage; fatigue failure of concrete structure unusual

TABLE 3.1. (cont.) a. Concrete (cont.)

Ageing stressors/ service conditions	Ageing mechanism	Ageing effect	Potential degradation sites	Remarks (e.g., significance)
Exposure to flowing gas or liquid carrying particulates and abrasive components	Abrasion; Erosion; Cavitation	Section loss	Cooling water intake and discharge structures	Unlikely to be an issue for containment
Exposure to thermal cycles at relatively low temperatures	Freeze/thaw	Cracking; spalling	External surfaces where geometry supports moisture accumulation	Air entrainment utilized to minimise potential occurrence
Thermal exposure/ thermal cycling	Moisture content changes and material incompatibility due different thermal expansion values	Cracking; spalling; strength loss; reduced modulus of elasticity	Near hot process and steam piping	Generally an issue for hot spot locations; can increase concrete creep that can increase prestressing force losses
Irradiation	Aggregate expansion; hydrolysis	Cracking; loss of mechanical properties	Structures proximate to reactor vessel	Containment irradiation levels likely to be below threshold levels to cause degradation
Consolidation or movement of soil on which containment is founded	Differential settlement	Equipment alignment, cracking	Connected structures on independent foundations	Allowance is made in design; soil sites generally include settlement monitoring instrumentation
Exposure to water containing dissolved salts (e.g., seawater)	Salt crystallisation	Cracking	External surfaces subject to salt spray; intake structures	Minimized through use of low permeability concretes, sealers, and barriers

TABLE 3.1. (cont.)

b. Mild steel reinforcing

Ageing stressors/ service conditions	Ageing mechanism	Ageing effect	Potential degradation sites	Remarks (e.g., significance)
Depassivation of steel due to carbonation or presence of chloride ions	Composition or concentration cells leading to corrosion	Concrete cracking and spalling; loss of reinforcement cross-section	Outer layer of steel reinforcement in all structures where cracks or local defects (e.g., joints) are present	Prominent potential form of degradation; leads to reduction of load-carrying capacity
Elevated temperature	Microcrystalline changes	Reduction of yield strength and modulus of elasticity	Near hot process and steam piping	Of significance only where temperatures exceed ~200°C
Irradiation	Microstructural transformation	Increased yield strength; reduced ductility	Structures proximate to reactor vessel	Containment irradiation levels likely to be below threshold levels to cause degradation
Cyclic loading	Fatigue	Loss of bond to concrete; failure of steel under extreme conditions	Equipment/piping supports	Localized damage; fatigue failure of concrete structures unusual

TABLE 3.1. (cont.) c. Prestressing

Ageing stressors/ service conditions	Ageing mechanism	Ageing effect	Potential degradation sites	Remarks (e.g., significance)
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	Potential degradation mechanism due to lower tolerance to corrosion than mild steel reinforcement
Elevated temperature	Microcrystalline changes	Reduction of strength; increased relaxation and creep	Near hot processes	Thermal exposure not likely to reach levels that can produce ageing effects in prestressing
Irradiation	Microstructural transformation	Increased strength; reduced ductility	Structures proximate to reactor vessel	Containment irradiation levels likely to be 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	Not likely as cyclic loadings are generally small in number and magnitude
Long term loading	Stress relaxation; creep and shrinkage of concrete	Loss of prestressing force	Prestressed concrete containments	Larger than anticipated loss of prestressing forces

TABLE 3.1. (cont.) d. Containment liners

Ageing stressors/ service conditions	Ageing mechanism	Ageing effect	Potential degradation sites	Remarks (e.g., significance)
Electrochemical reaction with environment (metallic)	Composition or concentration cells leading to general or pitting corrosion	Loss of cross-section; reduced leaktightness	Areas of moisture storage/accumulation, exposure to chemical spills, or borated water	Corrosion has been noted in several containments near the interface where the liner becomes embedded in the concrete
Elevated temperature (metallic)	Microcrystalline changes	Reduction of strength; increased ductility	Near hot process and steam piping	Thermal exposure not likely to reach levels that can produce ageing effects in metallic liners
Irradiation (metallic and non-metallic)	Microstructural transformation (metallic); increased cross-linking (nonmetallic)	Increased strength; reduced ductility	Structures proximate to reactor vessel	Containment irradiation levels likely to be below threshold levels to cause degradation
Cyclic loading due to diurnal or operating effects (metallic and non-metallic)	Fatigue	Cracking; reduced leaktightness	Inside surfaces of concrete containment building	Not likely as cyclic loadings are generally small in number and magnitude
Localized effects (non-metallic liners)	Impact loadings; stress concentrations; physical and chemical changes of concrete	Cracking; reduced leaktightness	Inside surfaces of concrete containment building	Potential problem in high traffic areas

categories: (1) hydrolysis of cement paste components by soft water; (2) cation exchange reactions between aggressive fluids and the cement paste; and (3) reactions leading to formation of expansion products [3.6]. The rate of chemical attack on concrete is a function of the pH of the aggressive fluid and the concrete permeability, alkalinity, and reactivity. Chemical attack of concrete may occur in several different forms as highlighted in the following sections.

Leaching and efflorescence

Water that contains little or no calcium ions, or acidic groundwater present in the form of dissolved carbon dioxide gas, carbonic acid, or bicarbonate ion, tends to hydrolyze or dissolve the alkali oxides and calcium-containing products. The rate of leaching is dependent on the amount of dissolved salts contained in the percolating fluid, rate of permeation of the fluid through the cement paste matrix, and temperature. Extensive leaching causes an increase in porosity and permeability, thus lowering the strength of the concrete and making it more vulnerable to hostile environments (e.g., water saturation and frost damage, or chloride penetration and corrosion of embedded steel). Leaching can also reduce the alkalinity of the concrete locally (i.e., lowers the pH and reduces the ability to passivate steel reinforcement at this location). The rate of leaching can be controlled by minimising the percolation of water through the concrete. Concretes produced using low water-cement ratios, adequate cement content, and proper compaction and curing are most resistant to leaching.

Efflorescence occurs on the surface of concrete following the percolation of a fluid (e.g., water) through the material, either intermittently or continuously, or when an exposed surface is alternately wetted and dried. It forms due to crystallization of the dissolved salts as a result of evaporation of the fluid or interaction with carbon dioxide in the atmosphere. As such, efflorescence is primarily an aesthetic problem rather than a durability problem, but may indicate that alterations to the cement paste are taking place in the concrete.

Sulphate attack

Magnesium and alkali sulphates present in soils, groundwater, and seawater react with the calcium hydroxide and alumina-bearing phases of Portland cement to form gypsum and ettringite. These reactions, if enough water is present, result in expansion and irregular cracking of the concrete that can lead to progressive loss of strength and mass. Structures subjected to seawater are more resistant to sulphate attack because of the presence of chlorides that form chloro-aluminates to moderate the reaction. Concretes that use cements low in tricalcium aluminate, contain pozzolanic materials such as fly ash or ground-granulated blast-furnace slag, and those that are dense and of low permeability are most resistant to sulphate attack. Also, the presence of steel reinforcement and prestressing in CCBs influences the existence and appearance of concrete cracking.

Acids and bases

Acids present in groundwater (e.g., sulfuric or carbonic) and certain plant internal fluids (e.g., boric and sulfuric acids) can combine with the calcium compounds in the hydrated cement paste (i.e., calcium hydroxide, calcium silicate hydrate, and calcium aluminate hydrate) to form soluble materials that are readily leached from the concrete to increase its porosity and permeability. The main factor determining the extent of attack is not so much the aggressiveness of the attacking acid, but more the solubility of the resulting calcium salt. The rate of deterioration is also accelerated if the aggressive chemical solution is flowing. Since under acid attack there is a conversion of the hardened cement, the concrete permeability is not as important as for other types of chemical attack (e.g., leaching and sulfate attack). Due to the large buffering capacity of concrete, acid rain will convert only an insignificant amount of the concrete in the structure [3.7]. Acid rain is even a smaller threat to a NPP than general civil engineering concrete structures because of their massive cross-sections.

As hydrated cement paste is an alkaline material, high quality concretes made with chemically stable aggregates normally are resistant to bases. However, sodium and potassium hydroxides in high concentrations (> 20%) can cause concrete to deteriorate. Under mild chemical attack, a dense concrete with low water-cement ratio may provide suitable resistance. As corrosive chemicals can attack concrete only in the presence of water, designs to minimise attack by acids and bases generally involve the use of protective barrier systems. Table 3.2 presents a listing of the reactivity with concrete of various chemicals that may be found in NPPs or the surrounding environment. References [3.8] and [3.9] present additional information on the effect of chemicals on concrete.

Alkali-aggregate reactions

Although all natural aggregate materials can react to some extent with the soluble alkaline components in concrete, some reactions can lead to deleterious expansion and cracking. Chemical reactions involving alkali ions (Portland cement), hydroxyl ions, and certain siliceous constituents that may be present in aggregate materials can form a gel. As the alkali-silica gel comes in contact with water, swelling (i.e., hydraulic pressure) occurs that can cause cracking that eventually could lead to complete destruction of the concrete [3.10]. Visible concrete damage starts with small surface cracks exhibiting an irregular pattern (or map cracking). The expansion 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 alkali-silica reactions. Expansion reactions also can occur as a result of alkali-carbonate reactions (i.e., dedolomitisation). Furthermore, it is quite common that once cracking has developed, the cracks permit access to the interior of the concrete and allow some other deleterious mechanisms to operate (e.g., leaching by percolating water accompanied by precipitation of calcium carbonate on surfaces).

Primary factors influencing alkali-aggregate reactions include the aggregate reactivity (i.e., amount and grain size of reactive aggregate), alkali and calcium concentrations in concrete pore water, cement content (i.e., alkali content), and adequate levels of moisture. Although alkali-aggregate reactions typically occur within 10 years of construction, deterioration has not occurred in some structures until 15 or even 25 or more years following construction. The delay in exhibiting deterioration indicates that there may be less reactive forms of silica that can eventually cause deterioration [3.11]. Prevention of the alkali-aggregate reactions is generally by elimination from consideration of deleteriously reactive aggregate materials through petrographic examinations, laboratory evaluations, and use of materials with proven service histories. Additional mitigating procedures include use of pozzolans, restricting the cement alkali contents to less than 0.6% by weight Na_2O equivalent, and application of barriers to restrict moisture ingress. Various codes and standards provide laboratory test methods meant to detect and minimise the occurrence of alkali-aggregate reactions [3.12-3.14].

TABLE 3.2. REACTIVITY OF VARIOUS MATERIALS WITH CONCRETE AND STEEL*

Material	Effect on concrete	Effect on steel
Acetone	Liquid loss by penetration (may cause slow disintegration)	None
Acidic Water (less than 6.5 pH)	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	Leaches	Slight
Deicing Salts	Scaling of non-air entrained concrete	Highly corrosive
Diesel Exhaust Gases	May disintegrate moist concrete by action of carbonic, nitric, or sulphurous acid; minimal effect on hardened dry concrete	Minimal
Hydrochloric Acid	Disintegrates concrete rapidly	Highly corrosive
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
Seawater	Disintegrates concrete with inadequate sulfate 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 sulfate resistance such as Type I Portland cement concrete)	Harmful at certain concentrations
Sulphuric Acid (sulphurous)	Disintegrates rapidly in concentration between 10 and 80%	Very corrosive

*Primary source: Ref. [3.8].

Carbonation

Carbon dioxide, present in the atmosphere, may react with the calcium hydroxide or other lime-bearing compounds in hardened concrete to produce a reduction in volume (i.e., shrinkage) and an increase in weight. Carbonation of concrete made with ordinary Portland cement also results in a slightly increased strength and a reduced permeability [3.15]. However, much more importantly, carbonation neutralises the alkaline nature of the hydrated cement paste and thus the protection of steel from corrosion is reduced (see Section 3.2.2.1).

3.2.1.2. Physical attack

Physical attack involves the degradation of concrete due to external influences and generally involves cracking due to exceeding the tensile strength of the concrete, or loss of surface material. Concrete attack due to overload conditions is not considered as an ageing mechanism because an assessment would generally be required at the time of occurrence. However, loads in excess of design could accelerate the ageing process due to effects such as formation of cracks that could permit entry of hostile environments.

Crystallization of chlorides and other salts

Salts can produce cracks in concrete through crystal growth pressures that arise through physical causes (e.g., repeated salt crystallisation due to evaporation in the pores). Structures in contact with fluctuating water levels or in contact with groundwaters containing large quantities of dissolved salts (e.g., $NaC\ell$, $CaSO_4$, and $NaSO_4$) are susceptible to this type of deterioration. The problem of salt crystallisation is minimised for low permeability concretes and where sealers or barriers have been effectively applied to prevent water ingress or subsequent evaporation.

Freeze-thaw attack

Concrete, when in a saturated or near saturated condition, can be susceptible to damage during freezing and thawing cycles caused by hydraulic pressure generated in the capillary cavities of the cement paste as water freezes. Damage to concrete resulting from freeze-thaw attack can take several forms: scaling, spalling, and pattern cracking (e.g., D-cracking) [3.11]. The damage is incurred after an extended number of cycles and is observed on exposed surfaces of affected structures. Factors controlling the resistance of concrete to freeze-thaw action include air entrainment (i.e., size and spacing of air bubbles as opposed to entrapped air), water-cement ratio, curing, strength, and degree of saturation. Selection of durable aggregate materials is also important. Guidelines for production of freeze-thaw-resistant concrete are provided in Refs [3.16-3.18] in terms of total air content as a function of maximum aggregate size and exposure condition.

Abrasion/erosion/cavitation

Progressive loss of material at the concrete surface can occur due to abrasion, erosion, or cavitation. Abrasion generally refers to dry attrition, while erosion is normally used to describe wear by the abrasive action of fluids containing solid particles in suspension. Cavitation relates to the loss of surface material by formation of vapor bubbles and their subsequent collapse, due to sudden change of direction or pressure in rapidly flowing water, on the surface of the structure. Resistance of concrete to abrasion and erosion is dependent on the quality of the concrete (low porosity, high strength) and in particular the aggregate particles used in the mix. While good quality concrete may exhibit resistance to abrasion and erosion, it may still suffer severe loss of surface material due to cavitation. The best way to guard against the effects of cavitation is to eliminate the cause(s) of cavitation.

Elevated temperature/thermal cycling

Exposure to elevated temperature and thermal gradients is important to concrete structures in that it affects the concrete's strength (i.e., ability to carry loads) and stiffness (i.e., structural deformations and loads that develop at constraints). The mechanical property variations result largely because of changes in the moisture content of the concrete constituents and progressive deterioration of the bond between the cement paste and aggregate (especially significant where thermal expansion coefficients for cement paste and aggregate are markedly different). Although resistance of concrete to elevated temperature is dependent on mix constituents, significant deterioration of the concrete uniaxial strength does not generally occur until the exposure temperature reaches ~400°C at which dehydration of calcium hydroxide occurs [3.19]. The response of concrete to elevated temperature exposure depends on a number of factors (e.g., type and porosity of aggregate, rate of heating, permeability, and moisture state). In addition to potential reductions in strength and modulus of elasticity, thermal exposure of concrete can result in cracking, or when the rate of heating is high and concrete permeability low, surface spalling can occur. Elevated temperatures also are important in that they affect the volume change and creep of concrete [3.20]. Codes pertaining to nuclear power plant structures generally handle elevated temperature applications by requiring special provisions (e.g., cooling) to limit the concrete temperature at or below a specified value (e.g., 65°C). References [3.21–3.23] provide additional information on the effects of elevated temperature on concrete materials and structures.

Thermal cycling, even at relatively low temperatures (<65°C), can have deleterious effects on concrete's mechanical properties (i.e., compressive, tensile and bond strengths, and modulus of elasticity are reduced). Most reinforced concrete structures are subjected to thermal cycling due to daily temperature fluctuations and are designed accordingly (i.e., inclusion of steel reinforcement). At higher temperatures (200° to 300°C), the first thermal cycle causes the largest percentage of damage, with the extent of damage markedly dependent on aggregate type and is associated with loss of bond between the aggregate and matrix [3.24]. Thermal cycles, also can become important if the deformation of the structure resulting from the temperature variations is constrained.

Irradiation

Irradiation due to either fast and thermal neutrons emitted by the reactor core or gamma rays produced as a result of capture of neutrons by members (particularly steel) in contact with concrete, can affect the concrete. The fast neutrons are mainly responsible for the considerable volume increase, caused by atomic displacements, that has been measured in certain aggregate (e.g., flint). Nuclear heating occurs as a result of energy introduced into the concrete as the neutrons or gamma radiation interact with the molecules within the concrete. Reference [3.25] indicates that nuclear heating is negligible for incident energy fluxes less than 10^{10} MeV/cm^2 per s. Gamma rays produce radiolysis of water in cement paste that can affect concrete's creep and shrinkage behavior to a limited extent and also result in evolution of gas. Prolonged exposure of concrete to irradiation could result in decreases in strength and modulus of elasticity. Irradiation has little effect on shielding properties of concrete beyond the effect of moisture loss due to temperature increase. Approximate threshold levels necessary to create measurable damage in concrete have been reported in limited research studies [3.26]. These levels are 1×10^{19} neutrons/cm^2 for neutron fluence and 10^{10} rads of dose for gamma radiation. Additional information on the interaction of radiation and concrete is available in Ref. [3.27].

Fatigue/vibration

Concrete structures subjected to changes in loading, temperature, or moisture content (that are not large enough to cause failure in a single application) can be damaged by fatigue. Fatigue damage initiates as irreversible deformations (e.g., microcracks in the cement paste or viscous flow) proximate to the large aggregate particles, reinforcing steel, or stress concentrations (e.g., defects). Upon continued or reversed load application, these microcracks may propagate to form structurally significant cracks that can expose the concrete and reinforcing steel to hostile environments or produce increased deflections. Ultimate failure of a concrete structure in fatigue will occur as a result of excessive cracking, excessive deflections, or brittle fracture. Fatigue failure of concrete is unusual because of its good resistance to fatigue [Ref. 3.28] and concrete structures are designed using codes that limit design stress levels to values below concrete's endurance limit. However, as structures age, there may be instances of local fatigue damage at locations where reciprocating equipment is attached, or at supports for pipes that exhibit vibrations.

Settlement

All structures have a tendency to settle during construction and early life. Excessive settlement or differential settlement can cause misalignment of equipment and lead to overstress conditions in the containment (e.g., cracking). The amount of settlement is dependent on the physical properties of the foundation material at the site, which may range from bedrock (minimal settlement expected) to compacted soil (some settlement expected), and on quality control of foundation construction, including piling. Settlement is considered in the design of the containment and is not expected to be significant. When the containment is sited on soils, the potential for settlement is acknowledged and monitoring programmes are implemented to confirm that design criteria are met. In general, most of the settlement will occur within a few months after construction and become negligible after this period.

3.2.2. Mild steel reinforcing

Mild steel reinforcing is provided in concrete structures to resist tensile stresses and compressive stresses for elastic design, provide structural reinforcement where required by limit condition design procedures, and to control the extent and width of cracks at operating temperatures. Potential causes of degradation of the mild reinforcing steel are corrosion, elevated temperature, irradiation, and fatigue. Of these, corrosion is the factor of most concern for ageing management of NPP structures. Information on the other potential degradation factors is provided for completeness and special situations that might occur.

3.2.2.1. Corrosion

Corrosion of steel in concrete is an electrochemical process that can assume the form of either general or pitting corrosion. Both water and oxygen must be present for corrosion to occur. The electrochemical potentials that form the corrosion cells may be generated in two ways: (1) composition cells formed when two dissimilar metals are embedded in concrete, such as steel rebars and aluminum conduit, or when significant variations exist in surface characteristics of the steel; and (2) concentration cells formed due to differences in concentration of dissolved ions in the vicinity of steel, such as alkalies, chlorides, and oxygen [3.6]. As a result, one of the two metals (or different parts of the same metal when only one metal is present) becomes anodic and the other cathodic to form a corrosion cell. Other potential causes of corrosion include the effects of stray electrical currents or galvanic action with an embedded steel of different metallurgy. The transformation of metallic iron to ferric oxide (rust) is accompanied by an increase in volume that can cause cracking and spalling of the concrete. In addition, corrosion will result in a reduction in effective steel cross-section (e.g., load capacity). Depending on the type of corrosion, local embrittlement may occur.

In good-quality, well-compacted concretes, 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 to form on the iron surface (i.e., metallic iron will not be available for anodic activity). However, when the concrete pH falls below 11, a porous oxide layer (rust) can form on the reinforcing steel due to corrosion. Carbonation or the presence of chloride ions can destroy the passive iron oxide film.

Reduction of the concrete pH can occur as a result of carbonation [i.e., calcium hydroxide is converted to calcium carbonate (calcite)] or leaching of alkaline substances by water. The penetration of carbon dioxide from the environment is generally a slow process dependent on the concrete permeability, the concrete moisture content, and the carbon dioxide content and relative humidity of the ambient medium (e.g., carbonation occurs primarily at relative humidities between 40 and 70% reaching a maximum at about 50%). The extent of carbonation in good quality concretes such as used in the fabrication of NPP containment buildings is limited. Reference [3.29] notes that the time required to carbonate 20 mm of good quality concrete is on the order of tens of years. Carbonation, however, may be accelerated due to the concrete being porous (i.e., poor quality) or the presence of microcracks. In NPPs carbonation is most likely to be a problem at the inside concrete surfaces, especially those exposed to relatively low humidities and elevated temperatures [3.30]. The extent of carbonation can be determined by treating a freshly exposed concrete surface with phenolphtalein [3.31].

The passive iron oxide film on the steel reinforcement can also be destroyed by the penetration of chloride ions, even at high alkalinites (pH >11.5) (e.g., Ref. [3.30] notes that at a pH of 13.2 more than 8000 ppm of chloride ions are required to induce corrosion, however, at a pH of 11.6 only about 71 ppm are required). Maximum permissible chloride contents, as well as minimum recommended cover requirements have been provided in codes and guides (e.g., Refs [3.12] and [3.18]). The threshold acid-soluble chloride contents reported by various investigators to initiate steel corrosion range from 0.15 to 1.0% by weight of cementitious materials, whereas codal limits range from 0.2 to 0.4% [3.32]. Chlorides may be present in concrete due to external sources (e.g., seawater effects and deicing salts) or may be naturally introduced into the concrete via aggregate or mix water transport. Furthermore, when large amounts of chloride are present, concrete tends to hold more moisture, which also increases the risk of steel corrosion by lowering concrete's electrical resistivity. Once the passivity of the steel is destroyed, the electrical resistivity of concrete and availability of oxygen control the rate of corrosion. Methods of excluding external sources of chloride ions from concrete are provided in Refs [3.8] and [3.33].

3.2.2.2. Elevated temperature

The property of mild steel reinforcement of most importance to design is the yield strength. The reinforcing steel yield strength can be affected by elevated temperature exposure, but the temperatures to cause an affect are significantly higher (except under unusual conditions) than would be experienced by a NPP concrete structure. Data for German reinforcing steels [3.34] indicate that for temperatures up to ~200°C, the yield strength is reduced by 10% or less, and at 500°C it falls to about 50% its reference room temperature value. Hot-rolled steels tend to resist the effects of temperature better than cold drawn or twisted steel. The steel modulus of elasticity exhibits similar reductions with increasing temperature. Other data [3.35 and 3.36] confirm the effects of temperatures above 200°C on mild steel reinforcing as well as providing a threshold temperature of about 300°C for loss of bond properties with the concrete.

3.2.2.3. Irradiation

Neutron irradiation produces changes in the mechanical properties of carbon steels (e.g., increased yield strength and rise in the ductile-to-brittle transition temperature). The changes result from the displacement of atoms from their normal sites by high-energy neutrons, causing the formation of interstitials and vacancies. A threshold level of neutron fluence of 1×10^{18} neutrons per square centimeter has been cited for alteration of reinforcing steel mechanical properties [3.37]. Fluence levels of this magnitude are not likely to be experienced by the safety-related concrete structures in NPPs, except possibly in the concrete primary biological shield wall over an extended operating period [3.38].

3.2.2.4. Fatigue

Fatigue of the mild steel reinforcing would be coupled with that of the surrounding concrete. The result of applied repeated loadings, or vibrations, is generally a loss of bond between the steel reinforcement and concrete. At a large number of cycles, the strength of the mild steel reinforcing 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 published occurred at relatively high stress/cycle combinations [3.28]. Because of the typically low stress levels at normal operating conditions in reinforcing steel elements in NPP safety-related concrete structures, fatigue failure is not likely to occur.

3.2.3. Prestressing systems

Prestressing systems[3] are contained in concrete structures to impart forces to the concrete to maintain the concrete in compression during postulated design basis conditions (i.e., essentially crack free). Potential causes of degradation of the prestressing systems include corrosion, elevated temperature, irradiation, fatigue, and loss of prestressing force. Of these, corrosion and loss of prestressing force are most pertinent from a NPP ageing management perspective.

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[3]The terms prestressing and post-tensioning systems are used interchangeably.

3.2.3.1. Corrosion

Corrosion of prestressing systems (tendons and anchorages) can be highly localised or uniform. Most prestressing corrosion-related failures involving general civil engineering structures have been the result of localised attack produced by pitting, stress corrosion, hydrogen embrittlement, or a combination of these. Pitting is the electrochemical process that results in locally intensified material loss at the tendon surface, potentially reducing the cross-section to the point where it is incapable of supporting load. Stress corrosion cracking results in the fracture of a normally ductile metal or alloy under stress (tensile or residual) while in specific corrosive environments. Hydrogen embrittlement, frequently associated with hydrogen sulfide exposure, occurs when hydrogen atoms enter the metal lattice and significantly reduce its ductility. Hydrogen embrittlement also may occur as a result of improper application of cathodic protection to post-tensioning systems [3.39]. Failure of post-tensioning systems also can occur as a result of microbiologically-induced corrosion. Due to the stress state in the prestressing systems, the tolerance for corrosion attack is much less than for the mild steel reinforcement. Protective systems, generally consisting of grout-, grease- or wax-filled conduits, are commonly used to shield the prestressing system from environmental effects, primarily corrosion.

3.2.3.2. Elevated temperature

The effect of elevated temperature on all heat-treated and drawn wires can be significant, and on cooling the wires may not regain their initial strength because the heating destroys the crystal transformations achieved by the heat-treating process. Short-term heating, on the order of 3 to 5 min., even to temperatures as high as 400°C, however, may not harm the prestressing wire's mechanical properties [3.40]. Results of a Belgian study [3.34] involving 30 types of prestressing steels indicate that thermal exposures up to ~200°C do not significantly reduce (<10%) the tensile strength of prestressing wires or strands. References [3.36] and [3.41] support results of the Belgian study. The effect of elevated temperature, for exposures from 21° to 649°C, on the stress-strain behaviour of one type of prestressing steel (i.e., ASTM A 421) is provided in Ref. [3.42].

Elevated-temperature exposures also affect the relaxation and creep properties of prestressing tendons. Reference [3.43] indicates that losses in a 15.2-mm-diameter strand initially stressed to 75% of its guaranteed ultimate tensile strength at 40°C will be 5 to 6.4% after 30 years. Relaxation losses of tendons composed of stress-relieved wires are of about the same magnitude as stress-relieved strand, but relaxation of a strand is greater than that of its straight constituent wire because of the combined stress relaxation in the helical wires [3.44]. Creep (length change under constant stress) of stress-relieved wire is negligible up to 50% of its tensile strength. Also, the creep effect in steel varies with its chemical composition as well as with mechanical and thermal treatment applied during the manufacturing process. As temperature levels experienced by the prestressing tendons in concrete containment buildings generally are significantly below 200°C, the possibility for thermal damage to the prestressing steels under normal operating conditions is low.

3.2.3.3. Irradiation

Irradiation of prestressing system steel affects its mechanical properties because atoms are displaced from their normal sites by high-energy neutrons to form interstitials and vacancies. These defects can propagate or combine and effectively both strengthen the steel and reduce its ductility; or at higher temperatures they can recombine and annihilate each other and, for a given neutron dose, reduce the irradiation damage [3.37]. Results obtained from studies [3.37] 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} neutrons/cm^2 (flux of 2×10^{10} neutrons·per sq cm·per s) showed that for exposures up to this level, the relaxation behavior of irradiated and unirradiated materials was similar. As these flux levels are higher than the level likely to be experienced in a concrete containment building, under normal operating conditions irradiation is not expected to impact the performance of the prestressing system.

3.2.3.4. 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: (1) failure of the concrete due to flexural compression; (2) failure of the concrete due to diagonal tension or shear; (3) failure of the prestressing steel due to flexural, tensile-stress variations; (4) failure of pre-tensioned beams (grouted tendons) due to loss of bond stress; and (5) failure of the end anchorages of post-tensioned structures [3.45]. The majority of fatigue failures that occurred while testing prestressed concrete beams have resulted from fatigue of the tendons due to stress concentrations that occurred in the tendon at a location where a crack forms. In unbonded post-tensioned construction the end anchorages could be subjected to some variation in stress under the action of changing external load, but unbonded tendons are not generally used in members subjected to frequent variations in stress.

3.2.3.5. Loss of prestressing force

Maintaining an adequate level of prestressing force in post-tensioned concrete containments is important to the overall safety of the NPP, especially during postulated accident conditions. Primary contributors to the loss of initial force level that was applied by the prestressing tendons include (1) friction, (2) end anchorage deflection (take up and slip), (3) elastic shortening, (4) tendon relaxation, and (5) concrete creep and shrinkage [3.46]. Of these factors, tendon relaxation and concrete creep and shrinkage are time-dependent factors and thus ageing related.

Stress relaxation, defined as loss of stress (force) in the prestressing steel when the strain (elongation) does not vary, is related to tendon material properties, initial stress level, exposure temperature, and time. Creep and shrinkage of concrete represent volume changes of the concrete that occur over the life of the structure that can significantly affect the force levels in the tendons. Information pertaining to concrete creep and shrinkage and its relationship to prestress loss in concrete containment buildings is presented in Ref. [3.47]. Guidelines for developing surveillance programmes to monitor loss of prestressing forces in concrete containment buildings have been developed (e.g., Refs [3.48] and [3.49]).

3.2.4. Liners

Both non-metallic and metallic liner systems have been used in concrete containment buildings to provide a leaktight boundary and facilitate cleanup operations should decontamination be required.

The primary sources of degradation of non-metallic liners are cracks due to localised effects (e.g., stress concentrations) or physical or chemical changes of the concrete. The integrity of non-metallic liners also can be compromised due to localised impacts or accumulated radiation effects.

Metallic liners are subject to the same general degradation mechanisms as mild steel reinforcement, of which corrosion and fatigue are the most important from an ageing management perspective. For some components a corrosion allowance may have been added to the thickness during the design stage. However, little allowance will have been provided for the relatively thin liner plate (i.e., ~6.3-mm thick). Typically the liner plate is coated, either with a primer or a primer-finish coat system to prevent corrosion (e.g., zinc-rich primer with polyamide epoxy or modified phenolic coatings). The influence of local corrosion attack that can lead to loss of leaktightness is of most concern. Local attack may result due to accumulation of moisture in areas experiencing loss of coating integrity, or failure of adjoining floor-liner sealant. Reference [3.50] contains corrosion data for structural steel in numerous environments. In general, depending on the environmental parameters, this reference notes that surface corrosion rates generally range from 0.001 mm/year to 0.03 mm/year. Fatigue related problems may occur at local stress intensification points in the liner (e.g., metal delaminations, weld defects, arc strikes, shape changes near penetrations, structural attachments, and concrete-floor interfaces). Potential sources of repeated loads include polar crane operations, vibrations, or thermal cycling.

3.2.5. Structural steel piles

Corrosion of structural steel piles, used in certain containment buildings for transferring foundation loadings to greater depths below grade, is also a possible degradation mechanism. Similar to other containment steel, the concern for piles is from localised corrosion resulting in significant loss of cross-sectional area. One study [3.51] examined corrosion data from 43 piling installations of varying depths (up to 41.5 m) with times of exposure ranging from 7 to 50 years in a wide variety of conditions. The conclusion of this study was that the type and amount of corrosion observed in steel pilings driven in undisturbed soil, regardless of soil characteristics and properties, was not sufficient to significantly affect the piling's performance as load-bearing structures. However, pilings placed in oxygen-enhanced fills, those exposed above grade, or those exposed to seawater, salt spray, or underground aggressive water may be somewhat affected [3.52].

3.2.6. Anchorages

Anchorage to concrete is required for heavy machinery, structural members, piping, ductwork, cable trays, towers, and many other types of structures in nuclear power plants. An anchorage might have to meet certain requirements for ease of installation, load capacity, resistance to vibration, preload retention, temperature range, corrosion resistance, post-installation or pre-installation, and ease of inspection and stiffness [3.53]. In meeting its functions, loads that the anchor must transfer to the concrete vary over a wide combination of tension, bending, shear, and compression. Several potential factors related to failure or degradation of the anchorage systems include design detail errors, installation errors (improper embedment depth or insufficient lateral cover, improper torque), material defects (low anchor or concrete strengths), shear or shear-tension interaction, slip, and preload relaxation [3.53]. Ageing mechanisms that could impair the ability of an anchorage to meet its performance requirements would be primarily those that result in deterioration of concrete (e.g., fatigue and vibration), because if a failure did occur it would most likely initiate in the concrete. Corrosion could also impact the performance of an anchorage.

REFERENCES TO CHAPTER 3

[3.1] NAUS, D.J., et al., Report on Aging of Nuclear Power Plant Reinforced Concrete Structures, NUREG/CR-6424 (ORNL/TM-13148), Lockheed Martin Energy Research Corp., Oak Ridge National Laboratory, Oak Ridge, TN (1996).

[3.2] NAUS, D.J., Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants, NUREG/CR-4652 (ORNL/TM-10059), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, TN (1986).

[3.3] PWR Containment Structures License Renewal Industry Report: Revision 1, EPRI TR-103835 (NUMARC 90-01), Electric Power Research Institute, Palo Alto, CA (1994).

[3.4] HOOKHAM, C.J., SHAH, V.N., Insights for Aging Management of Major Light Water Reactor Components, Vol. 2 — Reinforced and Prestressed Concrete Containments, NUREG/CR-5314 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, ID, July 1994 (draft).

[3.5] INTERNATIONAL ATOMIC ENERGY AGENCY, Pilot Studies on Management of Ageing of Nuclear Power Plant Components — Results of Phase I, IAEA-TECDOC-670, Vienna (1992).

[3.6] MEHTA, P.K., Concrete-Structure, Properties, and Materials, Prentice-Hall, Inc., Englewood Cliffs, NJ (1986).

[3.7] COMITE EURO-INTERNATIONAL DU BETON, Durable Concrete Structures — Design Guide, Thomas Telford Service Publisher, London (1992).

[3.8] ACI COMMITTEE 515, "A Guide for the Use of Waterproofing Dampproofing, Protective, and Decorative Systems for Concrete", pp. 515.1R-6 to 515.1R-11 in ACI Manual of Concrete Practice–Masonry, Precast Concrete, Special Processes, Table 2.5.2 of Part 5, American Concrete Institute, Detroit, MI (1990).

[3.9] DEUTSCHE INDUSTRIE NORMEN, Rating the Effect of Aggressive Liquid, Soil, and Gases Upon Concrete (Beurteilung Beton Angreiffender Wässer, Boden, und Gase), DIN 4030, Berlin (1969).

[3.10] AMERICAN SOCIETY FOR TESTING AND MATERIALS, Alkalies in Concrete, ASTM STP 930, (V.H. Dodson, Ed.), West Conshohocken, PA (1986).

[3.11] MINDESS, S., YOUNG, J.F., Concrete, Prentice-Hall, Inc., Englewood Cliffs, NJ (1981).

[3.12] CANADIAN STANDARDS ASSOCIATION, Concrete Materials and Methods of Concrete Construction, CAN/CSA-A23.1, Appendix B, Toronto (1993).

[3.13] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Proposed Test Method for Accelerated Expansion of Mortar Bars Due to Alkali-Silica Reaction", ASTM P 214, Annual Book of ASTM Standards, Vol. 04.02, West Conshohocken, PA (1991).

[3.14] 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, West Conshohocken, (1991).

[3.15] NEVILLE, A.M., BROOKS, J.J., Concrete Technology, Longman Scientific & Technical, Essex (1987).

[3.16] ACI COMMITTEE 201, Guide to Durable Concrete, Journal of American Concrete Institute **74** (12), Detroit, MI (1977) 573–609.

[3.17] CANADIAN STANDARDS ASSOCIATION, Concrete Materials and Methods of Concrete Structures, CAN/CSA - A23.1, Appendix B, Toronto (1990).

[3.18] ACI COMMITTEE 318, Building Code Requirements for Reinforced Concrete, ACI Standard 318-71, American Concrete Institute, Detroit, MI (1971).
[3.19] BAZANT, Z.P., et al., Normal and Refractory Concretes for LMFRB Applications — Vol. 1: Review of Literature on High-Temperature Behavior of Portland Cement and Refractory Concrete, EPRI NP-2437, Electric Power Research Institute, Palo Alto, CA (1982).
[3.20] UDDIN, T., CULVER, C.G., Effects of Elevated Temperature on Structural Materials, J. Str. Div. 101(7), American Society of Civil Engineers, New York (1975).
[3.21] MALHOTRA, H.L., The Effect of Temperature on Compressive Strength of Concrete, Mag. of Concrete Res. **8** (23), London (1956) 85–94.
[3.22] CRUZ, C.R., Elastic Properties of Concrete at High Temperature, J. PCA – Research and Development Laboratories **8**(1), Portland Cement Association, Skokie, IL (1966) 37–45.
[3.23] ABRAMS, M.S., "Compressive Strength of Concrete at Temperatures to 1600°F", Temperature and Concrete, paper SP 25-2 in SP-25, American Concrete Institute, Detroit, MI (1971) 33–58.
[3.24] BERTERO, V.V., POLIVKA, M., "Influence of Thermal Exposure on Mechanical Characteristics of Concrete", SP-34, paper 28, in Concrete for Nuclear Reactors, American Concrete Institute, Detroit, MI (1972) 505–531.
[3.25] 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, La Grange Park, IL (1985).
[3.26] HILSDORF, H.K., et al., "The Effects of Nuclear Radiation on the Mechanical Properties of Concrete", ACI SP-55, Douglas McHenry Int. Symp. on Concrete and Concrete Structures, American Concrete Institute, Detroit, MI (1978).
[3.27] KAPLAN, M.F., "Concrete Radiation Shielding", Concrete Design & Construction Series, Longman Scientific & Technical, Essex (1989).
[3.28] ACI COMMITTEEE 215, Considerations for Design of Concrete Structures Subjected to Fatigue, ACI 215R-74, American Concrete Institute, Detroit, MI (1974).
[3.29] PAGE, C.L., TREADWAY, K.W.J., Aspects of the Electrochemistry of Steel in Concrete, Nature **297** (1982) 109–115.
[3.30] CLIFTON, J.R., Predicting the Remaining Service Life of Concrete, NISTIR 4712, US Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD (1991).
[3.31] RILEM RECOMMENDATION CPC-18, Measurement of Hardened Concrete Carbonation Depth, Materials and Structures **24** (126), Cachan, France (1988) 453–455.
[3.32] HUSSAIN, S.E., AL-GANTANI, A.S., RASHEEDUZZAFAR, Chloride Threshold for Corrosion of Reinforcement in Concrete, ACI Materials Journal **93** (6), American Concrete Institute, Detroit, MI (1996) 534–548.
[3.33] ACI COMMITTEE 222, Corrosion of Metals in Concrete, ACI 222R-89, American Concrete Institute, Detroit, MI (1989).
[3.34] SCHNEIDER, U., et al., Effect of Temperature on Steel and Concrete for PCPV's, Nuclear Engineering and Design **67**, Elsevier Science, Netherlands (1981) 245–258.
[3.35] SMITH, P., "Resistance to High Temperatures", STP 169B, Significance of Tests and Properties of Concrete and Concrete-Making Materials, American Society for Testing and Materials, West Conshohocken, PA (1978).
[3.36] ABRAMS, M.S., CRUZ, C.R., The Behavior at High Temperature of Steel Strand for Prestressed Concrete, Research Department Bulletin No. 134, Portland Cement Association, Skokie, IL (1961).

[3.37] COWEN, A., NICHOLS, R.W., "Effect of Irradiation on Steels Used in Pressure Vessels", Prestressed Concrete Pressure Vessels, Group D, paper 20, (Marilyn S. Udell, Ed.), The Institute of Civil Engineers, London (1968) 229–235.

[3.38] SCIENCE APPLICATIONS, INC., Study of Radiation Dosage to Structural Components in Nuclear Reactors, EPRI NP-152, Electric Power Research Institute, Palo Alto, CA (1977).

[3.39] 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, United Kingdom, 1990), Elsevier Applied Science, New York (1990) 515–524.

[3.40] PODOLNY, W., Jr., MELVILLE, T., Understanding the Relaxation in Prestressing, Prestressed Concrete Institute Journal **14**(4), Chicago, IL (1969).

[3.41] HARMATHY, T.Z., STANZACK, W.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).

[3.42] 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).

[3.43] CAHILL, T., BRANCH, G.D., "Long-Term Relaxation Behavior of Stabilized Prestressing Wires and Strands", Group D, paper 19, Prestressed Concrete Pressure Vessels (Marilyn S. Udell, Ed.), The Institute of Civil Engineers, London (1968) 219–228.

[3.44] FREEDMAN, S., "Properties of Materials for Reinforced Concrete", Handbook of Concrete Engineering, (M. Fintel, Ed.), Van Nostrand Reinhold, New York (1974).

[3.45] LIBBY, J.R., Modern Prestressed Concrete-Design Principles and Construction Methods, Van Nostrand Reinhold, New York (1971).

[3.46] PODOLNY, W., Jr., Understanding the Losses in Prestressing, Journal of the Prestressed Concrete Institute **14** (5), Chicago, IL (1969).

[3.47] 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), International Association for Bridge and Structural Engineering, Zürich (1995) 1411–16.–

[3.48] BPEL, "Règles Techniques de Conception et de Calcul des Ouvrages et Constructions en Béton Précontraint Suivant la Méthode des États Limites", Fascicule 62 de CCTG (1991).

[3.49] UNITED STATES NUCLEAR REGULATORY COMMISSION, "Inservice Inspections of Ungrouted Tendons in Prestressed Concrete Containment Structures", Regulatory Guide 1.35, Rev. 3, Office of Standards Development, Washington, DC (1990).

[3.50] ASM INTERNATIONAL, Metals Handbook, Ninth Edition, Vol. 13, Metals Park, OH (1987).

[3.51] ROMANOFF, M., Corrosion of Steel Pilings in Soils, Monograph 58, National Bureau of Standards, Gaithersburg, MD (1962).

[3.52] PECK, R., et al., Foundation Engineering, Wiley and Sons, New York (1974).

[3.53] MEINHEIT, D.F., HEIDBRINK, F.D., Behavior of Drilled-In Expansion Anchors, Concrete International **7** (4) (1985) 62–66.

4. DETECTING AGEING

Concrete containment buildings in NPPs are subjected in use to many types of environmental influence that can impact their ability to continue to meet functional and performance requirements. Due to the significant safety as well as economic influences that could result if these structures were to deteriorate to unacceptable performance levels, it is important that they be inspected at regular intervals. As noted in Fig. 1.1, inspection is one of the key elements of a condition assessment programme.

Continuing satisfactory performance of CCBs over an extended period of time is dependent in large measure on the durability of its basic components. Techniques for detection of degradation should therefore concentrate on these elements (i.e., concrete, reinforcing steel, prestressing system, penetrations, liners, waterstops/seals/gaskets, and protective coatings). Contained in the balance of this chapter is a description of the current practice relative to techniques that are available for inspection of NPP CCBs.

4.1. CURRENT PRACTICE

There is a vast variety of test methods available for use in performing inspections of reinforced concrete structures and their materials of construction. This chapter focuses on methods most commonly used and on those that represent good practice for the detection of degradation of elements used in the construction of concrete containment structures. Tables 4.1 and 4.2 provide a summary listing of methods to assess properties or characteristics of concrete, and metallic materials and protective media, respectively. Additional information to that provided below may be obtained from Refs [4.1–4.6]. Often the most effective approach to detecting ageing effects is to use a combination of testing methods.

4.1.1. Concrete

Primary manifestations of distress that are present or can occur in concrete used to fabricate CCBs include cracking, voids, and delaminations; and strength losses.[1] Methods used to detect discontinuities in concrete structures generally fall into two categories: direct and indirect. Direct methods involve a visual inspection of the structure, removal/testing/analysis of material(s), or a combination of the two. The indirect methods generally measure some parameter from which an estimate of the extent of degradation can be made through existing correlations. Quite often, however, evaluation of concrete structures and materials requires use of a combination of test methods since no single testing technique is available that will detect all potential degradation factors. For discussion purposes, testing methods are grouped into categories of nondestructive and destructive testing. Assessments of inaccessible concrete components would be done either through removal of material to expose the component of interest and applying the methods described below, or indirectly through environmental evaluations (i.e., quantification of the aggressiveness of the ambient environment).[2]

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[1]Corrosion of embedded steel reinforcement, which also is one of the primary forms of degradation of concrete structures, is addressed in Section 4.1.2

[2]If the ambient environment is determined to be potentially aggressive, additional testing and evaluation is required that may involve removal of material to expose the component for direct inspection and testing

4.1.1.1. Non-destructive testing

Non-destructive test methods can be used to indicate the strength, density and quality of concrete; locate and characterize voids or cracks in concrete; and locate steel reinforcement and indicate depth of concrete cover. Nondestructive testing methods include (1) visual inspection, (2) leakage rate, (3) audio, (4) infrared thermography, (5) magnetic, (6) acoustic, (7) radiation/nuclear, (8) tomography, (9) rebound hammer, (10) ultrasonic pulse velocity, (11) modal analysis, (12) radar, and (13) instrumentation.

Visual inspection

Visual inspection generally is the basic method used as a first step in a typical inspection programme. A high quality visual inspection of exposed concrete is able to detect and define areas of ageing-related distress that result in visible effects on the surface of the structure (e.g., cracking, moisture movement, mechanical degradation, spalling, volume change, or cement-aggregate interactions). Visual inspections also include periodic mapping and measurements to provide a history of crack appearance and development that can assist in identifying the cause and establish whether the crack is active or dormant. The primary limitation of this method is that it cannot reveal internal degradation of the concrete structure when there are no visible symptoms on the surface (e.g., subsurface cracking, voids, and delaminations; and extent of cracking).

Integrated leakage rate

The leakage rate of concrete containments is evaluated by pressurising the containment with air to a preestablished level (e.g., peak pressure associated with a design-basis accident), and monitoring the leakage as a function of time. Both full- and partial-pressure testing have been utilised. Pressure, temperature, and vapor pressure sensors are used during the test to sample the containment atmosphere. Changes in the contained air mass define the leakage rate. The method can be augmented by spraying a thin film of soap solution on cracked areas and visually monitoring the formation of bubbles during pressurisation. The presence and rate of bubble formation indicate openness of cracks. The primary limitation of the leakage-rate test is that it is done while the plant is shut down. Concerns have been raised that each pressurisation imposes a high magnitude, low frequency cyclic load on the containment that may affect its performance.

Audio

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. Solid areas of concrete will produce a characteristic "metallic ringing" sound when impacted, while defects in the form of debonds, cracks, or other delaminations, will produce a "hollow" sound when struck. Basic limitations of this method relative to application to NPP CCBs are that it only can be applied to local and selected test areas because of accessibility constraints and the large size of CCBs (i.e., thicknesses up to several meters), it is usually effective for defects not exceeding the concrete cover depth, and it may miss small delaminations.

Infrared thermography

Infrared thermography is based on the theory of heat transfer. Since subsurface anomalies in a material affect heat flow through the material, heat transfer sensed through surface temperature variations can be used to locate subsurface voids, delaminations, or other defects. The basic equipment includes an infrared scanner head and detector that is capable of measuring small temperature variation to 0.1°C. The magnitude of the temperature difference between deteriorated and sound areas provides an indication on the depth of the defect. The advantage of this method is its capability to cover a large concrete surface area within a short period of time. The primary limitation of the method is that in order to execute this inspection method, 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 the accuracy of the readings.

Magnetic

The principle of magnetic induction forms the basis for this method and is applicable to ferromagnetic materials only. The method is useful in measuring the thickness of the concrete cover, and determining the size of embedded steel reinforcement and its spacing. The accuracy of rebar sizing is better than 90% when the equipment is properly calibrated. The method has a maximum range of about 90 mm of concrete depth and its accuracy for normal concrete cover thicknesses (e.g., ≤50-mm) is on the order of ~3 mm. Basic limitations of this method are that for best results the spacing between two adjacent reinforcement bars must be greater than the concrete cover, and since the method is based on the induction principle, the results are affected by anything that affects the magnetic field within the range of the instrument (e.g., electrical cables, metal tie wires, and iron content of cement).

Acoustic

Acoustic or stress wave propagation methods encompass all forms of testing based on the transmission and reflection of stress waves. Acoustic wave transmission can be used to obtain information about the physical condition of concrete structures. They are used either to characterise the properties of the concrete by wave speed measurements or to locate and identify discrete defects or objects in the concrete by transmission and reflection of stress waves. The latter may be referred to as the pulse-echo technique. Acoustic measurement techniques include surface and compression wave measurement, ultrasonic pulse-echo, impact echo, and other related techniques.

Ultrasonic pulse velocity methods are commonly used to examine homogeneous materials such as metals. The methods are based on the principle that the velocity of sound in a material is related to its elastic modulus and material density. Basic components of the equipment include a means for producing and introducing a pulse into the material examined, and a means of accurately measuring the time required for the pulse to travel through the material to a receiver (Fig. 4.1) [4.7]. The condition of the material is assessed through determination of the pulse velocity and the amplitude of the stress wave at the receiver. When displaying the travel time of the sound waves between the generator and receiver versus the location, there will be a deviation in the curve at the position of the subsurface defect. Ultrasonic pulse velocity equipment for examination of concrete materials is essentially the same as that used for metallic materials except a 30 to 200 kHz transducer is used instead of a 0.1 to 25 MHz transducer because of the greater attenuation characteristics of concrete materials. By using this method 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. For detecting internal structural

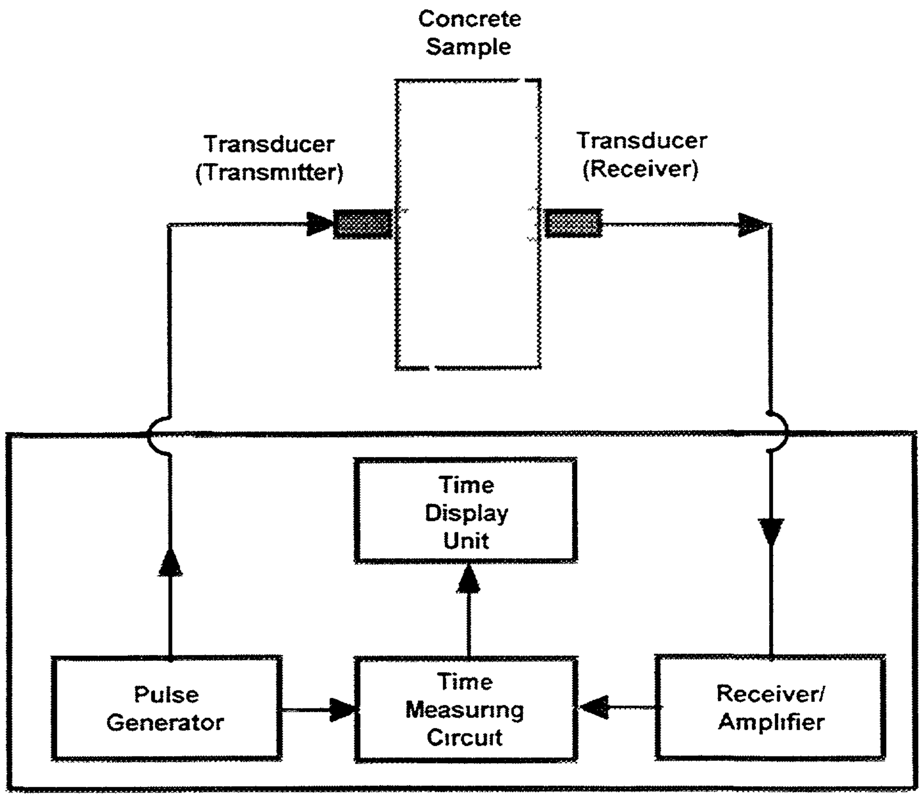

FIG 4 1 Schematic diagram of pulse velocity test circuit

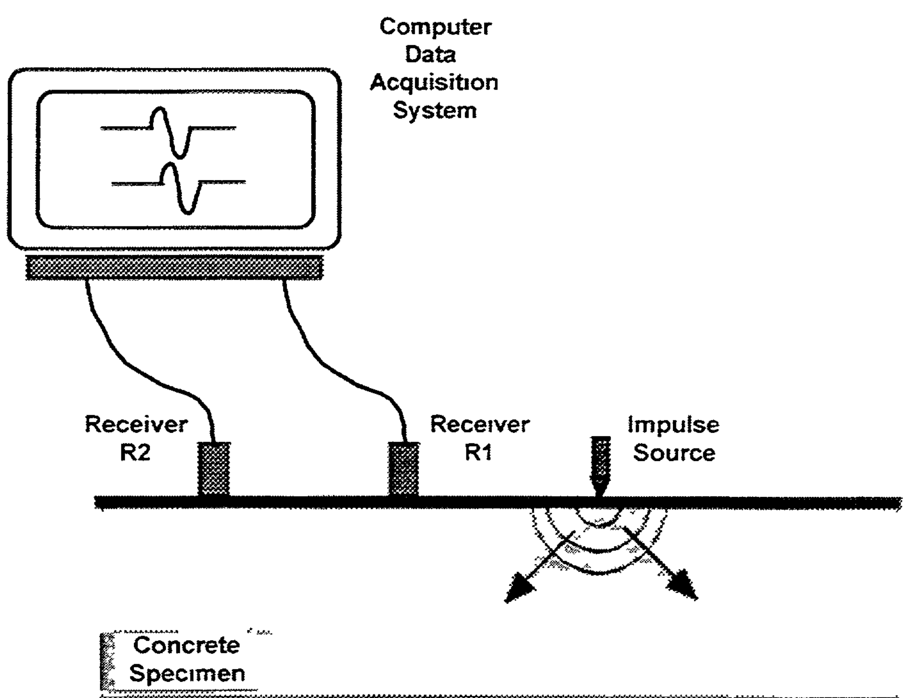

FIG 4 2 Spectral analysis of surface waves (SASW) test setup changes, the method is limited by segregation/inhomogeneity of the concrete and quality of the acoustical contact. For strength and related properties, the test must be calibrated to the specific concrete as the results are influenced by aggregate size, type, and gradation; cement type; water-cement ratio; admixtures; degree of compaction; curing conditions and age of the concrete; acoustical contact; concrete temperature; moisture content; size and shape of specimen; and presence of reinforcement. Despite the dependence on so many variables, the ultrasonic pulse velocity method can be used effectively. It is most useful when carrying out comparative surveys of concrete quality in or between similar concrete structures. Changes in signal amplitude or attenuation characteristics can be used to indicate changes in material properties that occur with time (e.g., detection of the action of frost or alkali-silica reactions through measurement of frequency-dependent attenuation of direct transmission ultrasonic pulses).

Spectral analysis of surface waves (SASW) has recently found use in testing concrete and in geophysical surveys. A mechanical impact on the surface of the concrete structure is used to generate surface waves that are picked up by two transducers placed at fixed distances from the impact source (Fig. 4.2). The transducers are placed in line with the impact source and their spacing is determined by the depth to be measured. In the case of a massive concrete element this may require access to a large surface area. Surface wave velocity is affected by the material properties, and by analysing the relationship between velocity and frequency, it is possible to obtain a profile of the velocity with depth (i.e., dispersion curve). The depth to which the surface waves are affected by the material is dependent on the wave frequency, with lower frequency waves affected by material stiffness at greater depths. The method is particularly well suited for testing layered systems and for determining the depths of foundations or the condition of underlying material.

Ultrasonic pulse echo involves the use of transmitting and receiving transducers that are normally placed close to each other on the testing surface (Fig. 4.3). Rapid-hardening cement is used to effectively attach the transducers to the concrete surface. The transmitter also may be designed as a receiver. The pulsed signal inputs may be produced by a piezoelectric transducer. 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 to reflectors (e.g., cracks or large voids). Because of the heterogeneity of concrete, it may be difficult to distinguish actual defects. Large aggregates have a significant scattering effect on the signals thus restricting the method to relatively low frequency inputs. Modern pulse-echo equipment has achieved some success in this respect by using transducer "arrays" of up to twelve transducers. Signals are transmitted and received between combinations of these transducers and by averaging the response it is possible to more clearly define relevant reflectors. The pulse-echo method is the acoustic method most similar to conventional ultrasonic testing such as used for examining metallic materials. It has the potential to locate and identify discrete defects or objects if sufficient focusing can be achieved by the transducers. Concrete made with aggregates of 16-mm maximum size have been tested successfully and cylindrical voids with a 100-mm diameter can be detected at depths to 600 mm. Reflected signals from large planar surfaces can be detected at depths to 1300 mm.

The impact-echo method involves striking the concrete surface with a small ball of given diameter to produce a transient stress wave that propagates into the concrete [4.8]. The impact source and receiving transducer are placed adjacent to each other on the concrete surface. Some control of the input can be achieved by varying the size of the impactor thus determining the frequency of the input signals (i.e., smaller diameter impactors create higher

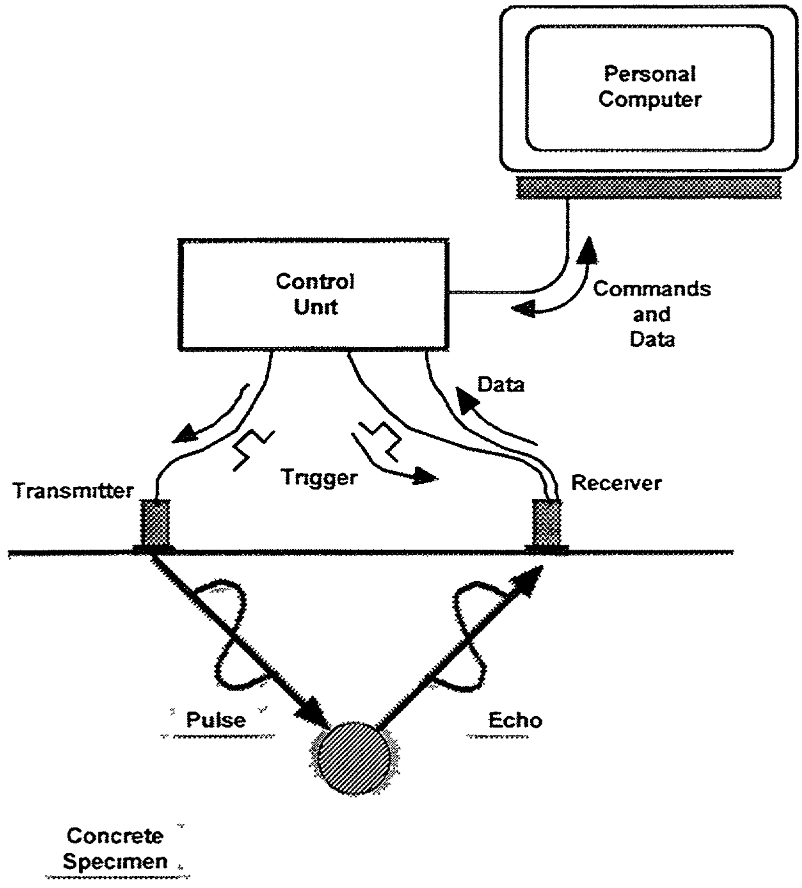

FIG 4 3 Schematic of ultrasonic pulse-echo test setup

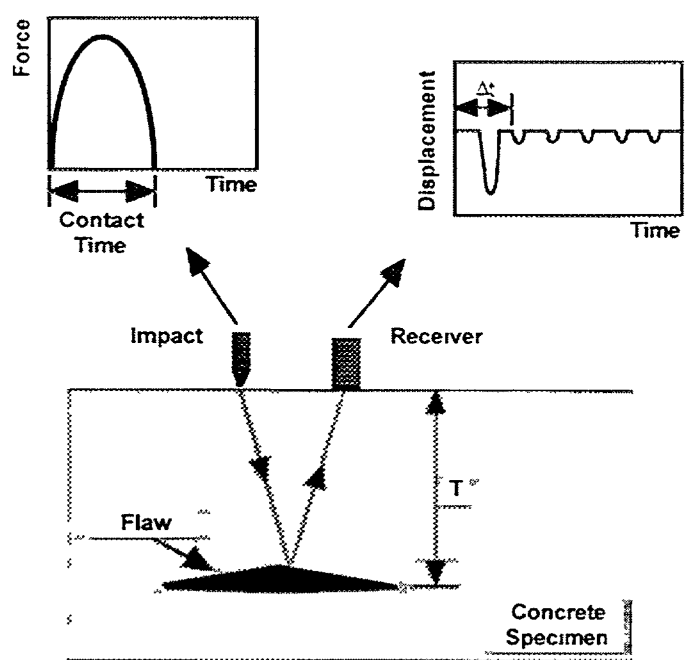

FIG 4 4 Principle of impact-echo system frequency waves that are more sensitive to small reflectors at shallower depths, and vice versa). The sound pulse or compression wave is reflected from the back side of the concrete element, internal reflectors (e.g., cracks), or from other objects that may cause changes in the acoustic impedance and material density along the path of pulse propagation (Fig. 4.4). Information is obtained related to the complete, or a significant volume of the concrete (i.e., the signal can not be focused as with the ultrasonic pulse-echo method). In this respect it can be seen as a global measuring technique which may be an advantage, but it also might complicate the process of interpretation. Testing normally involves a study of the compression wave only and frequencies usually are below the ultrasonic level. The response signal is analysed in the frequency domain using a Fast Fourier Transform technique. Although in principle the impact-echo technique may be used for thicknesses up to several meters, it normally is used in concrete structures up to one meter in section. Impact echo is most effectively used for testing large concrete areas and if the geometry of the structure is quite simple, the analysis procedure is also relatively simple. Since the signal input is by mechanical impact, the testing can be carried out quickly without the need for a coupling medium.

Radiation/nuclear

Radiation and nuclear techniques involve radiometry, radiography, and neutron source. Of these methods, radiography is well established and the method most often used to examine the quality of construction or materials in concrete (e.g., prestressing tendons). The basic system consists of a radiation source (gamma ray) emitting a beam through the test article and a photographic film placed on the opposite side of the test article from the source (Fig. 4.5). Since a high density medium absorbs a greater amount of emitted energy, the density of the material determines the energy being absorbed by the film. A two-dimensional projection of the area being inspected is displayed on the film. Other factors that can affect the intensity of radiation passing through the test article include its thickness and absorptive characteristics. Although g-scintillation can be used in members greater than 1-m thick, most systems are capable of detecting small voids in members up to about 700-mm thick. The present sensitivity of the technique probably is not sufficient to detect voids in tendon ducts. 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. The count or count rate is used to measure the specimen dimensions or physical characteristics (e.g., density and composition). Neutron methods consist of an emission source and a gamma ray collection and counting system. The method can be used to measure the moisture content in a structure. Primary limitations of the most commonly used of these methods, radiography, are that radiation protection has to be observed while applying this method, personnel must be licensed or certified, the concrete structure must be accessible from both sides, and concrete sections are generally limited to 1 m or less in thickness.

Tomography

Tomography is an advanced nondestructive evaluation method based on radiography that is used to examine concrete structures for cracks, voids, and other internal defects. The advantage of this method over radiography is that it provides the possibility of internal inspections through development of three-dimensional displays from a series of reconstructed digitised detector measurements obtained from planes or slices through the thickness of the object inspected. This is accomplished by conducting a large number of two-dimensional examinations of the structure and analyzing the results with sophisticated computer software

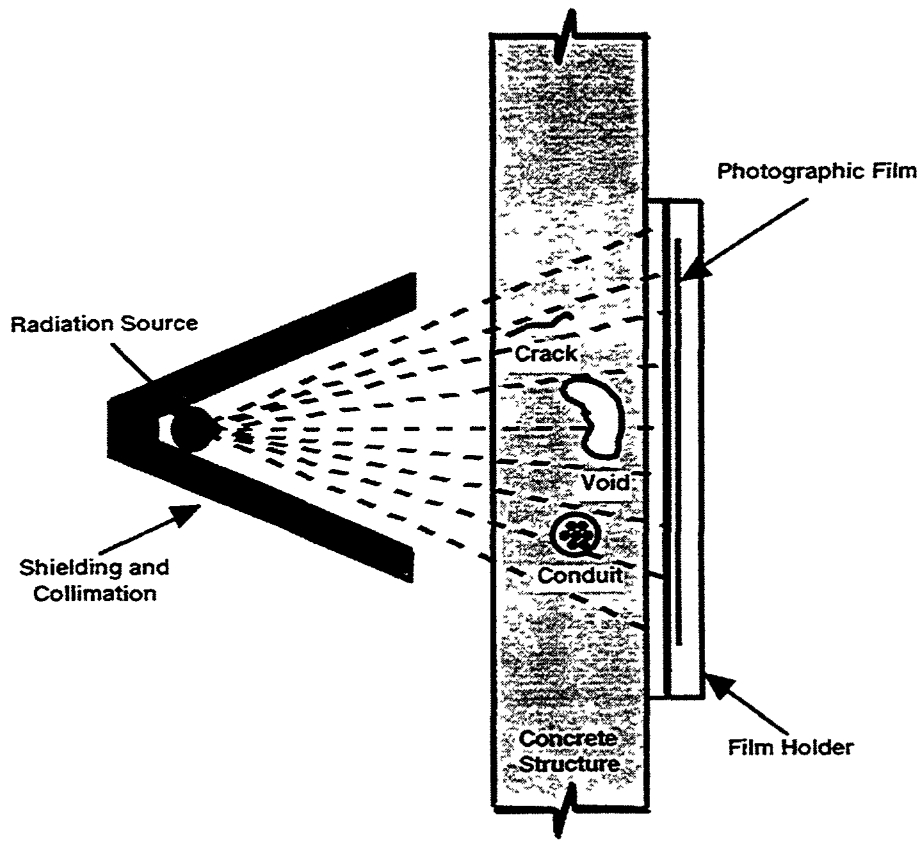

FIG. 4.5. Schematic of radiography method.

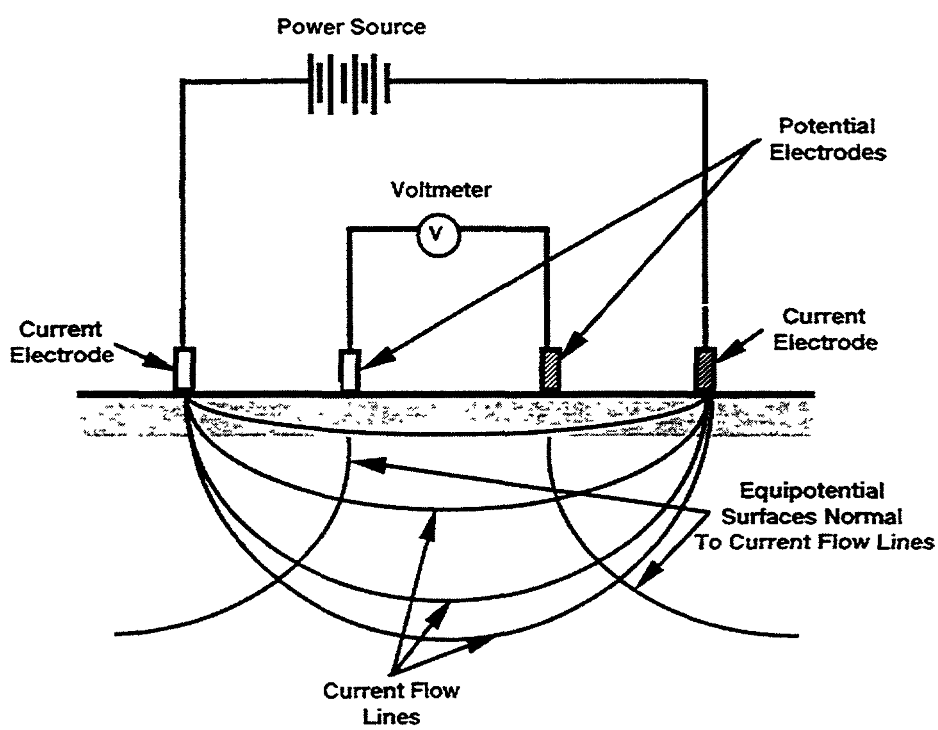

FIG. 4.6. Schematic of four-electrode method for measurement of concrete resistivity.

that has been developed especially for this application. The primary limitations of this method are that if a complete radiographic examination is not possible because of geometrical boundary conditions, additional calculations must be made for those areas, and the method is presently costly to perform. Acoustic- and stress wave-based methods also have been developed.

Rebound hammer

The rebound hammer is one of the most commonly used of the nondestructive evaluation methods. The method uses the rebound distance (measured on an arbitrary scale) of a spring-loaded weight impacted against the concrete to estimate quality or compressive strength of the in situ concrete. The primary usefulness of the rebound hammer is in assessing concrete uniformity in situ, delineating zones (or areas) of poor quality or deteriorated concrete in structures, and indicating changes with time of concrete characteristics. The effectiveness of the rebound hammer method is often enhanced through combination with other techniques such as ultrasonic pulse velocity measurements. Primary limitations of this method are that the test results only measure surface characteristics and test results may be influenced by parameters such as the test surface smoothness and moisture content, orientation of the hammer during impact, type of cement used, and type of aggregate; and application-specific calibration curves have to be developed to provide reasonably accurate (e.g., $\pm 15\%$) compressive strength results. Surface treatments may also exclude direct application of the technique.

Modal analysis

Modal analysis is used to obtain response data from a structure under external or internal stimulus. Basically, 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). Mathematical modeling is used to compare field measured and theoretically obtained vibration modes. Two different approaches are used to obtain the dynamic response of a structure: (1) theoretical approach – involves use of models to describe a structure's physical characteristics (e.g., mass, stiffness, and damping properties), structure's behaviour as a set of vibration modes, and structure's vibration response under given excitation conditions; and (2) experimental approach – response of structure to a certain excitation mode is determined. The response simulation could be used for NPP concrete structures to evaluate the structural capacity to provide safe-shutdown in earthquake situations. The primary limitation of this method is that modal analysis has to be executed by experienced staff and a baseline response is required for comparison.

Radar

Ground-penetrating radar is the electromagnetic analogue of sonic and ultrasonic pulse-echo techniques and is well developed in the geophysical field. It has been adapted and can be used in its various forms to obtain information from concrete structures and their foundations and substrate. The use of radar for inspection of concrete structures is relatively common in some countries. One of the advantages of radar is that the antenna used for scanning does not require contact with the test surface and large areas can be scanned rapidly. Short pulses of electromagnetic energy are transmitted through the structure and the energy is reflected by boundaries between layers of different dielectric properties. The receiving antenna and readout circuitry indicate the depth to these layers. The method has the ability to

TABLE 4.1. METHODS TO ASSESS CONCRETE PROPERTIES OR CHARACTERISTICS

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)	Pullout 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															
Alkali-Silica Reaction											X									
Bleeding Channels											X									X
Cement Content											X									
Chemical Composition											X									X
Chloride Content					X	X														
Compressive Strength			X			X						X	X			X			X	
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									
Pullout Strength													X							
Aggregate Quality											X									X
Freeze/Thaw Resistance											X									
Soundness						X									X			X		
Splitting-Tensile Strength						X														
Sulfate Resistance											X									
Tensile Strength			X			X														
Concrete Uniformity											X					X				X
Voids						X								X	X		X	X	X	X
Water-Cement Ratio											X									

(N) = nondestructive method, (S) = semidestructive method, and (D) = destructive method.

TABLE 4.2. 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
Surface Cracks (L, P)							X		X				X
Ultimate Strength (T)											X		
Wall Thickness (L)										X		X	
Yield Strength (T)											X		

*L = liner, P = protective media, R = mild steel reinforcement, and T = tendon.

penetrate considerable depth into concrete structures. A major use of radar inspection is detection of steel reinforcement, other embedments, and voids. The ability to detect the depth of reflectors such as reinforcing bars or tendon ducts is dependent on knowledge of the dielectric properties of the concrete, which in turn is dependent on the moisture level. In thick concrete structures the moisture variations can be considerable even decades after construction. Closely spaced reinforcement near the concrete surface tends to disrupt radar signals and screen deeper lying objects of interest like prestressing tendon ducts. At present the tendon ducts can be detected in concrete to a depth of at least 300 mm, provided the concrete is not too moist (i.e., high moisture levels hinder radar signals from penetrating concrete, particularly if the water contains salts that increase conductivity). The presence of moisture may be an advantage, however, in trying to detect leaks in water-retaining structures such as dam walls or water-proof membranes. Currently the primary limitation of the method is the resolution capability, but there are on-going programmes to develop signal processing tools to overcome this limitation. Many significant developments in system hardware, data analysis, and enhancement software have been reported. The development of antenna with frequencies in the 1-5 GHz range is on-going and will improve resolution and increase the capability of the technique to detect objects that lie behind near-surface reinforcement mats. Other developments include methods of measuring moisture profiles and determining the dielectric constant on site. At the moment this has to be estimated, although there is some equipment that can help in establishing this information.

Instrumentation

The use of instrumentation in NPP CCBs currently 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 the instruments are normally cast into the structure at the time of construction. Usually the first structure of a new design or the first of multiple structures at a new site is instrumented extensively. Subsequent structures normally are not instrumented or contain only nominal instrumentation. While retrofitting of instruments for long-term performance monitoring is currently unusual in the nuclear industry, it is possible, though locally destructive, and is fairly common for dams in the hydroelectric power industry.

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, and precision level surveying. These instruments are useful in detecting and monitoring overall and local stress, strain, deformation, and temperature change in CCBs. Particular results may be helpful in assessing concrete creep and loss of force in prestressing systems – changes that are not detectable by common visual inspection techniques.

The general advantages of instrumentation are its ability to provide frequent quantitative measurements of structural performance, be automated, obtain results without plant outages, and detect changes at a level below those detectable by visual and many other nondestructive test methods. In terms of general limitations, instruments only provide data at discrete points in the structure, redundancy of instruments is required to protect against individual instrument failure or damage, reference conditions apply when the instrument is placed in service and read (which may not reflect the full history of the structure), and post-construction installation will cause local damage to the structure. Specific usage, advantages and limitations of various instruments are described below.

Vibrating-wire strain gauges

The vibrating wire strain gauge may be installed either as a single reinforcing bar with large end anchor plates embedded in the concrete, or as a "sister bar" attached to the reinforcing steel within the concrete. A key component of the gauge is a tensioned steel wire within the body of the vibrating wire transducer that forms the core of the gauge. An electrical coil around this wire electromagnetically "plucks" the wire and monitors its frequency of vibration which changes with tension in the wire (e.g., change in length per unit length). A special readout controls the coil and converts the feedback into a strain reading. These gauges can loose accuracy with time due to zero drift, and radio or electromagnetic interference in the proximity of the installation can affect results. Gauges will not properly reflect strain in the concrete if they are not properly dimensioned to avoid the "inclusion effect." Typical accuracy is in the range of 5 to 50 microstrain.

Thermocouples

Thermocouples consist of two wires of dissimilar metals that are joined at the ends to form a complete circuit. The 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. The thermocouple is used to 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. For copper-constantan thermocouples, accuracy is about $\pm 0.5^{\circ}C$.

Pendulums

Hanging pendulums monitor the horizontal movement (e.g., radial and tangential displacements) between two points in the CCB. A small diameter stainless steel wire hangs from an upper anchor point and connects to a weight below a reading table at the 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 micrometers, optical micrometers, and industrial optical vision systems. The instrument requires a clear vertical path between its upper and lower points, and protection against draughts, other environmental effects, and accidental physical damage. Accuracy is very dependent on the reading method and can range from 0.003 to 0.5 mm.

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 micrometers to linear-variable-differential transformers. Limitations and accuracies are similar to the pendulum above.

Load cells

Load cells (or dynamometers) are inserted between structural components to measure load transfer within the structure. Various forms 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. [4.6]. Accuracy ranges from ± 2 to 10% of load.

Liquid level gauges

Liquid level gauges incorporate a liquid filled tube between two reservoirs. The relative elevation between the end reservoirs is determined by manometer readings or pressure measurement in the reservoirs. Liquids used range from water to oil to mercury. The 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.

Humidity gauges

Two types of humidity sensors commonly used during containment integrated-leakage-rate testing are the chilled-mirror dew point hydrometer and the lithium-chloride electrical resistance hydrometer (Dewcell) [4.12]. The chilled-mirror dew point hydrometer uses photoelectric cells to detect condensation on a mirror whose temperature is controlled by a thermoelectric heat pump. A platinum-resistance temperature detector senses the mirror surface temperature which is a direct measure of the dew point. The dewcell maintains a 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 nonconducting tube contains the aqueous solution. It is heated by two parallel helically-wound gold wires with a potential of 25 volts (ac) maintained between them. A resistance bulb embedded in the glass tape measures the element temperature which is then converted to dew point by means of the unique relationship between the vapour pressure of the saturated salt solution and its temperature. Both sensors do not function reliably in the presence of condensing moisture without some modification.

Precise level surveying

Precise (usually first or special 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.

4.1.1.2. Destructive testing

Destructive testing can be utilized to determine concrete strength, density, and quality; locate voids or cracks in concrete; locate steel reinforcement and determine depth of concrete cover; and detect corrosion of steel reinforcing materials. Destructive testing techniques include (1) air permeability, (2) break-off, (3) core testing, (4) probe penetration, (5) pullout, (6) chloride-ion content, (7) carbonation depth, and (8) petrography.

Air permeability

This method is used in situ to obtain values of air permeability of concrete structures. These values can give indications of the concrete compressive strength, resistance to carbonation, and the potential for steel reinforcement corrosion (i.e., penetration of aggressive ions). The test is conducted by first drilling a small diameter hole into the concrete and sealing it with a rubber plug. Air is extracted from the sealed hole by means of a needle that is inserted through the rubber seal and connected by a rubber hose to a mercury manometer and vacuum pump. The rate of recovery of air to the hole provides a value of the concrete air permeability. Primary limitations of this method are that sealing of the drilled hole must be complete, and the results may be affected by the number of neighboring holes, diameter of test hole, and moisture in the concrete.

Break-off

The break-off method is used in situ to indicate concrete compressive strength. To perform this test, a specimen of 55 mm diameter and 70 mm deep is formed using a plastic cylinder placed into the fresh concrete, or drilling a core with the same outer dimensions in existing concrete. A load cell is placed into a circular groove at the top of the concrete 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 the concrete strength by using calibration curves. Limitations of this method are that it cannot be used with concrete mixes having maximum aggregate sizes exceeding 19 mm or concrete structures having sections less than 100 mm thick.

Core testing

Removal and evaluation of concrete core samples from structures provides a direct method for examination of the concrete. Requirements for obtaining concrete samples to provide a sufficient number of specimens for statistical evaluations are generally described in national codes and standards for building and construction. When cores are removed from areas exhibiting distress, strength tests and petrographic studies (discussed later in this section) can be used to investigate the cause and extent of deterioration. Other applications of concrete cores include calibrations of nondestructive testing devices, and down-hole cameras can be used to examine the interior of the structure in locations where concrete cores were removed. Primary limitations of the method are with respect to the number of samples that must be removed to meet requirements related to ensuring that the probability of obtaining a strength less than desired is below a certain level, the results can be influenced by several factors (e.g., aggregate size, core diameter and slenderness ratio), and areas where cores are removed may require repair.

Probe penetration

Probe penetration tests estimate concrete compressive strength, uniformity, or general quality through measurements of the resistance of concrete to penetration of a steel probe that is driven by a given amount of energy. Compressive strength is determined by using calibration curves. Advantages of the method are that it is relatively simple to operate and the results correlate fairly well to concrete compressive strength. Primary limitations of this method are that the thickness of the specimen to be tested has to be at least three times the depth of the penetration, the method should not be applied within about 200 mm of specimen edges or oth :r tests, and aggregate size and hardness influence results.

Pullout test

Originally known as cast-in-place pullout, this test is performed by using a hydraulic device to pull an embedded metallic disc from concrete. The concrete compressive strength is related to the pullout force through calibration curves. Recent developments have eliminated the requirement that the pullout inserts be cast into the specimen. Primary limitations of the test are that the results are affected by the size of coarse aggregate, and a correlation relationship between pullout strength and compressive strength is required. Also some repair may be required.

Chloride-ion content

Determination of the concrete chloride-ion content is an important aspect of the analysis of concrete structures relative to the potential for corrosion of embedded steel reinforcement. Two of the most commonly used methods for determination of chloride contents in concrete are the water-soluble and total-chloride tests. 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 the cement or concrete. 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 analyzed. Other methods that require a powder sample include X ray fluorescence, gas chromatograph, Quantab chloride titrator strips, specific ion electrode, spectrophotometer, and argentometric digital titrator. Primary limitations of these methods are that they 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. A potential technique to determine the amount of chlorides that was present in the mix is to obtain (if possible) a baseline for the chloride ion content in an area of the structure where chlorides from external sources are known not to be present.

Carbonation depth

Depth of carbonation can be easily determined either in situ or in a laboratory by treating a freshly broken concrete surface with phenolphthalein. The carbonated portion will be uncolored. Periodic determinations can be used to establish the rate of penetration. This test often is performed in conjunction with chloride-ion content determinations. The primary limitation of this method is that it requires exposure of a fresh concrete surface for each test.

Petrography

Petrographic examinations of samples of hardened concrete removed from existing concrete structures such as CCBs can provide valuable information for use in an ageing management programme. Several purposes for which petrographic examinations of these structures may be conducted include detailed determination of the condition of the concrete in the structure; determination of causes of inferior quality, distress, or deterioration; determination of whether the concrete in the structure was or was not as specified; description of the cementitious materials matrix (e.g., kind of binder, degree of hydration, nature of hydration products, and presence of mineral admixtures); determination of the presence of alkali-aggregate reactions; determination if the concrete has been subjected to chemical attack or early freezing; determination of the nature of the air void system; and survey of the structure relative to its safety. The primary disadvantage of petrographic examinations is that they require removal of samples from the structure for test and evaluation.

4.1.2. Reinforcing steel

Assessments of mild steel reinforcing are primarily related to determining its characteristics (e.g., location and size) and evaluating corrosion occurrence. Only evaluation of the occurrence of corrosion will be addressed in this section as magnetic methods associated with determining characteristics of embedded steel reinforcing were addressed in the previous section. Methods available for corrosion monitoring and inspection of steel include visual inspection, mechanical and ultrasonic tests, core sampling with chemical and physical testing, electrical methods (four-electrode and half-cell potential), rate of corrosion probes, and galvanostatic pulse. Of these methods, only the electrical, rate of corrosion probe, and galvanostatic methods will be discussed as the other methods already have been addressed. These methods are semi-destructive.

4.1.2.1. Four-electrode

The four-electrode method measures the resistivity of concrete and relates it to the probability of reinforcement corrosion. Alternating current is passed through the outer electrodes of four linearly arranged and equally spaced contact points and the potential drop between the two inner electrodes is measured (Fig. 4.6). The resistivity is then determined (in ohm-cm). Depending on the value of resistivity measured, corrosion is unlikely, corrosion will probably occur, or corrosion is almost certain to occur. Results may also provide input for assessing the relative concrete quality as sensed by amount of moisture present. Limitations of this method are that the resistivity measurements are obtained relatively close to the concrete surface, and when probe spacings are increased to allow evaluations at deeper concrete depths, the steel reinforcement may interfere with results obtained.

4.1.2.2. Half-cell potential

Potential measurements at a number of locations on the concrete surface using a reference half cell (e.g., copper-copper sulphate) connected to the steel reinforcement can be used to indicate the likelihood of corrosion occurrence (i.e., >90% probability of no corrosion, corrosion activity is uncertain, or >90% probability that corrosion is occurring) (Fig. 4.7). The surface of the concrete 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 exhibiting potential corrosion can be readily identified. Modified types 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 are that neither the magnitude nor rate of corrosion are provided, surface coatings or coated steel reinforcement present problems, measurements are affected by temperature and moisture, electrical continuity is required, and 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 areas on concrete surfaces where risk of reinforcement 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 the reinforcement, or to monitor the effect of cathodic protection systems.

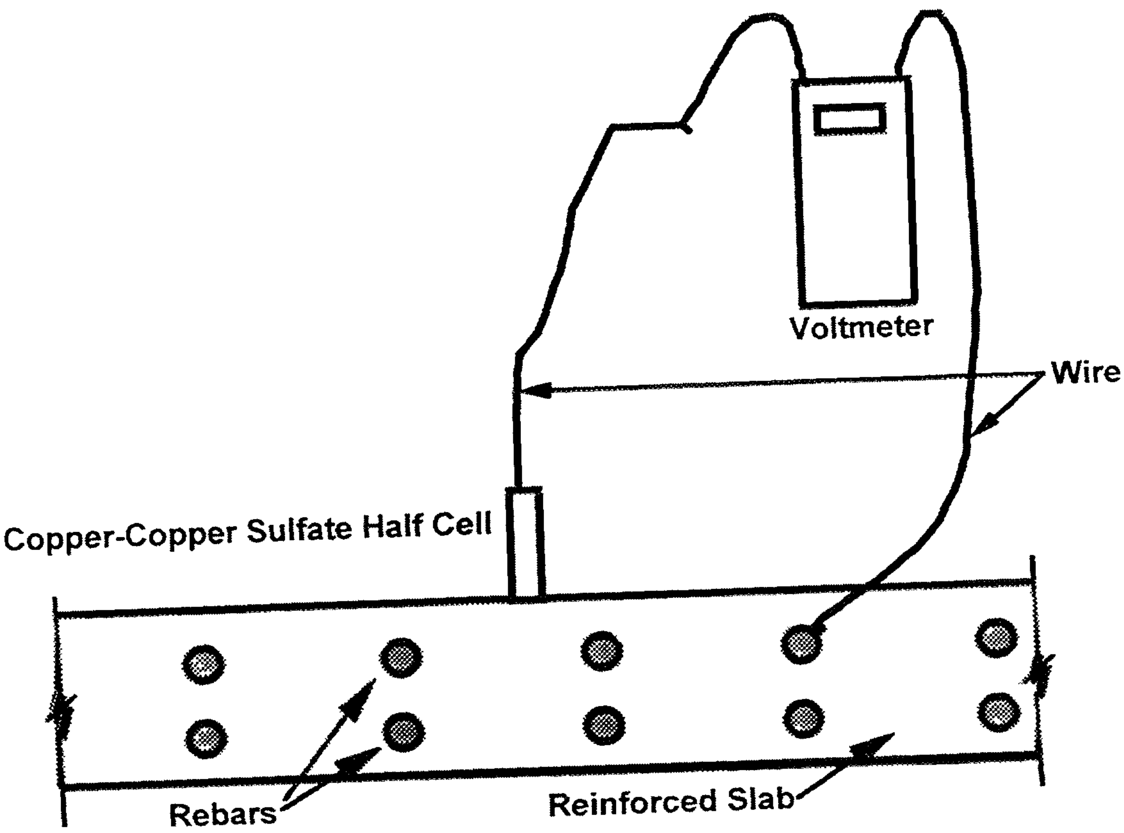

FIG 4 7 Schematic of copper/copper sulfate half-cell potential system

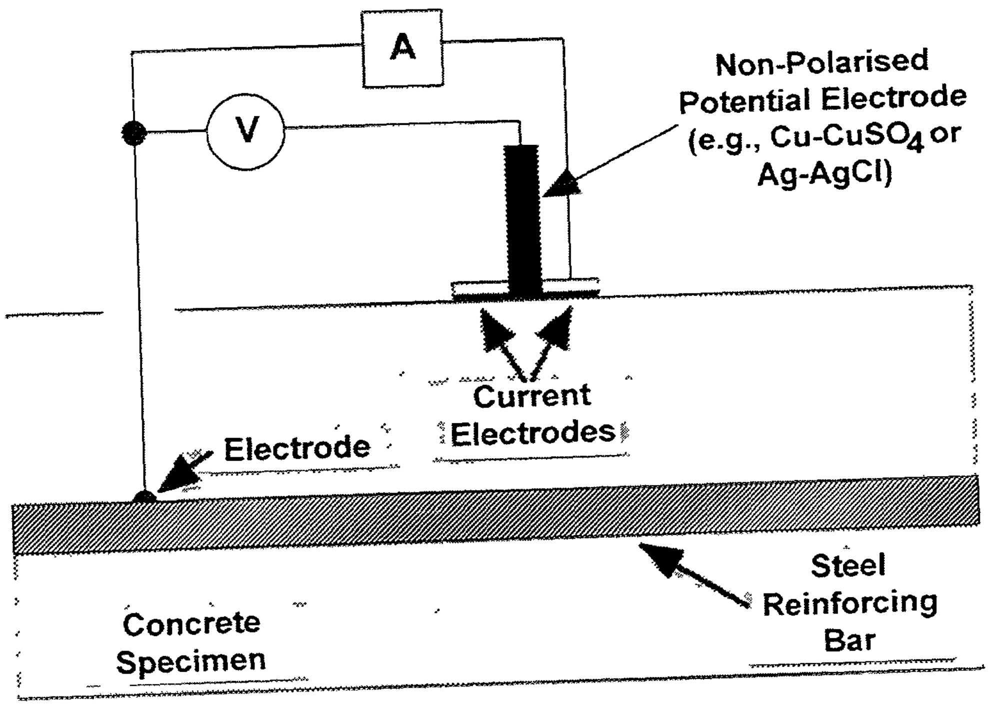

FIG 4 8 Schematic of setup for a galvanostatic pulse measurement

4.1.2.3. *Rate of corrosion probes*

Probes embedded into the concrete can be used to indicate the rate of corrosion. Two primary types are available: (1) two to three short sections of steel wire or reinforcement in conjunction with polarisation techniques, and (2) steel wire or hollow cylinder to provide cumulative rate of corrosion data from periodic measurements. The primary limitation of this technique is that it requires some excavation of the concrete to insert the probe(s). As a consequence, rate of corrosion probes have found primary application in evaluation of the effect of rehabilitation procedures.

4.1.2.4. *Galvanostatic pulse technique*

The galvanostatic pulse technique is a polarization method that has recently been developed for in situ application and can be used for detecting corrosion in wet and anaerobic environments [4.9-4.11]. The method set up is similar to the half-cell potential method and involves use of a counter electrode and reference electrode that are placed on the concrete surface above the reinforcement (Fig. 4.8). A short-time anodic current pulse is impressed galvanostatically from the counter electrode which in turn shifts the reinforcement potential with the shift recorded by a data logger. The reinforcement is polarized in the anodic direction relative to its free corrosion potential. The extent of polarization depends on the corrosion state. The reinforcement is easy to polarize in the passive state, as illustrated 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 when the half-cell potential method is used. 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 the corrosion current. If the area of the polarised reinforcement is known, then the corrosion current can be converted to a corrosion rate. It is possible in this way to estimate corrosion rates in the case of general corrosion, but not in the case of local pitting corrosion. The galvaonstatic pulse technique measurements take considerably longer to execute than the half-cell potential measurements and require an experienced person to perform the technique.

4.1.3. Prestressing systems

The primary potential ageing mechanisms associated with the prestressing systems used in NPP CCBs are concrete creep, and stress relaxation and corrosion of the prestressing steel. The first two of these mechanisms will result in loss of prestressing force, and the third one in a reduction in the load-carrying capacity. Inspection methods associated with detection of both of these manifestations are discussed in the balance of this section. Although both grouted and nongrouted prestressing systems have been used in the construction of CCBs, only the nongrouted systems will be addressed because of the difficulties associated with inspection of grouted tendon systems.

4.1.3.1. *Prestressing force determinations*

Loss of prestressing force with time is not completely predictable and should be measured at regular intervals to ensure that the CCB retains adequate capacity to resist accident pressure and coincident design loads with acceptable margins. The CCB design establishes the minimum prestressing force necessary to maintain the concrete in compression (full prestressing), with a reasonable margin, under the postulated loads. Determination of the level of prestresing force is performed routinely as part of the inservice inspection programme that is mandated by the regulators in many of the 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).

Although several prestressed CCBs have incorporated load cells to monitor the prestressing level (loss of prestressing force with time), most CCBs utilise liftoff load determinations to indicate the level of prestressing force. The tendon end anchorage force is determined by measuring the jacking load required to lift the tendon anchorhead clear of the shim stack or bearing plate. The primary limitation associated with this method is that it is applicable only to ungrouted tendon systems.[3] Also, there may be some uncertainty associated with determination of the actual point of liftoff of the anchorhead.

4.1.3.2. Mechanical tests on tendon materials

Associated with some ungrouted tendon inservice inspection programmes is a requirement to remove representative samples of the tendon materials to monitor for any ageing effects, notably corrosion. After removal, sections of the wire or strand, depending on tendon type, are taken from each end and the midlength of the selected tendons, cleaned, 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.

4.1.3.3. Tests on grease

In order to provide a corrosion protection medium to the ungrouted tendons, the tendons are coated or the space between the post-tensioning tendon and metal sheath generally is filled with a specially-formulated grease. As part of some of the inservice inspection programmes for the tendons, samples of the grease are taken at both ends of the tendons selected for examination and 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.

4.1.3.4. Anchorage inspection

The end anchorage system (e.g., end cap, exposed bearing plate surfaces, and anchorheads) is examined visually for evidence of cracking, distortion, major corrosion, and broken or protruding wires. Visual inspection also includes examination of the concrete adjacent to the bearing plates for cracking or spalling that would be indicative of a bearing failure. The primary limitations of this procedure are related to the limited sample size and only visible locations can be examined.

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[3]Certain plants that utilize grouted tendon systems have fabricated a number of companion ungrouted tendon members for the purpose of monitoring prestressing system performance and investigating corrosion occurrence.

4.1.4. Penetrations

Access to the interior of CCBs is provided through penetrations in the containment shell (e.g., electrical penetrations, piping penetrations, and manways). Since these penetrations are part of the containment pressure boundary, inservice inspections are periodically performed. Primary methods that have been used to demonstrate the integrity of these penetrations include visual examination and local leakage tests.

4.1.4.1. Visual examination

Containment inservice inspections generally require periodic visual examinations of accessible metal surfaces; seals, gaskets, and moisture barriers; dissimilar metal welds; and pressure-retaining bolting. These inspections are intended to detect problems that could adversely affect the structural capacity of the containment and its leaktight integrity. Visual inspection covers detection of corrosion on exposed metal surfaces; weld defects (e.g., cracks); and 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 can not be examined.

4.1.4.2. 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: (1) differential pressure measurements, (2) soap bubble testing, and (3) 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 pressurising the component and applying a thin film of liquid soap to exposed surfaces. Bubbles will occur 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).

4.1.5. Liners

Both non-metallic and metallic liner systems have been used in CCBs. Containment leaktightness is verified periodically through pressure testing. Visual examinations of exposed liner surfaces normally are conducted as part of this procedure. Supplemental testing (e.g., ultrasonic, magnetic particle, and liquid penetrant) is normally performed if abnormal conditions are identified.

4.1.5.1. Integrated leakage-rate test

The integrity of both metallic and nonmetallic liners used to form the pressure boundary of many CCBs is evaluated by pressurising the containment with air to a preestablished level. Information on this method was provided previously under Section 4.1.1.1.

4.1.5.2. Visual examination

A general visual examination of accessible liner surfaces provides a key component of an ageing management programme as it can uncover evidence of possible degradation that could affect containment structural integrity or cause unacceptable leakage. Corrosion of uncoated liner areas, due to the presence of 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 defects of the weld material and the base metal adjacent to the edge of the weld. However, without removal of material, visual examinations cannot inspect inaccessible areas that may be vulnerable to corrosion such as portions of the liner embedded in concrete.

4.1.5.3. Ultrasonic test

Ultrasonic testing is a nondestructive method in which beams of high-frequency sound (0.1 to 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. Limitations of the method are that at least one surface has to be accessible, experienced personnel are required, discontinuities just below the surface may not be detectable, and parts that are rough or thin are hard to inspect.

4.1.5.4. Magnetic particle test

Magnetic particle testing 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, are detected by spraying ferromagnetic particles onto the surface, with some of the particles gathering and being 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. Limitations of the method are that it can only be used on ferromagnetic materials, 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, and extraneous magnetic fields (e.g., from power cables) influence the test results.

4.1.5.5. Liquid penetrant test

Liquid penetrant testing can be used to reveal discontinuities that are open to the surfaces of solid and essentially nonporous materials. Indications of a wide spectrum of flaw sizes can be found regardless of the configuration of the test article or the flaw orientation. Liquid penetrants applied to the surface of the test specimen will seep into minute surface openings due to capillary action. After allowing time for penetration, excess penetrant is removed and a developer is applied to enhance the visibility of any flaws present. Limitations of the technique are that it can only detect surface flaws, and results are affected by surface roughness and porosity.

4.1.6. Waterstops, seals and gaskets

Waterstops at concrete joints, and seals and gaskets between adjacent components make a significant contribution to the leaktightness of the CCB. These elements are primarily constructed of metal or elastomeric compounds. During operation, the elastomeric components are subject to degradation by exposure to oxygen, ozone, microorganisms, ultraviolet radiation, radiation, elevated temperature, and stress. Each of these mechanisms act to cause depolymerisation of the elastomer that manifests itself in increased hardness, increased brittleness, dimensional shrinkage, loss of adhesion, and formation of cracks. Visual inspections are effective in identifying potential problem areas. Degradation of these components also can be detected through periodic leakage testing. Durometer testing is utilized as an ageing management tool to indicate the condition of elastomeric materials. The durometer has a calibrated spring that forces an indentor point into the test specimen. An indicator scale provides a direct reading of the hardness that can be related to the suitability of the elastomer material for its intended application.

4.1.7. Protective coatings

Protective coatings are applied to metallic liners, structural steel, and concrete materials to provide barriers to potential environmental stressors.

The integrity of coatings applied to liners to inhibit corrosion can be evaluated visually, with supplemental testing in suspect areas to measure dry-film coating thickness, bond, and determine if holidays (i.e., uncoated areas) are present. Thickness of liner coatings can be determined using magnetic pulloff, magnetic flux, eddy current, or Tooke gauges. The magnetic pulloff gauge uses a spring that is calibrated to determine the force required to pull a permanent magnet from a ferrous base coated with nonmagnetic film. Magnetic flux gauges relate coating thickness to change in magnetic flux due to differences in distance between the instrument probe and the substrate. Tooke gauges use microscopic observations of a small V-groove cut into the coating film. Locations where the Tooke gauge has been used may require recoating. Coating bond to the substrate can be determined using a cross cut test. In this test six equally spaced cutting edges, with the distance between cutting edges dependent on coating thickness, are used to cut the coating to the substrate. The bond performance is determined by comparing the amount of dislodgement of the coating (peeling) from its backing using reference pictures.

Sealers for concrete are liquids that are applied as a coating to either prevent or decrease the penetration of liquid or gaseous media. Coatings or membranes are different from sealers in that they are applied to concrete in some thickness. Typical sealers, coatings, and membranes that have been applied to concrete are identified in Chapter 5. Evaluation of sealing and coating systems can be done through adhesion measurements by tape adhesion, direct tension (elcometer), or direct peel. Visual examinations are also effective in identifying potential problem areas (e.g., blistering, peeling, delaminations, and voids).

REFERENCES TO CHAPTER 4

[4.1] AMERICAN SOCIETY OF CIVIL ENGINEERS, Guidelines for Structural Condition Assessment of Existing Buildings, ANSI/ASCE 11-90, New York (1991).

[4.2] MALHOTRA, V.M., CARINO, N.J. (Eds.), Handbook of Nondestructive Testing of Concrete, CRC Press, Boca Raton, FL (1991).

[4.3] MATHEY, R.G., CLIFTON, J.R., 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).

[4.4] MALHOTRA, V.M. (Ed.), In Situ/Nondestructive Testing of Concrete, SP-82, American Concrete Institute, Detroit, MI (1984).

[4.5] LEW, H.S. (Ed.), Nondestructive Testing, SP-112, American Concrete Institute, Detroit, MI (1988).

[4.6] DUNNICLIFF, J., Geotechnical Instrumentation for Monitoring Field Performance, John Wiley and Sons, Toronto (1988).

[4.7] NAIK, T.R., MALHOTRA, V.M., "The Ultrasonic Pulse Velocity Method", Chapter 7 in Handbook on Nondestructive Testing of Concrete, (V.M. Malhotra, N.J. Carino, Eds), CRC Press, Boca Raton, FL (1991) 169–188.

[4.8] WIBERG, U., Material Characterization and Defect Detection in Concrete by Quantitative Ultrasonics, KTH, TRITA-BKN, Bulletin 7, Sweden (1993).

[4.9] ELSENER, B., WOJTAS, H., BOHNI, H., Galvanostatic Pulse Measurements — Rapid On Site Corrosion Monitoring, Proc. Int. Conf., University of Sheffield, 1994.

[4.10] NEWTON, C.J., SYKES, J.M., A Galvanic Pulse Technique for Investigation of Steel Corrosion in Concrete, Corrosion Science **28** (1988) 1051-1073.

[4.11] KLINGHOFFER, O., In-Situ Monitoring of Reinforcement Corrosion by Means of Electro-Chemical Methods, Nordic Concrete Research **95** (1) (1995).

[4.12] STONE & WEBSTER ENGINEERING CORPORATION, Containment Integrated Leak-Rate Testing Improvements, EPRI NP-2726, Electric Power Research Corporation, Palo Alto, CA (1982).

5. ASSESSMENT AND REPAIR OF AGEING EFFECTS

Reinforced concrete structures almost from the time of construction start to deteriorate in one form or another due to exposure to the environment (e.g., temperature, moisture, and cyclic loadings) [5.1]. The rate of deterioration is dependent on the component's structural design, materials selection, quality of construction, curing, and aggressiveness of its environment. Figure 5.1 presents the relationship between concrete performance and time. Termination of a component's service life occurs when it no longer can meet its functional and performance requirements. As noted by the deterioration of many roadways and bridges in the USA [5.2], this often occurs prior to achieving the desired design life. Experience also has shown that the CCBs are subject to degradation mechanisms that require maintenance and repair actions [5.3].

As described in the previous chapter, in-service inspection techniques are available that can indicate the occurrence and extent of age- or environmental-stressor-related deterioration. Periodic application of these techniques as part of a condition assessment programme can monitor the progress of deterioration. Results obtained from these programmes can be used to develop and implement a remedial action prior to the structure achieving an unacceptable level of performance. Depending on the degree of deterioration and the residual strength of the structure, the function of a remedial measures activity may be structural, protective, cosmetic, or any combination of these three requirements. Basic components of a remedial measures programme include diagnosis (damage assessment), 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), and application [5.4]. Figure 5.2 indicates the basic steps of a typical repair strategy for three conditions that could result from an assessment of the condition of the structure in question - existing damage, damage to be expected, or no damage likely [5.5]. A detailed discussion of each of these steps is provided in the reference.

5.1. INITIAL CONSIDERATIONS

The first step in any repair activity is a thorough assessment of the damaged structure or component including evaluation of the (1) cause of deterioration, (2) extent of deterioration, and (3) effect of deterioration on the functional and performance requirements of the structure or component. Basic elements of the assessment include (1) preplanning and accumulation of background data (e.g., age, previous condition surveys, design documents, as-built drawings, and materials data sheets), (2) visual examination, (3) in situ and laboratory testing, and (4) evaluation of collated survey data and determination of cause(s) of deterioration. From this information a remedial measures strategy is developed based on the consequence of damage (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) [5.6]. Basic remedial measures options include (1) no active intervention; (2) more frequent inspections; (3) carry out repairs to restore deteriorated or damaged parts of structure to a satisfactory condition; (4) if safety margins are presently acceptable, take action to prevent deterioration from getting worse; and (5) demolish and rebuild all or part of structure. Quite often options (3) and (4) are considered jointly.

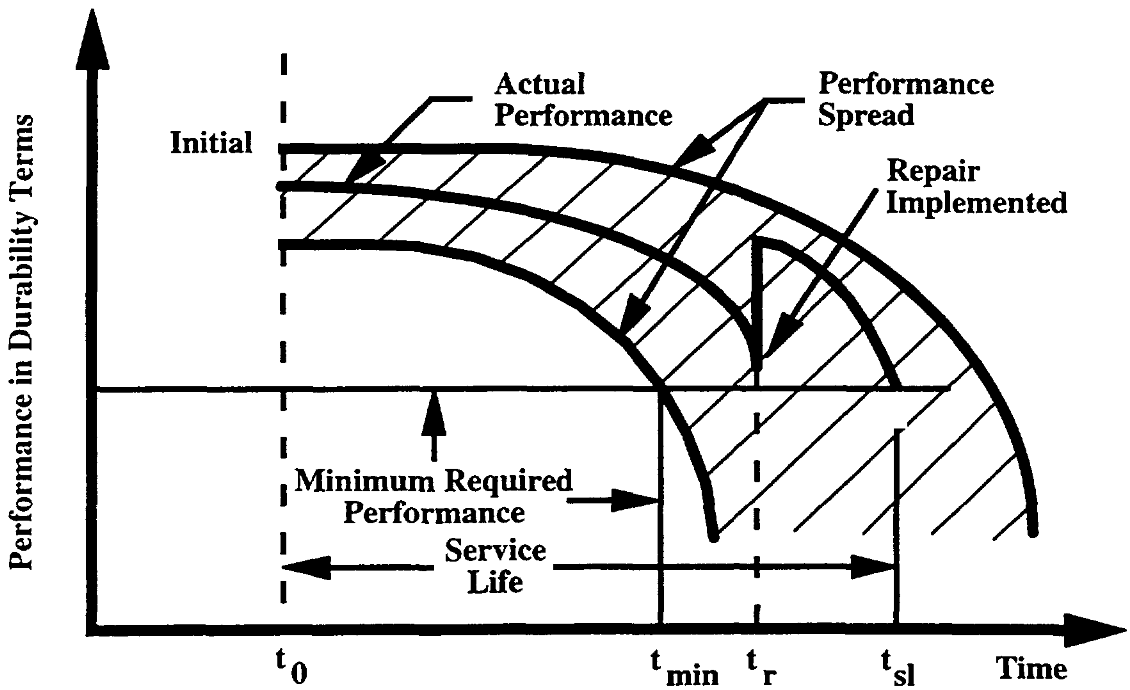

FIG. 5.1. Schematic showing relationship between concrete performance and time, including potential effect of a repair action.

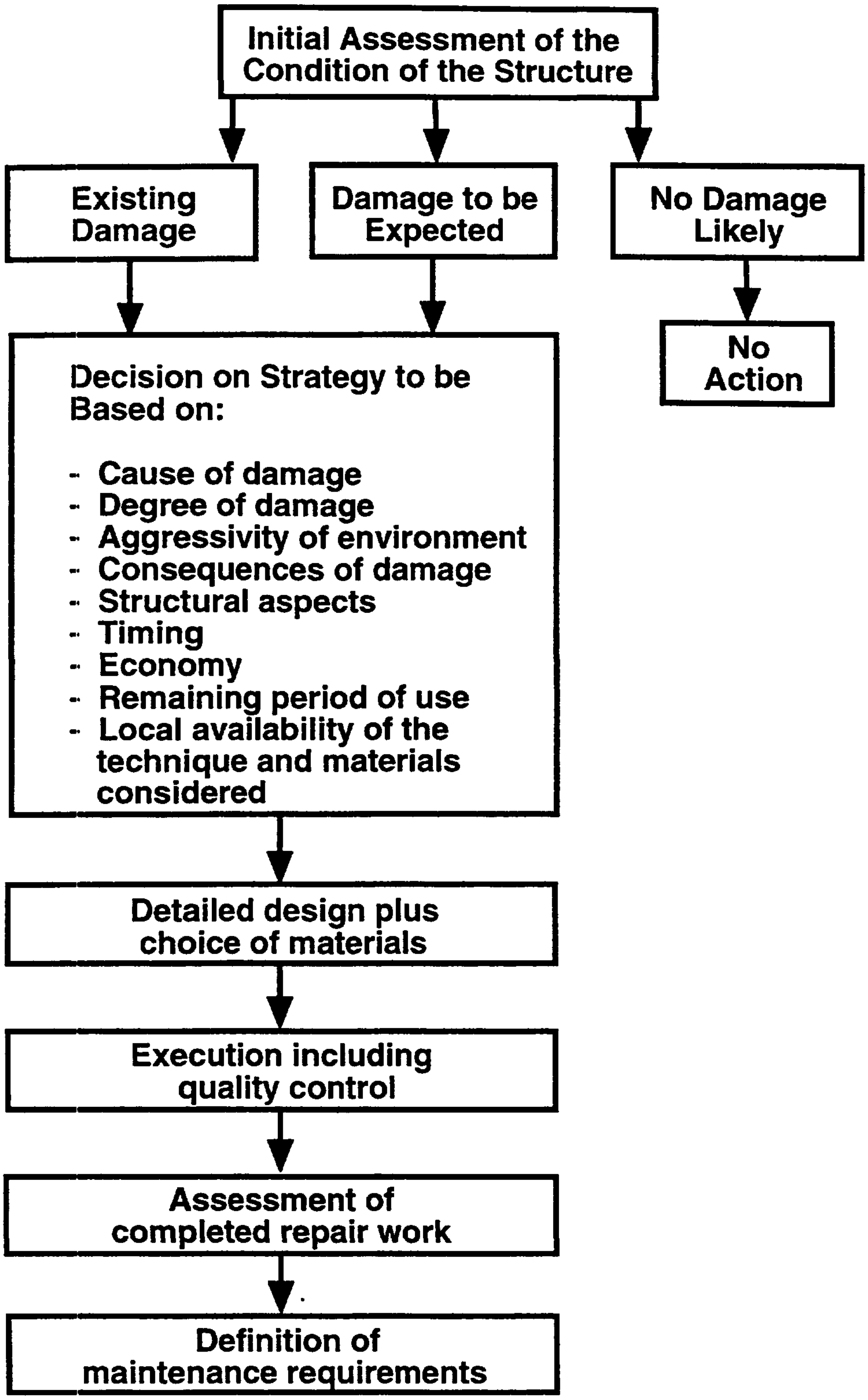

FIG. 5.2. Steps to be taken in a repair process [5.5].

5.2. MITIGATION AND REPAIR METHODS

Application of the basic remedial measures strategy includes the repair of damaged concrete and mitigation of the cause of deterioration.

5.2.1. Techniques and materials for repair of damaged concrete

Deterioration of reinforced concrete structures generally will result in cracking, spalling, or delamination of the cover concrete. Of these, surveys of general civil engineering construction [5.7] and nuclear power plant (NPP) structures [5.3, 5.8, 5.9] indicated that cracking was by far the most frequently occurring problem. Seepage of water through construction joints or cracks also was reported in the NPP surveys as well as the presence of honeycombs and voids.

5.2.1.1. Cracks

As discussed in Chapter 3, cracking in concrete structures can be expected to occur for a number of reasons, including plastic and drying shrinkage, thermal effects, fatigue, reactive aggregates, and excessive loads. Table 5.1 [5.9] indicates that these cracks may be active or dormant; this influences the selection of a repair method. The size of the crack also influences selection and timing of the repair method. All concrete structures will have cracks that can be classified into two categories: microcracks and macrocracks. Microcracks form within the cement paste adjacent to the aggregate particles and are discontinuous, very narrow, require magnification for identification, and necessitate no repair action. The microcracks are important from the standpoint that under increased loadings they become wider and propagate, and can eventually reach a size (i.e., macrocracks) sufficient to deteriorate the concrete, accelerate corrosion of embedded steel, or produce leakage. Macrocracks are important to service life.

Crack width and durability

Two theories have been proposed regarding how cracks reduce the service life of reinforced concrete structures [5.10]. The first is that they permit access of chloride ions, moisture, and oxygen to the steel, not only accelerating the onset of corrosion, but also providing space for the deposition of corrosion products. The second theory agrees that cracks may accelerate the onset of corrosion at localised regions on the steel, but after a few years of service there is little difference between cracked and uncracked concrete. Factors that may influence corrosion occurrence include crack arrangement, width, depth, shape orientation with respect to steel, intensity, origin; type of structure; and service environment [5.11].

Both the crack geometry (i.e., width and depth) and environmental exposure condition are important to the durability of reinforced concrete structures. A wide surface crack that quickly narrows with depth may not be as detrimental as a narrower surface crack that penetrates to the steel reinforcement. For structures that are located in controlled environments, cracks are primarily an aesthetic concern. Researchers have provided a number of relationships between maximum permissible crack width and exposure condition. Table 5.2 presents a summary listing of permissible mean crack widths and environmental factors that have been proposed to prevent corrosion [5.9]. Crack width criteria noted in several codes and standards pertaining to water-retention structures are presented in Table 5.3. Larger crack widths increase the probability of corrosion, [5.12], however as noted in Ref. [5.13], the

TABLE 5.1 CAUSES OF CRACKING [5.9]

Cause	Type of crack: Active	Type of crack: Dormant	Comment
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 as indicated by the redesign.
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 the steel will continue to corrode and crack the concrete.
Foundation settlement	X	X	Measurements must be made to determine if the 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 procedures (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 limited to serve as a general guide only. It should be recognized that there will be exceptions to all of the items listed.			

Source: Adaptation of Corps of Engineers information, Washington, DC

TABLE 5.2. CRACK WIDTHS TO PREVENT CORROSION OF STEEL REINFORCEMENT [5.9]

Author	Environment factors	Permissible width, mm
Rengers	Dangerous crack width	1.0 to 2.0
	Crack width allowing corrosion within 1/2 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 seawater	0.025 to 0.15
Haas	Protected structures (interior)	0.3
	Exposed structures (exterior)	0.2
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 seawater 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 5.3. CRACK WIDTH CRITERIA NOTED IN SELECTED CODES AND STANDARDS PERTAINING TO WATER-RETAINING STRUCTURES*

Designation	Code/Standard	Acceptable maximum mean crack width (mm)	Commentary
ACI 350	"Concrete Sanitary Engineering Structures," American Concrete Institute, Detroit, Michigan (1989)	0.22 0.26	Normal exposure (retain liquids with pH > 5, sulphate solutions < 1500 ppm) Severe exposure (limits exceed normal)
ACI 224	"Control of Cracking in Concrete Structures," American Concrete Institute, Detroit, Michigan (1990)	0.41 0.30 0.18 0.15 0.10	Dry air or protective membrane Humidity, moist air, soil Deicing chemicals Seawater (inc. spray), wetting and drying Water-retaining structure
BS 8007	"Design of Concrete Structures for Retaining Aqueous Liquids," British Standards Institution, London (1987)	0.2 0.1	Severe and very severe exposure Critical aesthetic exposure
JSCE-SP-1	"Standard Specification for Design and Construction of Concrete Structures," Japan Society of Civil Engineers, Tokyo, Japan (1986)	0.005xC** 0.004xC** 0.035xC**	Outdoors, ordinary conditions, underground Severe alternating wetting and drying with water containing corrosive substance, mild marine environment Steel reinforcement under corrosive conditions, structures subject to tidal action
JTJ-073	"Technical Specification for Highway Maintenance," Ministry of Communications, People's Republic of China, Beijing, China (1986)	0.25 0.20-0.30 0.30-0.50	Reinforced concrete Prestressed concrete Arch ring

*Limitation of stresses in reinforcing bars by several codes (e.g., ASME Section III, Subsection CC 3000 [5.46]) also has consequence of limiting effective mean crack width.

**Maximum concrete crack width expressed in terms of concrete cover (C) times a constant with C limited to 100 mm or less.

values of crack width are not always reliable indicators of the corrosion and deterioration to be expected. In general, there are no acceptance standards for crack widths in NPP CCBs, but Ref. [5.9] provides the following guidance for crack width acceptabilities: for severe exposure to deicing chemicals or for water tightness, widths ≤0.1 mm; for normal exterior exposures or interior exposures subjected to high humidities, widths ≤0.2 mm; for internal protected structures, widths ≤0.3 mm; and for structures containing chemicals or fluids that must remain leaktight, widths ≤0.05 mm.

Selection of crack repair technique

After identifying that the crack is of sufficient size to require repair, it is important to determine if the crack is active or dormant. A basic procedure for use in identifying the cause of cracks has been developed [5.14]:

Step 1 Examine the appearance and the depth of the cracking to establish the basic nature of the occurrence, such as pattern or individual cracks, depth of cracks, open or closed cracks, and extent of cracking.

Step 2 Determine, if possible, when the cracking occurred. This step will require examination of prior inspection and test reports, and talking with the individuals who operate the structure and possibly those involved in the construction.

Step 3 Determine if the cracks are active or dormant. This step may require monitoring the 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 caused by thermal expansion. Cracks that are moving but not growing should be treated as active cracks.

Step 4 Determine the degree of restraint. This step will require a thorough examination of the structure and the construction drawings, if available. Both internal restraint (such as caused by reinforcing steel and embedded items) and external restraint caused by other elements such as adjacent structures that may be present should be considered. A checklist for determining the cause of cracking is presented below. Using the 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 a detailed stress analysis.

- Check for major errors in design;
- Check easily identifiable causes:
  corrosion of reinforcement,
  accidental or impact loading,
  poor design detailing, and
  foundation movement; and
- Check other possible causes, such as:
  incidents during construction,
  shrinkage induced stresses,
  volume changes,
  chemical reactions,

moisture changes, and
freezing and thawing.

Having established the cause of cracking, several questions should be addressed:

- 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 the damage is such that loss of concrete mass is probable, treatment of the cracks may not be adequate. For example, cracking due to corrosion of embedded metal or freezing and thawing would be better treated by removal and replacement of concrete than by one of the crack repair methods.
- 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 5.3 and 5.4 present repair methodologies for dormant and active cracks, respectively [5.14]. Typical properties of an epoxy and a polyester chemical grout used to repair concrete are provided in Table 5.4 [5.15]. General guidance on crack repair options including perceived durability is presented in Table 5.5 [5.9]. Detailed descriptions of techniques available for repair of dormant or active cracks in reinforced concrete are available elsewhere [5.9, 5.14, 5.16-5.18]. Final selection of a repair technique should take into account durability, life-cycle costs, and labor skill and equipment requirements.

5.2.1.2. Spalls

Spalls can occur due to impacts, corrosion of embedded metals, erosion, or problems such as alkali-aggregate reactions, freeze-thaw, and fire exposure. Surface preparation is critical to a successful spall repair. The concrete substrate must be sound, and the exposed surface dry and free of grease, oil, and loose particles. Suitable techniques for surface preparation include use of small chipping hammers (followed by abrasive blasting, and removal of dust and chips by compressed air) and high-pressure water blasting. If steel reinforcement is exposed during the removal of degraded material, the excavation should be extended so that the steel will be enclosed in the patch material. If the steel reinforcement is corroded, the corrosion products should be removed and the steel coated with a barrier material such as epoxy resin, or a high electrically-resistant patch material utilized [5.9]. Corrosion-inhibiting admixtures (e.g., calcium nitrate and organic-based) can also be included in the concrete patch material[1] Shallow spalls (≤20 mm) are generally repaired using Portland cement-based mortar materials. Polymer concretes containing epoxies or methyl methacrylates also have been successfully utilised. Deep spalls are treated in a similar manner to shallow spalls except coarse aggregate is added to the repair material. To ensure good durability of the repair it is important that the repair material have mechanical and physical properties similar to the in-place concrete and that it is properly consolidated and cured. For mass concrete requiring extensive replacement of material, the repair patch may be built up in two or more

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[1]Additional information specifically addressing remedial measures for corrosion damaged concrete is presented in Section 5 2 2

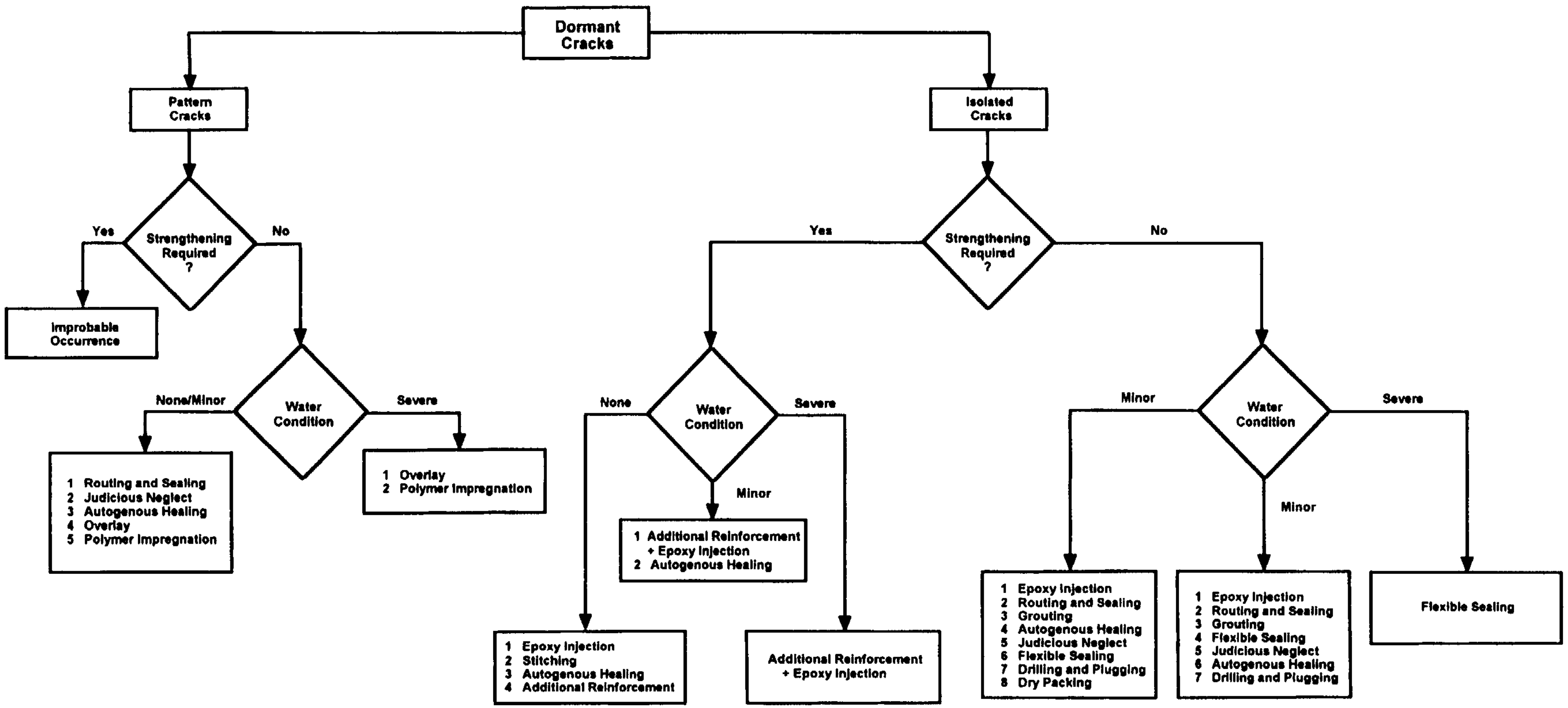

FIG. 5.3. Selection of repair technique for dormant cracks [5.14].

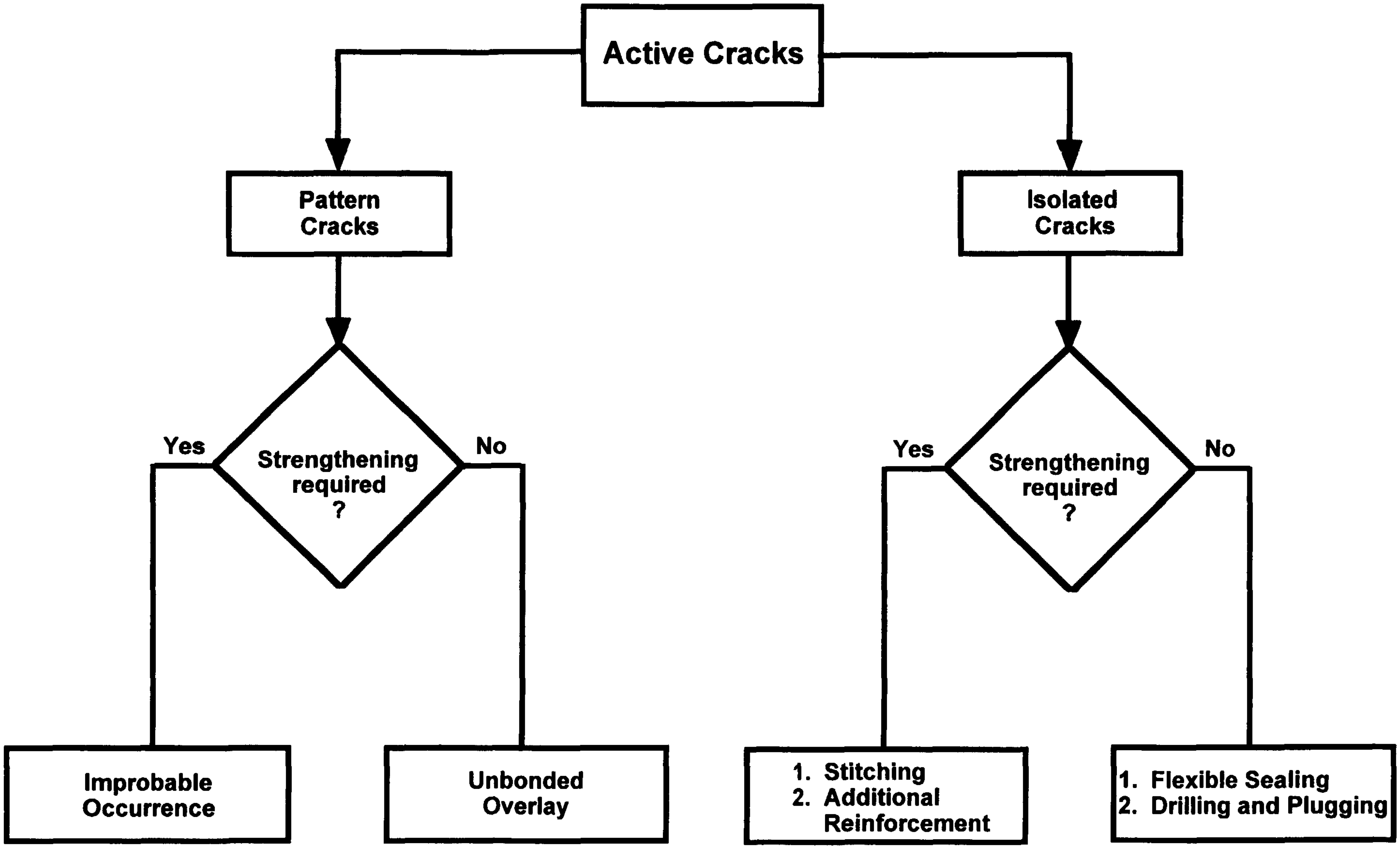

FIG. 5.4. Selection of repair technique for active cracks [5.14].

TABLE 5.4. PROPERTIES OF AN EPOXY AND POLYESTER CHEMICAL GROUT [5.15]

Property	Epoxy resin system	Polyester resin system*
Tensile strength, Mpa	35.2	55.8
Tensile modulus, Mpa	1.17×10^3	—
Elongation, %	15.8	2.6
Flexural strength, MPa	60.0	84.8
Flexural modulus, Mpa	1.45×10^3	4.2×10^3
Deflection, mm	13.5	—
Compressive strength, MPa	52	—
Compressive modulus, MPa	1.3×10^3	—
Deflection at yield, mm	4.6	—
Izod impact strength, J/m	55	—
Hardness at 25°C (Shore D)	79	38–40**
Water absorption, %	0.21	0.15
Shrinkage, % (volume)	0.001***	6.0

*Typical values.

**BARCOL.

***Effective (after gel information).

TABLE 5 5 GENERAL GUIDE TO REPAIR OPTIONS FOR CONCRETE CRACKING [5 9]

Description	Repair options	Perceived durability rating (1–5)*	Commentary
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_2O, Cl resistance
	Coatings	3	Use coating for abrasion & 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	
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 layers to prevent excessive heat build-up due to cement hydration. Fly ash can be used as a partial replacement for the cement to reduce the maximum temperature build up of the repair mass. Also, if the spall to be repaired is located in 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 [5.19]. Typically properties of materials commonly used for spall repairs are presented in Table 5.6 [5.20]. General guidance on spall repair options including perceived durability is provided in Table 5.7 [5.9]. Additional information on spall repair techniques is available elsewhere [5.9, 5.14, 5.16, 5.19].

5.2.1.3. Delaminations

Delaminations are horizontal voids in concrete domes or slabs that commonly occur due to corrosion of steel reinforcement or separation of concrete layers that do not develop adequate bond. Spalls will occur if the delaminations are not repaired. Delaminations can be repaired by removal and replacement of the delaminated concrete using procedures similar to those for repair of spalls. In areas where removal of concrete is not required, the delaminated area can be repaired by injection of an epoxy resin. Several holes are made into the delamination using either a drill with a vacuum attachment to remove the fines developed or by coring. If water is used in either process, the concrete should be permitted to dry prior to epoxy injection under low pressure or by gravity feed. Dowell pins can be used to enhance shear transfer. Additional information on delamination repair techniques is available elsewhere [5.9, 5.15, 5.17, 5.19].

5.2.1.4. Water seepage

Water seepage through construction joints and cracks may result in leaching of the concrete, entry of aggressive environments into the concrete matrix, steel reinforcement corrosion, or unacceptable flow of fluids either into or out of a facility. Long term reactions that require the presence of moisture such as efflorescence, sulfate attack, or alkali-aggregate reactions also may initiate as seepage into the concrete occurs. Implementation of a repair activity, therefore, can prevent possible future deterioration and the unacceptable migration of fluids either into or out of the facility.

A properly implemented repair procedure first will identify the source and then repair the path. Chemical grouting using silicate, acrylamide, lignin, or resin (i.e., epoxy, polyester, and urethane) systems is the most effective repair technique when moisture is present. The 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 the 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 [5.18]. Reaction of the chemical grout can be in the form of either soft flexible, semirigid, or rigid gels. When the 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.

5.2.1.5. Honeycomb and voids

Non-visible voids such as rock pockets, honeycomb, or excessive porosity can be repaired by drilling small diameter holes to intercept the voids; determining the extent and

TABLE 5.6. TYPICAL PROPERTIES OF RAPID SET PATCHING MATERIALS BY GENERIC FAMILY [5.20]

	Approx. working time (min)	Approx. time to traffic (min)	Compressive strength (MPa)		Abrasion loss (g)	Flexural strength (MPa)	Bond strength (MPa)		E (10^3 MPa)	a (10^{-6} per C)	Linear shrinkage (%)
MATERIAL	@ 22°C	@ 22°C	@ 3 h	@ 24 h	@ 24 h	@ 24 h	@ 24 h Dry PCC	@ 24 h Wet PCC			
Inorganic											
PCC w/accelerator	120+	300+	—	20	22	3.0	2.0	2.5	15–40	7–20	0.02–0.08
Magnesium phosphate	15	60	28	42	25	5.6	3.3	1.1	25–35	11–14	0.10–1.15*
High alumina cement	15	60	35	46	20	4.2	2.8	2.6	25–35	7–20	0.02–0.08
Gypsum based	20	60	25	42	18	2.8	2.1	2.6	15–20	7–20	0.03–0.05
Organic											
Epoxy	30–60	90–200	15	55–80	0–1	16–21	Failed in PCC	Failed in PCC	0.7–40	27–54	0.02–0.2
Methacrylate	20–40	60–120	50	55–65	10	14–21	Failed in PCC	Failed in PCC	7–25	13–23	1.5–5.0
Polyester-styrene	15–40	60–120	15	20–35	3	10–14	Failed in PCC	Failed in PCC	7–35	32–54	0.3–3.0
Urethane	5–45	30–90	3 - 15	3–35	3	10–27	Failed in PCC	3.4	0.7–40	54–126	0.02–0.2

E = Modulus of elasticity in compression. a = Thermal coefficient of expansion .

*High exotherm.

TABLE 5.7. GENERAL GUIDE TO REPAIR OPTIONS FOR CONCRETE SPALLING [5.9]

Description	Repair options	Perceived durability rating (1–5)*	Commentary
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 to 50 mm thickness
	Latex-modified concrete overlays	2	Greater than 30 mm thickness
	Portland cement concrete overlays	3	Use low water/cement ratio and high-range water reducer
	Silica fume overlays	3	High strength
	Pre-placed aggregate	2	Low shrinkage
	Shotcrete	3	Good for large areas

*Scale from 1 to 5, with 1 being most durable.

configuration of the void system by injection of compressed air or water into the void system, or by visual inspection using a borescope; and, depending on the magnitude of the delamination, injecting either epoxy resin, expansive cement grout or mortar, or epoxy-ceramic foam. Proper injection of the cement grouts requires prewetting of the substratum, with excess water removed prior to injection.

5.2.1.6. Alkali-aggregate reactions

Three basic requirements exist for occurrence of swelling and cracking of concrete due to alkali-aggregate reactions: (1) reactive silica or siliceous components in the aggregate, (2) sufficiently high hydroxyl ion concentration in the concrete pore solution, and (3) sufficient moisture availability [5.21]. Mitigation procedures generally involve elimination of one (primarily elimination or minimisation of exposure to moisture) or all of these requirements. In new construction, control of alkali-aggregate reactions is done by eliminating deleteriously reactive aggregate materials from consideration through petrographic examinations, laboratory evaluations, and use of materials with proven service histories. An improved test method for evaluating the reactivity of siliceous aggregates has been proposed [5.22]. A method for evaluating potential reactivity of carbonate aggregates is also available [5.23]. Additional mitigation procedures for new construction include use of pozzolans, restricting the cement alkali contents to less than 0.6% Na_2O equivalent, and application of 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 tried in the laboratory to reduce or alter the course of alkali-aggregate reactivity in concrete [5.24]. Application of lithium hydroxide, lithium carbonate, and lithium nitrite have been found to be effective [5.24]. Mitigating alkali-aggregate reactions in existing concrete structures can be done by interfering with the reaction mechanisms (e.g., drying and sealants), or treatment of symptoms (i.e., restraint and crack filling) [5.21]. Application of surface film coatings (e.g., acrylic/polyvinylacetate) to prevent moisture penetration has been found not always 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 the expansion caused by alkali-silica reactions [5.25]. If the alkali-aggregate reactivity has stopped, the only repair generally required is to fill the surface cracks with cement grout or resin. Other methods that have been utilised to not mitigate but to counteract the effects of alkali-aggregate reactions include strengthening by addition of external steel reinforcement [5.26], and cutting joints into the structure to accommodate expansions [5.27].

5.2.2. Remedial measure techniques for corrosion-damaged concrete

Corrosion of steel reinforcement is by far the greatest threat to the durability of reinforced concrete structures. A high percentage of the corrosion damage that has occurred to general civil engineering concrete structures has been the result of design deficiency, incorrect estimation of environmental actions, and by workmanship. As a consequence, many of these structures have had to be repaired, or under extreme conditions, removed from service because of corrosion of steel reinforcement. Fortunately, incidences of corrosion of NPP concrete structures have been limited [5.9, 5.16], probably due to more detailed considerations associated with material selection and construction workmanship, and effective use of quality assurance/quality control procedures. However, the history of these structures is somewhat limited and as they age, incidences of corrosion can be expected to increase.

5.2.2.1. *Preventative measures for in-service concrete*[2]

Corrosion of steel reinforcement can be expected to initiate if the concrete is sufficiently moist and either carbonation or sufficient chlorides have reached the surface of the steel. The most cost effective approach to treating corrosion of reinforced concrete structures obviously is to provide preventative protection, or if necessary, intervention early in the process. The application of barriers in the form of sealers, coatings, or membranes to exposed surfaces provides one commonly used measure of intervention.

Sealers are liquids applied to the surface of hardened concrete to either prevent or decrease the penetration of liquid or gaseous media (e.g., water, carbon dioxide, or aggressive chemicals) [5.28]. A number of 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 have been found to be effective in bridge deck applications – polyurethanes, methyl methacrylates, certain epoxy formulations, relatively low molecular weight siloxane oligomers, and silanes [5.29]. Of these, the silanes and oligomers presently are most commonly used.[3] Newer formulations of these materials penetrate the concrete surface to some degree, but still permit the transmission of air or water vapor. Therefore, coatings also have to be applied to protect against carbonation.

Coatings and membranes differ from sealers in that they are applied in some thickness, generally measured in hundredths of a millimetre, and generally do not penetrate the concrete. Coating types include epoxy resins, polyester resins, acrylics, vinyls, polyurethanes, and cementitious materials. Membrane types include liquid applied acrylics, urethanes, neoprenes, vinyls, ruberized asphalts, silicones, and preformed membranes such as ruberized asphalts, neoprenes, and butyl rubbers, hypalons, vinyls, and ethylene propylene diene monomer. Characteristics, advantages, and disadvantages of these materials are provided in Ref. [5.9].

Selection of sealer, membrane, or coating materials involves a number of factors (e.g., compatibility with new or old concrete, compatibility with joint sealant materials, crack-bridging ability, effective service life, and weatherability). Water absorption and vapor transmission properties for selected coating materials are presented in Table 5.8 [5.9]. An indication of the relative performance of several coating systems is provided in Table 5.9 [5.6]. Guidelines for selection of barrier systems for concrete are provided in Ref. [5.30]. Surface preparation is extremely important in the use of any sealer or coating material (i.e., cleanliness and moisture condition). Adhesion of film forming coatings can be evaluated by ASTM D 3359 [5.31], 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 gage, and pulled at 180° to the surface [5.9].

5.2.2.2. *Remedial measures for active corrosion*

In situations where application of preventative measures may not be possible, or the corrosion process has initiated, remedial measures are required. Basic remedial measures to strengthen or repair reinforced concrete structures damaged by corrosion include (1) taking no action, (2) replacement of damaged components, (3) stopping the corrosion process, and

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[2]Cathodic protection is covered in Section 5.2.2.2.

[3]Polyurethanes may degrade under ultraviolet exposure and the methacrylate monomer is highly volatile

TABLE 5.8. WATER ABSORPTION AND VAPOR TRANSMISSION OF SELECTED COATING MATERIALS [5.9]

		7-day water absorption, %			7-day water vapor transmission, %			
Generic type	No.	Average	Mean	Range	Average	Mean	Range	Application rate range sq ft/gal
Control	1	4.79	—	—	3.53	—	—	
Acrylics	9	0.82	1.84	0.50–3.08	1.56	1.47	0.83–2.81	75–110
Cementitious	5	2.75	2.94	0.49–4.56	1.80	2.18	1.02–2.28	—
Epoxy	7	0.31	0.16	0.06–0.37	0.20	0.21	0.06–0.46	60–125
Hypalon	2	0.36	—	0.29–0.42	0.23	—	0.18–0.37	75–95
Neoprene	1	1.06	—	—	0.37	—	—	70
Polyurethane	16	0.55	0.20	0.04–3.79	0.60	0.54	0.15–1.54	70–175
Polyester	1	0.12	—	—	0.27	—	—	90
Silicone	1	1.76	—	—	2.76	—	—	80

TABLE 5.9. RELATIVE PERFORMANCE OF SEVERAL COATING SYSTEMS FOR CONCRETE MATERIALS [5.6]

Coating	Application*				Protection*			
	Damp conditions	Alkali resistance	Ease to apply	Low hazard	Resist Cl^-	Resist CO_2	Vapor transmission	Resist rain
<u>Film Forming</u>								
Epoxy resin	4	1	5	5	1	1	5	1
Coal tar epoxy	4	1	5	5	1	1	5	1
Polyurethane	4	1	5	5	1	1	5	1
Chlorinated rubber	2	1	1	5	1	1	5	1
Bituminous	2	3	1	3	3	3	3	1
Acrylic resin	2	3	1	1	3	1	1	1
<u>Densifying</u>								
Silicate/flurosilicate	1	1	1	1	5	5	1	3
Cementitious	1	1	1	1	3	3	1	3
<u>Non-film Forming</u>								
Silane/siloxane	2	2	1	5	1	5	1	1
Oil impregnation	4	5	1	1	3	5	1	4
Silicone	4	5	1	2	3	5	1	3

*1 = very good, 5 = very poor.

(4) reducing the corrosion rate [5.5]. An example where no action might be taken would be in situations where the structure may be nearing the end of its desired service life and an assessment indicates that it can continue to meet its functional and performance requirements. Some local repairs and monitoring may be part of this strategy. When corrosion damage is localised on exposed surfaces, replacement or partial reconstruction may provide the most feasible solution. Often, however, some form of intervention is required to inhibit the corrosion process (i.e., reduce the 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.[4] Basic principles for halting the anodic and electrolytic processes are presented in Fig. 5.5 [5.5]. Brief descriptions of each of these basic principles are provided below.[5] More detailed information on each of these principles, including proper application, effectiveness, advantages, and disadvantages, and any limitations is provided elsewhere [5.5, 5.6, 5.32, 5.33].

- *Repassivation* — Three basic methods for repassivation of steel reinforcement are available: (1) use of alkaline cement or mortar, (2) electrochemical realkalisation, and (3) chloride extraction.

(1) Use of alkaline cementitious materials involves the placement of a cement-based mortar or concrete in the form of a patch (local) or layer over the entire concrete surface (general) and relies on the migration of alkalis into the old concrete. If cracks or spalls are present, loose material should be removed and loose rust cleaned from steel. The depth of placement of new material should 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 reinforcement or if steel depassivation has been caused by chlorides [5.5].

(2) Electrochemical realkalization restores a high pH to the concrete by generation of hydroxide ions at the steel and transport of 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 in the field to arrest corrosion, but its long-term effectiveness is questionable. Also, 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 evolution of hydrogen during the process, this method is not recommended where prestressing steel is present, and it may affect the steel-to-concrete bond. Also, the potential for alkali-aggregate reactivity is increased by high concentration of sodium ions [5.6].

(3) Chloride extraction removes chloride ions from the concrete in order to achieve a residual chloride ion concentration low enough to stop the

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[4]Halting the cathodic process requires the total blockage of oxygen access to the steel reinforcement. Since this cannot generally be accomplished, this process will not be addressed [5.5].

[5]Reduction of the moisture content will not be discussed as it is essentially the same as described in the previous section covering sealers, coatings, and membranes as preventative measures.

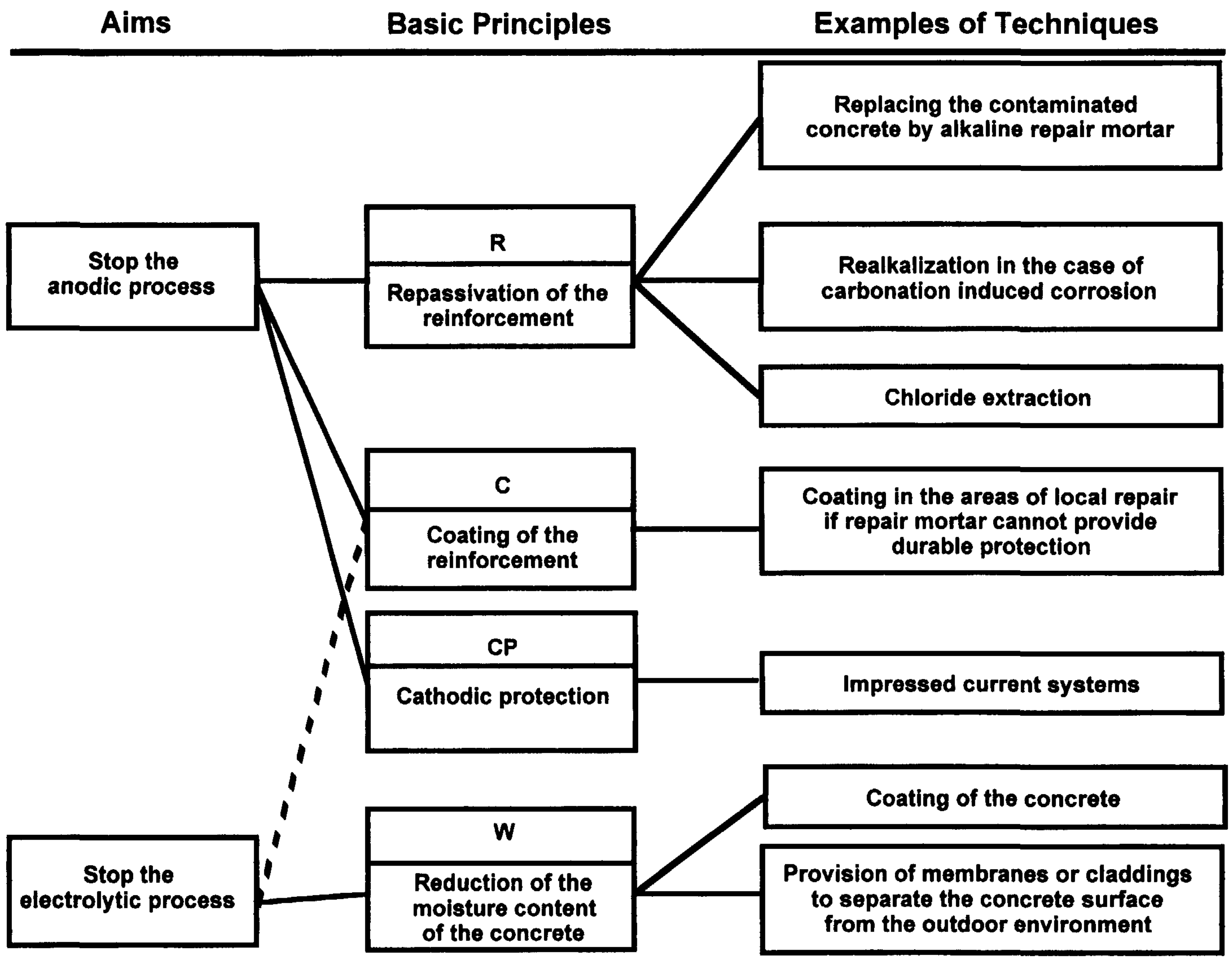

FIG. 5.5. Basic principles for halting the anodic and electrolytic process [5.5].

corrosion process. A direct-current field is applied through the concrete by means of an external anode on the concrete surface. The chloride ions migrate to the surface where they are captured in the electrolyte and removed when the process is completed. This method often has been combined with electrochemical realkalization and has the same basic precautions [5.5, 5.6].

- *Steel reinforcement coating* — Some repair systems require or include the application of a physical barrier (e.g., epoxy) to protect the steel and provide electrical resistance. The procedure requires the removal of concrete to a depth below the steel, a clean steel surface free of rust, and sometimes, if the steel is badly corroded, replacement of portions of the steel reinforcement. If the remainder of the steel in the structure is uncoated, and the coating in the repaired area is damaged and becomes corrosively active, a high rate of corrosion can occur. Also, in chloride environments, the chlorides may penetrate the ends of the coating to cause crevice corrosion where the steel has not been repassivated. Use of low water-cement ratio cementitious coatings may be preferred in chloride-contaminated concrete [5.5].

- *Cathodic protection* — The corrosion process can be effectively halted, or its rate decreased, through application of a small direct current to the steel reinforcement to make it slightly cathodic relative to an externally applied anode at or near the concrete surface. Two types of cathodic protection systems are available: (1) impressed current, and (2) 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 to be protected. In the sacrificial systems, a metal that is more anodic (higher tendency to corrode) than the embedded steel is used as the source of energy. 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. Special precautions are required when the following are encountered: alkali-aggregate reactivity, lack of steel reinforcement continuity, highly electrically-resistant concrete, epoxy coated rebars, or galvanised steel. Also, generation of hydrogen 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 other structures [5.5, 5.33]. Applications of cathodic protection systems to nuclear power plant concrete structures are somewhat limited and have generally involved safety-related concrete structures other than containments.

5.2.3. Remedial measures for liners and coatings

Protective coatings (e.g., inorganic zinc and polyamide epoxy) generally are applied to exposed interior surfaces of the containment metallic liners. Repair procedures may range from cleaning and recoating to weld overlays. The primary location where corrosion has been observed is along the circumference of the liner adjacent to the upper portion of the concrete basemat [5.34, 5.35]. The cause was attributed to a breakdown of the waterstop that permitted fluids to accumulate in this region. Remedial measures generally involved inspecting the region, cleaning, recoating, and reapplication of the waterstop. In once instance [5.35] where corrosion had progressed to the state that one-cm-diameter holes had penetrated the liner, the repair procedure involved removing some of the concrete slab adjacent to this 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 (i.e., more durable) waterstop installed.

The primary form of nonmetallic liner degradation has been occurrence of cracks due to localised effects (e.g., stress concentrations) or physical or chemical changes of the concrete. Leaktightness is generally reestablished through surface preparation and application of an additional coating such as polyurethane.

As coatings are subjected to a number of potential deterioration factors (e.g., temperature, abrasion, and high humidity), the coatings are inspected and areas of deterioration are repaired by cleaning and recoating with a compatible material.

5.3. EXISTING GUIDELINES

Reviews of repair procedures for reinforced concrete structures have been done from both the European and North American perspectives [5.6 and 5.9]. Although a number of codes and standards have been developed for new construction, none are presently available that specifically address repair of degraded structures. Several activities are presently underway, but this is expected to be a rather lengthy process. However, several documents are available in the form of guidelines or recommended practices.

In the USA, American Concrete Institute (ACI) Committee Reports 201.2 [5.17] and 546 [5.19] discuss concrete repair. The ACI also has produced several documents used in educational seminars that are of use — *Troubleshooting Concrete Problems - And How to Prevent them in the Future* [5.36] and *Concrete Repair Basics.* [5.37]. Both the US Army Corps of Engineers and US Bureau of Reclamation have produced concrete repair manuals [5.14, 5.18]. The manual by the Corps of Engineers provides a standard format for repair techniques and includes chapters on evaluation of concrete structures, causes of distress and deterioration, selection of materials and methods for repair or rehabilitation, maintenance of concrete, specialised 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 recently developed a notebook in the form of a computer data base that provides material data sheets on specific products [5.38].

In Europe, the most widely developed regulations for repair of concrete have been prepared by the *German Committee on Reinforced Concrete* [5.39]. The German guidelines address four major areas: general regulations and design principles, design and execution, quality assurance and execution, and technical delivery conditions and test regulations. Outside Germany, the Austrians have revised their standards to define an "orderly" basis for the future repair of concrete structures and a basis for the evaluation of existing structures [5.40]. Guidelines or recommended practices have been produced by the *Concrete Society* [5.41], *The Construction Industry Research and Information Association* [5.33], *The United Kingdom Department of Transport* [5.42, 5.43] *Comité Euro International du Béton* [5.44] and *Fédération International de la Précontrainte* [5.45]. Most of the European regulations address repair of corrosion-damaged concrete, indicating the magnitude of the problem.

Although codes and standards are not presently available, sufficient documentation, such as noted above, is available to develop an effective repair strategy for safety-related NPP concrete structures. As structures in the general civil engineering community have aged and incidences of degradation have increased, there is increasing awareness of potential problems.

Research being conducted to address these problems can be transferred to NPP concrete structures. The basic mechanisms leading to concrete degradation are generally understood. The importance of recognizing the critical role played by the environment, at both the macro and micro level, is being recognized, as well as the importance of workmanship during installation. Repair strategies are becoming more global in that they are looking at the entire repair process as opposed to merely the selection and application of a repair material based on information provided by the vendors. Long-term data on the effectiveness or durability of various remedial measures is required. Knowledge of the durability of various repair materials is required to formulate the most effective repair.

REFERENCES TO CHAPTER 5

[5.1] BROWNE, R.D., "Durability of Reinforced Concrete Structures", New Zealand Concrete Construction, Parts 1 and 2 (1989).

[5.2] GANNON, E.J., CADY, P.D., 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).

[5.3] Summary Results of the Survey on Concrete Containment Ageing, internal IAEA report, 1995.

[5.4] WADDELL, J.J., Basic Steps of a Concrete Repair Programme, Concrete International **2** (9), American Concrete Institute, Detroit, MI (1980) 30–33.

[5.5] TECHNICAL COMMITTEE 124-SRC, Draft Recommendation for Repair Strategies for Concrete Structures Damaged by Reinforcement Corrosion, Materials and Structures **27** (171), International Union of Testing and Research Laboratories for Materials and Structures (RILEM), Cachan (1994) 415–436.

[5.6] PRICE. W.F., et al., Review of European Repair Practice for Corrosion Damaged Reinforced Concrete, Report No. 1303/91/5823, Taywood Engineering Ltd., R&D Division, London (1993).

[5.7] MCDONALD, J., CAMPBELL, R., Sr., The Condition of the Corps of Engineers Civil Works Concrete Structures, US Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS (1985).

[5.8] GREGOR, F.E., HOOKHAM, C.J., "Remnant Life Preservation of LWR Plant Structures", Transactions of the 12th Int. Conf. on Structural Mechanics in Reactor Technology, 1993, Stuttgart, paper DH06/2, Elsevier Science Publishers, Amsterdam (1993) 453–459.

[5.9] KRAUSS, P.D., Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants, ORNL/NRC/LTR-93/28, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, TN (1994).

[5.10] DARWIN, D., Debate: Crack Width, Cover, and Corrosion", Concrete International, **7** (5), American Concrete Institute, Detroit, MI (1985) 29–35.

[5.11] CAMPBELL-ALLEN, D., LAU, B., Cracks in Concrete Bridge Decks, Civil Engineering Research Report R313, University of Sydney (1978).

[5.12] BEEBY, A.W., The Prediction of Crack Width in Hardened Concrete, The Structural Engineer, **57A** (1) (1979) 9–17.

[5.13] ACI COMMITTEE 224, Causes, Evaluation, and Repair of Cracks in Concrete Structures, Report No. 224R-84, Journal of American Concrete Institute **81** (3), Detroit, MI (1984) 211–230.

[5.14] UNITED STATES ARMY CORPS OF ENGINEERS, Evaluation and Repair of Concrete Structures, EM-1110-2-2003, Washington, DC (1986).

[5.15] UNITED STATES ARMY CORPS OF ENGINEERS, Chemical Grouting, Engineer Manual EM 1110-2-3504, Office of Chief of Engineers, Washington, DC (1973).
[5.16] NAUS, D.J., Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants, NUREG/CR-4652 (ORNL/TM-10059), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, TN (1986).
[5.17] ACI COMMITTEE 201, "Guide for Making a Condition Survey of Concrete In-Service", Proc. of American Concrete Institute **65** (11), Detroit, MI (1968) 905–918.
[5.18] BUREAU OF RECLAMATION, Standard Specification for the Repair of Concrete, M-47, M0470000.390, US Department of the Interior, Denver, CO (1990).
[5.19] ACI COMMITTEE 546, "Guide for Repair of Concrete Bridge Superstructures", ACI 546.1R-80 in Manual of Concrete Practice, Part 2, American Concrete Institute, Detroit, MI (1988).
[5.20] JERZAK, H., Unpublished Test Data, Transportation Laboratory, California Department of Transportation, Sacramento, CA (1988).
[5.21] 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).
[5.22] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Proposed Standard Test Method for Accelerated Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction", ASTM P 214, Annual Book of ASTM Standards, Vol. 04.02, Philadelphia, PA (1991).
[5.23] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)", ASTM C 586, ibid.
[5.24] HELMUTH, R., et al., Alkali-Silica Reactivity: An Overview of Research, SHRP-C-342, Strategic Highway Research Programme, National Research Council, Washington, DC (1993).
[5.25] HOBBS, D.W., Alkali-Silica Reaction in Concrete, Thomas Telford Ltd., London (1988).
[5.26] HOPPE, G.E., "Rehabilitation of an Arch Bridge", Concrete Alkali-Aggregate Reactions, (Proc. 7th Int. Conf. Ottawa), (P.E. Grattan-Bellew, Ed.), Noyes Publications, Park Ridge, NJ (1986) 199–203.
[5.27] CAVALCANTI, A.J.C.T., "Alkali-Aggregate Reaction at Moxotó Dam, Brazil", Concrete Alkali-Aggregate Reactions (Proc. 7th Int. Conf. Ottawa) (P.E. Grattan-Bellew, Ed.), Noyes Publications, Park Ridge, NJ (1986) 168–172.
[5.28] ACI COMMITTEE 116, Cement and Concrete Terminology, ACI 116R-85, American Concrete Institute, Detroit, MI (1985).
[5.29] PFEIFER, D.W., SCALI, M.J., Concrete Sealers for Protection of Bridge Structures, NCHRP Report No. 244, Transportation Research Board, National Cooperative Highway Research Programme, Washington, DC (1981).
[5.30] 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, Table 2.5.2 of Part 5, American Concrete Institute, Detroit, MI (1990) 515.1R-6 to 515.1R-11.
[5.31] AMERICAN SOCIETY FOR TESTING AND MATERIALS, "Method of Measuring Adhesion by Tape Test", ASTM D 3359-87, Annual Book of ASTM Standards, Vol. 06.01, American Society for Testing and Materials, Philadelphia, PA (1989).

[5.32] ACI COMMITTEE 222, Corrosion of Metals in Concrete, ACI 222R-89, American Concrete Institute, Detroit, MI (1989).

[5.33] PULLAR-STRECKER, P., Corrosion Damaged Concrete Assessment and Repair, Butterworths, London (1987).

[5.34] SHAH, V N., SMITH, S.K., SINHA, U.P., Insights for Aging Management of Light Water Reactor Components – Metal Containments, NUREG/CR-5324 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, ID (1994).

[5.35] 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, Organisation of Economic Co-operation and Development, Issy-les-Moulineaux (1995).

[5.36] AMERICAN CONCRETE INSTITUTE, Troubleshooting Concrete Problems – And How to Prevent Them in the Future, SCM-17, Detroit, MI (1987).

[5.37] AMERICAN CONCRETE INSTITUTE, Concrete Repair Basics, SCM 24-91, CT92, Detroit, MI (1991).

[5.38] UNITED STATES ARMY CORPS OF ENGINEERS, The REMR Notebook, Washington, DC (1992).

[5.39] GERMAN COMMITTEE ON REINFORCED CONCRETE, "Richtlinie für Schutz und Instandesetzung von Betonbauteilen, Part 2: Bauplanung und Bauausführung", Deutsche Gesellschaft für Stahlbeton, Berlin, Germany (1990).

[5.40] "Austrian Standard ÖNORM B4200, Part 6", Conf. on Repair and Maintenance of Concrete and Reinforced Concrete Structures, University of Innsbruck, Austria, 1988.

[5.41] CONCRETE SOCIETY, Repair of Concrete Damaged by Reinforcement Corrosion, Technical Report 26, London (1984).

[5.42] DEPARTMENT OF TRANSPORT, Code of Practice for Routine Maintenance, DTp, London (1985).

[5.43] DEPARTMENT OF TRANSPORT, Materials for the Repair of Concrete Highway Structures, DTp, Department of Transport, London (1986).

[5.44] COMITE EURO-INTERNATIONAL DU BETON, Behavior in Service, Maintenance, and Repair, Bulletin 163, Paris (1983).

[5.45] FEDERATION INTERNATIONALE DE LA PRECONTRAINTE, "Inspection and Maintenance of Reinforced and Prestressed Concrete Structures", FIP Guide to Good Practice, Fédération Internationale de la Précontrainte, Thomas Telford, London (1986).

[5.46] 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). New York (1995).

6. OPERATING EXPERIENCE

Many nuclear power plant concrete structures, including concrete containment buildings (CCB), have experienced degradation that has required repair. Table 6.1 presents a summary of documented concrete problem areas in NPPs. Provided below is a discussion of operating experience related to CCB degradation, inspection, and repair actions identified through a survey of 154 operating units [6.1]. Current ageing management practices and operating experience of several Member States are described in Appendix I.

6.1. DEGRADATION EXPERIENCE

In general, the performance of NPP concrete containment buildings has been very good [6.1-6.3]. Tables 6.2 and 6.3 provide a summary of reported degradation occurrences for the containment concrete and steel components that resulted from responses to the IAEA survey on concrete containment ageing [6.1]. These results indicate that cracking and corrosion were the most reported incidences for the concrete and steel materials, respectively. About 30% of these instances were attributed to design or construction deficiencies. Examples of some of the specific problems that have been reported in the literature include low 28-day compressive strengths, voids under the prestressing tendon bearing plates resulting from improper concrete placement; cracking of prestressing tendon anchorheads due to stress corrosion or embrittlement; and containment dome delaminations due to low quality aggregate materials and absence of radial steel reinforcement or unbalanced prestressing forces [6.4-6.9]. Other construction-related problems have included occurrence of excessive voids or honeycomb in the concrete, contaminated concrete, cold joints, cadweld (steel reinforcement connector) deficiencies, materials out of specification, higher than code-allowable concrete temperatures, exposure to freezing temperatures during concrete curing, misplaced steel reinforcement, prestressing system buttonhead deficiencies, and water-contaminated corrosion inhibitors.

Several incidences of age-related degradation also have been reported in the literature [6.4-6.9]. Examples of some of these problems include corrosion of steel reinforcement 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 damage, low strengths of tendon wires, contamination of corrosion inhibitors by chlorides, and corrosion of concrete containment liners.

6.2. INSPECTION EXPERIENCE

6.2.1. Methods utilized by owners/operators

Although a number of 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 the CCB. Tables 6.4 and 6.5, developed from information provided by the survey questionnaire sent to NPP owners/operators, present summaries of inspection techniques for the primary

Text cont. on p. 107.

TABLE 6 1 SAMPLING OF DOCUMENTED CONCRETE PROBLEM AREAS IN NUCLEAR POWER PLANTS

Plant	Problem area	Remedial measure implemented	Reference*
Wolf Creek	Voids up to 1 8 m wide and through the wall thickness occurred under equipment and personnel hatches in reactor containment building	Voids repaired and quality assurance programme updated	[6 1 1]
Callaway 1	Nineteen randomly located areas of honeycomb extending to bottom layers of rebar of reactor building 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	[6 1 1]
South Texas 1/2	Crack in fuel handling building wall due to shrinkage	No structural significance	[6 1 1]
	Rebars improperly located in buttress region of Unit 1 containment	Detailed analysis of as-built condition determined that no safety hazard to public occurred	
	Voids occurred behind liner plate of Unit 1 reactor containment building exterior wall because of planning deficiencies, long pour times, and several pump	Sounding and fiberoptic exam through holes drilled in liner plate were used to determine extent, areas were repaired by grout injectionbreakdowns	
Palo Verde 2/3	Honeycombing around vertical tendon sheath blockouts with most voids at buttress/shell interface above last dome hoop tendon	Condition was localised so area was repaired with grout	[6 1 1]
Farley 2	Three anchorheads on bottom ends of vertical tendons failed and 18 cracked with several tendon wires fractured, occurred about 8 years after tensioning, cause attributed to hydrogen stress cracking	All tendons and anchorheads from same heat were inspected with no further problems noted, 20 tendons replaced	[6 1 1]
La Salle 1/2	Low concrete strength at 90 days	In-place strength determined acceptable from cores, cement contents for future pours increased, strength low in only small percent of pours so did not threaten structural integrity	[6 1 1]
Summer 1	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 nonshrink grout, liner repaired and all welds leak tested	[6 1 1]

TABLE 6.1 (cont.)

Plant	Problem area	Remedial measure implemented	Reference
Summer 1	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	[6.1.1]
Brunswick 1/2	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	[6.1.1]
Sequoyah 2	Concrete in outer 2.5 to 5 cm of Unit 2 shield building was under-strength because of exposure to freezing temperatures at early concrete age	Determined not to affect shield building capability	[6.1.1]
Beaver Valley 1	Void ~0.9-m long 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	[6.1.1]
North Anna 2	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 pressurisation, cores showed cracks extended into concrete vertically	Cracks no structural threat, routed and sealed to prevent fluid penetration	[6.1.1]
Farley 1	Cracks detected in six containment tendon anchors during refueling outage	Anchorheads replaced	[6.1.1]
San Onofre 3	Tendon liftoff forces in excess of maximum value listed in plant technical specifications, cause was lower relaxation rate than expected	No threat to structural integrity	[6.1.1]
Three Mile Island 1	Cracking <0.02-cm wide in containment building ring girder and around tendon bearing plates	Cracks repaired and monitored during subsequent surveillance	[6.1.1]
Zion 1	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	[6.1.1]
Crystal River 3	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	[6.1.1]
	Dome delaminated over ~32-m-diameter area due to low concrete properties and no radial reinforcement to accomodate radial tension due to prestressing	Upper delaminated section removed, additional rebars provided, concrete replaced, dome retensioned, and structural integrity test conducted	

TABLE 6 1 (cont)

Plant	Problem area	Remedial measure implemented	Reference
Salem 2	Incomplete concrete pour near equipment hatch due to use of wrong concrete mix	Voids repaired with high-strength nonshrink grout	[6 1 1]
Calvert Cliffs 1/2	Eleven of top bearing plates at Units 1 & 2 depressed into concrete because of voids, 190 plates of each containment exhibited voids upon further inspection	Tendons detensioned, plates grouted and tendons retensioned	[6 1 1]
Ginna	Excessive loss of prestressing force	Tendons retensioned with no recurrence noted in subsequent inspections	[6 1 1]
Indian Point 2	Concrete temperature local to hot penetration >66_C but <93_C	No safety problem due to relatively short periods of exposure	[6 1 1]
Grand Gulf 1/2	Seven of 19 cylinders for control building base slab concrete did not meet 28-day design strength	90-day values were acceptable	[6 1 1]
	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 rebars, replacing concrete and liner, and leak testing liner	
Turkey Point 3	Voids below containment wall and near reactor pit	Repaired with high-strength grout, guniting, or dry packing	[6 1 1]
	Dome delamination	Delaminated concrete removed, additional rebars provided, concrete replaced	
	Grease leakage from 110 of 832 tendons at casing	Tendon casings repaired and refilled	
	Concrete spalling of horizontal joint at containment ring girder with cavities 3 to 5 cm wide by 7 to 10 cm deep	No threat to structural integrity repaired by dry packing	
	Small void under equipment hatch barrel	No threat to structural integrity, repaired by grouting	

TABLE 6 1 (cont)

Plant	Problem area	Remedial measure implemented	Reference
Bellefonte 1/2	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	[6 1 1]
	Eight rock anchorheads failed during construction because of possible stress-corrosion cracking	Anchorheads replaced with cleaner steel	
Comanche Peak 1/2	Cold joint formed in reactor mat	Concrete removed, rebars exposed and new joint poured	[6 1 1]
	Inadequate concrete compaction under containment wall for 58 m at 1 8 to 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	Core holes drilled for inspection in conjunction with analytical evaluations revealed basemat was adequate for all loading conditions, core holes filled with mortar and interconnecting voids grouted	
Byron 1	Four anchorhead failures occurred in first year after stressing, cause was use of vanadian grain refinement process in conjunction with temperatures not high enough	Anchorheads replaced	[6 1 1]
Palisades	Sixty-three out of 3780 buttonheads inspected found split	No threat to structural integrity	[6 1 1]
Turkey Point 4	Approximately 0 1 m^3 of concrete with inadequate fines	Area removed and refilled with correct concrete mix	[6 1 1]
Monticello	Honeycomb voids in basemat	Repaired by epoxy injection	[6 1 2]
Oconee 2/3	Failed and corroded tendons observed during final containment testing prior to startup, cause was attributed to stress-corrosion cracking	Defective tendons replaced and drains installed on anchorage grease caps	[6 1 2]
Oconee 1	Tendon anchorhead thread problems due to overstressing	Failed anchorheads repaired, other anchorheads surveyed	[6 1 2]
Point Beach 2	Liner plate separated from concrete at several locations	Evaluation indicated no action required	[6 1 2]
H B Robinson	Grout under several bearing plates failed while tensioning tendons	Grout replaced	[6 1 2]
KKG Gosgen (CH)	Minor cracks in outer dome shell detected by visual inspection	Remote crack measurement vs time (data logging) to determine appropriate protective coating	[6 1 3]

TABLE 6.1 (cont.)

Plant	Problem area	Remedial measure implemented	Reference
KKL Leibstadt (CH)	Extensive micro-organic vegetation on dome causing dark areas	Detailed investigations showed limitation of effect to 2-mm depth, no further action taken	[6.1.3]
Pickering A	Cracks in vacuum building floor due to premature exposure to cold temperature during curing	Epoxy grout injection	[6.1.4]
	Shrinkage cracks in reactor building domes leading to increased leakage rates with time	After 20 years monitoring, structural evaluation conducted and polyurethane overlay applied to part of dome of one building	
	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	
	Epoxy liner in spent fuel storage bay damaged by local overexposure to radiation	Not structurally significant	
	Corrosion of reinforcing steel in one vacuum building column due to insufficient cover/tiewire exposure	Affected concrete removed and area patched with new concrete	
Bruce "A"	Steel intake roof structure damaged due to overload	Concrete slab applied to strengthen roof	[6.1.4]
	Steel intake roof structure corroding due to aggressive lake water	Cathodic protection system applied, coupon monitoring programme implemented (subsequent structures protected by coating)	
Gentilly "2"	Increasing leakage rates through reactor building containment walls due to presence and increase in number and width of cracks in epoxy liner with time due to concrete cracking	Portions of epoxy liner recoated with polyurethane	[6.1.5]
Various UK PCPVs	Small surface cracks observed during biennual inspection	Monitored to show cracks were passive, no further action required	[6.1.6]
	Minor spalling in vicinity of helical prestressing system anchorages	Judged to have no structural significance, no further action	
Dungeness B	Corrosion of prestressing tendons due to stray electrical currents at time of construction	Tendons replaced	[6.1.6]
Hunterston B	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	[6.1.7]

TABLE 6.1 (cont.)

Plant	Problem area	Remedial measure implemented	Reference
Oldbury	Localized honeycombing behind prestressing anchorages leading to loss of prestressing force	Injected with epoxy resin	[6.1.8]
	Circumferential crack at joint due to construction sequence	Injected with resin grout	[6.1.9]
	Local concrete temperatures higher than design values	Changes made to thermal shielding, analytical and experimental evaluations to demonstrate acceptability of local hot spots	[6.1.8]
	Settlement of circular raft foundation exceeded design predictions	Floors jacked and pipework adjusted, monitoring used to show that settlement has stabilised	[6.1.8]
Various French 900 MW(e) Units	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	[6.1.10]
	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, modeling studies of concrete creep and shrinkage being conducted	[6.1.10]
	Corrosion of some prestress tie rods of the reactor pit lateral supports and loose nuts	Increased inspections, tighten loose nuts and redo calculations underpinning the safety report to show that safe-shutdown earthquake criterion can be met	[6.1.11]
Doel 3	Electrical resistance strain gauges inside the concrete are defective	New vibrating wire strain gauges to be installed on external face of the shell	[6.1.12]

TABLE 6 1 (cont)

Plant	Problem area	Remedial measure implemented	Reference
Tarapur	Localized cracking/spalling of the cover concrete and initiation of rebar corrosion at a few locations on exterior faces of the 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 Note- Tarapur is a BWR-based plant having a primary containment with pressure surppression system Secondary containment does not play significant safety role as such Leakage-rate testing is carried out as per technical specifications and the secondary containment has met all requirements to date	Tarapur plant 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 the findings of the condition survey and monitoring, an assessment of the structure can be carried out	[6 1 13]

[6 1 1] NAUS, D J , *Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants*, NUREG/CR-4652 (ORNL/TM-10059), Martin Marietta Energy Systems, Inc , Oak Ridge National Laboratory, Oak Ridge, TN (1986)

[6 1 2] HOOKHAM, C J , SHAH, V N , "Insights for Aging Management of Major Light Water Reactor Components, Volume 2 - Reinforced And Prestressed Concrete Containments", NUREG/CR-5314 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, ID, July 1994 (draft)

[6 1 3] HEEP, W , "Letter from Mr W Heep (Nordostschweizerische Kraftwerke) to Mr D J Naus (Oak Ridge National Laboratory)", August 15, 1996

[6 1 4] CRAGG, C B H , "Overview of Ageing Management Programmes for Containment Structures at Ontario Hydro", IAEA Research Co-ordination Meeting on Ageing of Concrete Containment Buildings, June 30 July 2, 1993, IAEA, Vienna (1993)

[6 1 5] SENI, C , "Overview of AECL Research Related to Reducing Leakage Through Reactor Building Concrete Containment Walls," ibid

[6 1 6] IRVING, J , SMITH, J R , EADIE, D McD , 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)

[6 1 7] JONES, W C , TAYLOR, S J , "Construction of the PCPVs at Hinkley Point B and Hunsterston B,' ibid

[6 1 8] 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," ibid

[6 1 9] HOUGHTON-BROWN, A , DARTON, A J , "The Oldbury Vessels", Proc Conf on Prestressed Concrete Pressure Vessels, 1967, The Institution of Civil Engineers, London (1967)

[6 1 10] NUCLEAR ENERGY AGENCY, "Report of the Task Group Reviewing International Activities in the Area of Ageing of Nuclear Power Plant Concrete Structure' NEA/CSNI/R(95)19, Organisation of Economic Cooperation and Development Issy-les-Moulineaux (1995)

[6 1 11] UNITED STATES NUCLEAR REGULATORY COMMISSION, "DSIN Worried that Two New Defects may be Present in Many French Units', in Inside NRC, Washington, DC (1996) 12–14

[6 1 12] de MARNEFEE, L SEUNIER, D , "Ageing of Concrete Containment Buildings", BELGATOM, Brussels, Belgium, Letter to D J Naus Oak Ridge National Laboratory, Oak Ridge, TN (1997)

[6 1 13] 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

TABLE 6.2. OCCURRENCES AND MANIFESTATIONS OF DEGRADATION FACTORS FOR CONCRETE MATERIALS

Degradation Factor Reported \ Manifestation and Number of Incidences Reported	Cracking	Scaling	Delamination	Staining	Spalling	Efflorescence	Popout	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
Sulfate attack	--	--	--	--	--	--	--	--	--	--	--	--
Seawater exposure	6	--	--	--	--	--	--	--	--	--	--	6
Chemical attack	--	--	--	--	--	--	--	4	--	--	4	8
Leaching	--	--	--	3	--	11	--	--	--	--	--	14
Abrasion	--	--	--	--	--	--	--	--	--	--	--	--
Impact	--	--	--	--	--	--	--	--	--	--	--	--
Shrinkage	54	--	--	--	2	--	--	--	--	--	--	56
Sealant failure	1	--	--	--	--	1	--	--	--	--	--	2
Creep	10	--	--	--	--	--	--	--	--	--	--	10
Leakage test	7	--	--	--	--	--	--	--	--	3	--	10
Irradiation	--	--	--	--	--	--	--	--	--	--	--	--
Chloride penetration	--	--	1	--	2	--	--	--	--	--	--	3
Carbonation	--	--	--	1	--	--	--	--	--	--	--	1
Alkali/aggregate reaction	--	--	--	--	--	--	--	--	--	--	--	--
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

TABLE 6.3. OCCURRENCES AND MANIFESTATIONS OF DEGRADATION FACTORS FOR STEEL MATERIALS

Degradation Factor Reported \ Manifestation and Number of Incidences Reported	Reinforcing steel corrosion	Prestressing tendon corrosion	Containment penetration	Prestress loss	Subtotal of events
Freeze/thaw	--	--	--	--	--
Elevated temperature	--	--	4	--	4
Sulfate 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

TABLE 6.4. UTILIZATION OF DIFFERENT INSPECTION TECHNIQUES FOR ALL CONTAINMENT ELEMENTS

Inspection Technique	Percentage of Units Utilizing, %				
	Concrete	Anchorage Elements	Reinforcing Steel	Prestressing Steel	Liner/ Penetrations
Visual Inspection	84	73	26	26	53/--
Instruments	52	--	--	--	--/--
NDE/NDT	28	--	--	--	--/10
Coring	<1	--	--	--	--/--
Pullout 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 6.5. UTILIZATION OF DIFFERENT INSPECTION TECHNIQUES FOR ALL CONTAINMENT ELEMENTS

Inspection Technique	Percentage of Owners Utilizing, %				
	Concrete	Anchorage Elements	Reinforcing Steel	Prestressing Steel	Liner/ Penetrations
Visual Inspection	59	34	27	22	51/--
Instruments	20	--	--	--	--/--
NDE/NDT	24	--	--	--	--/10
Coring	<2	--	--	--	--/--
Pullout 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

containment elements in terms of the percentage of responding units which apply these techniques of the percentage of the responding owners who utilize them respectively. As noted in these tables, visual inspection was the most common method used to inspect the different parts of the CCB.

Visual inspection of the concrete structures was supplemented by crack mapping and instrumentation/nondestructive testing. Figure 6.1 presents the visual inspection/crack mapping intervals summarized by percentage of units and owners responding to the survey questionnaire. More detailed information on the crack mapping procedure used is provided in Fig. 6.2. Table 6.6 shows that thermocouples and strain gauges were the most common types of instrumentation employed and Table 6.7 shows that leakage-rate testing was the most frequently reported non-destructive testing method utilized. The amount of instrumentation contained in a CCB varied due to Member State code requirements. In some countries instrumentation was used mainly during the start-up phase to verify compliance with construction codes and assumptions for the plant response, (i.e., preoperational testing). In other countries, monitoring of the plants continues through the operation phase.

Visual inspections of the other elements also were supplemented by additional monitoring or testing. Half-cell potential testing of mild steel reinforcing systems were performed by about 6% of the units at intervals of 10 years. Where load cells were employed with prestessing 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 one to two years.

6.2.2. Observations on methods used

Based on the responses from 41 owners/operators representing 154 nuclear units, the following are general observation related to inspection of CCBs:

- The most commonly used methods of inspection (visual, leakage-rate tests, and prestressing system evaluations) were those prescribed by codes or regulations. Only a few owners used methods beyond those normally associated with code requirements.
- Inspections of the various elements were performed at uniform intervals, with intervals and number of inspection methods varying between owners and countries. This reflects different refueling cycles as well as regulatory requirements. Visual inspections of concrete were most commonly performed at five year intervals.
- Longer intervals (5 and 10 years) between inspections usually were associated with owners using instrumentation, supplemental inspection techniques, or other methods to augment the visual inspection 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. This could leave the impression 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 experience and judgment of the inspectors.

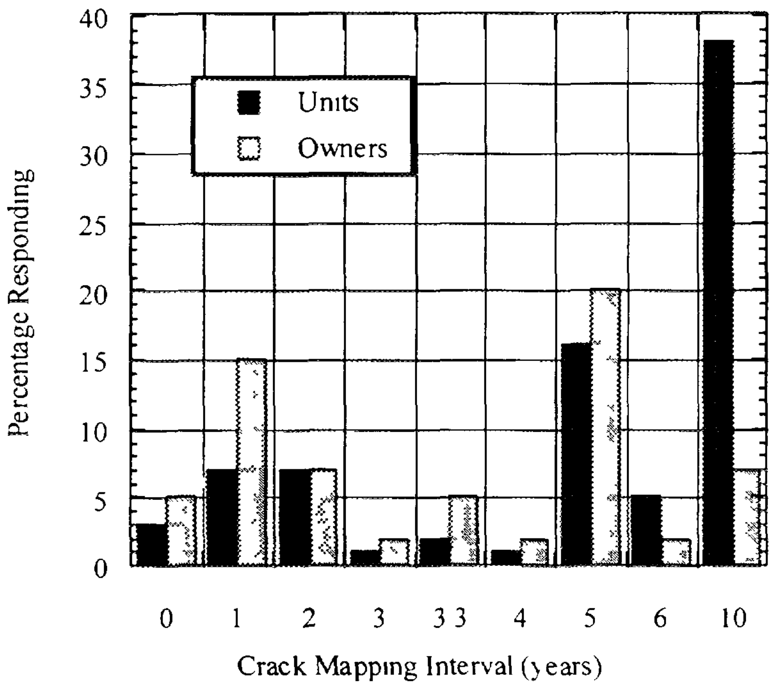

FIG 6.1 Concrete visual inspection/crack mapping frequency summarized by percentage of units and owners responding

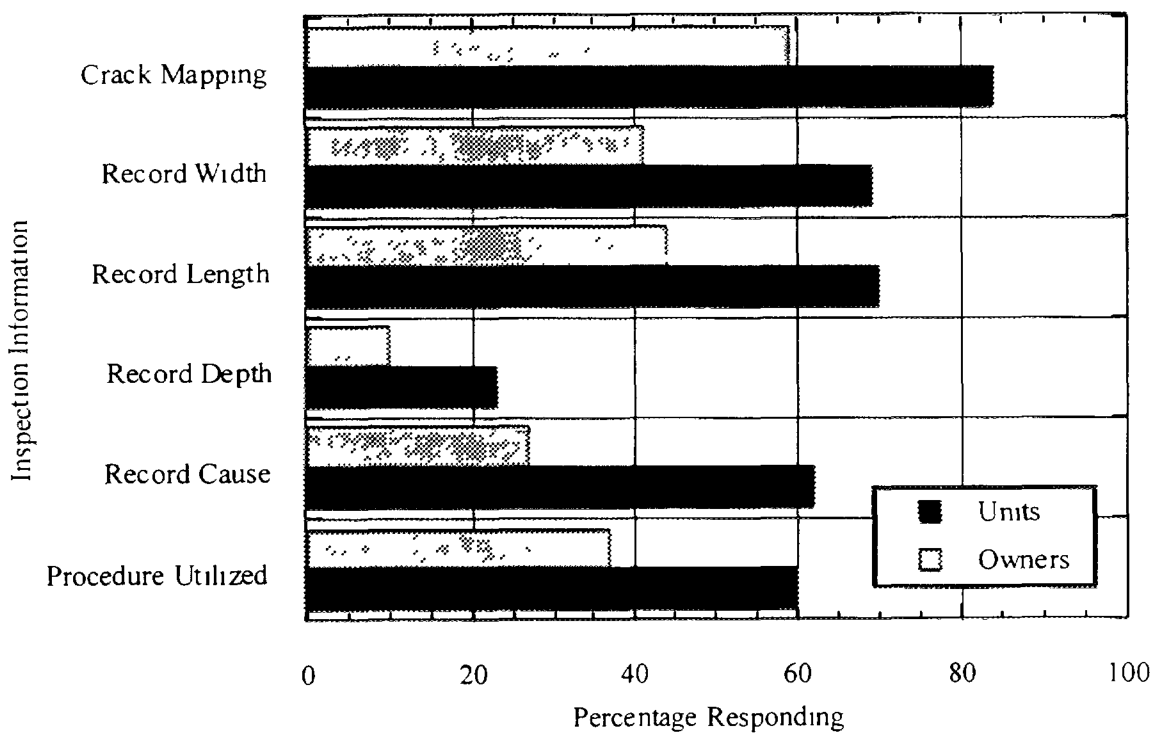

FIG 6.2 Crack mapping information summarized by percentage of units and owners responding

TABLE 6.6. USE OF CONCRETE INSTRUMENTATION

Instrumentation Type	Percentage of Units Utilizing, %	Percentage of Owners Utilizing, %	Number Used	
			Range	Median
Thermocouple	52	20	8 to 1335	27
Strain Gauge	46	17	18 to 240	103
Stress Cell	5	10	4 to 80	8
Humidity Gauge	3	5	1 to 2	1
Invar Wire	12	5	4	4
Other	42	10	--	--

TABLE 6.7. NONDESTRUCTIVE TESTING USED ON CONCRETE

Test Type	Percentage of Units Utilizing, %	Percentage of Owners Utilizing, %
Leakage Rate	26	24
Impact Hammer	17	7
Pulse Velocity	17	10
Permeability	5	5
Others	5	12

TABLE 6.8. REPORTED CONCRETE REPAIR ACTIONS

Type of Repair Action	Type of Degradation (Number of Repair Actions)					
	Voids/Honeycomb	Cracking	Delamination	Spalling	Popouts	Rebar Corrosion
Epoxy Injection		13	2			
Routing & Sealing		8				
Drilling & Plugging		1				
Flexible Sealing		7				
Grout Injection		1	1			
Dry Pack/Crack		2				
Polymer Impregnation		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

6.3. REPAIR EXPERIENCE

Results of survey questionnaires [6.1, 6.10] sent to plant owners/operators indicated that many of the repair activities were associated with problems during initial construction (e.g., cracks, spalls, and delaminations). As noted in Table 6.8, which summarizes concrete repair methods identified in the IAEA survey questionnaire [6.1], concrete cracking was the most common form of degradation reported. The most frequent approach used to address concrete 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. Table 6.1, which presented a summary of documented concrete containment problem areas, also identifies the remedial measure(s) that was implemented as a result of the observed degradation noted.

REFERENCES TO CHAPTER 6

[6.1] Summary Results of the Survey on Concrete Containment Ageing, internal IAEA report, 1995.

[6.2] NAUS, D.J., OLAND, C.B., ELLINGWOOD, B.R., Report on Aging of Nuclear Power Plant Reinforced Concrete Structures, NUREG/CR-6424 (ORNL/TM-13148), Lockheed Martin Energy Research Corp., Oak Ridge National Laboratory, Oak Ridge, TN (1996).

[6.3] HOOKHAM, C.J., SHAH, V.N., Insights for Aging Management of Major Light Water Reactor Components, Vol. 2 – Reinforced and Prestressed Concrete Containments, NUREG/CR-5314 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, ID, July 1994 (draft).

[6.4] GREGOR, F.E., HOOKHAM, C.J., "Remnant Life Preservation of LWR Plant Structures", pp. 453–459 in Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology held August 15–20, 1993, in Stuttgart, Germany, Paper DH06/2, Elsevier Science Publishers, Amsterdam (1993).

[6.5] 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, Toronto (1990).

[6.6] "Containment Liners Found Corroded at Some EDF 900-MW Reactors", Nucleonics Week, McGraw-Hill, New York (1992) 5–6.

[6.7] POCOCK, D.C. et al., Long-Term Performance of Structures Comprising Nuclear Power Plants, EUR 12758 EN, Commission of European Communities, Luxembourg (1990).

[6.8] DAWSON, P., WILSON, M.J.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).

[6.9] ZAKAIB, G.D., "Pickering Unit #1 Containment Leakage Characterization", Proc. Third Int. Conf. on Containment Design and Operation, 1994, Toronto (1994).

[6.10] KRAUSS, P.D., Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants, ORNL/NRC/LTR-93/28, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge (1994).

7. CONCRETE CONTAINMENT BUILDING AGEING MANAGEMENT PROGRAMME

The preceding chapters of this report dealt with the key elements of a CCB ageing management programme whose objective should be to maintain the fitness-for-service of a CCB at a NPP throughout its service life. Chapters 2 and 3 contain information on important aspects of understanding CCBs and their ageing. Chapter 4 provides information on various techniques for detecting and monitoring ageing. Chapter 5 contains information related to assessment procedures and methods for repair of unacceptable ageing effects. Chapter 6 summarizes operating experience in terms of degradation, inspection, and repair at NPP CCBs. Information provided in this report suggests that degradation of a CCB and its various subcomponents by chemical attack, elevated temperature, vibration-induced fatigue, and other ageing mechanisms may impair required CCB leaktightness and structural integrity. Although in all cases the CCB functional capability was preserved, the degradations experienced have been of concern to the safety authorities in various countries. Therefore, systematic CCB ageing management programmes are needed at NPPs.

The objective of an Ageing Management Programme (AMP)[1] for a CCB is to ensure the timely detection and mitigation of any degradation that could impact its safety functions. Its main characteristic is a systematic, comprehensive, and integrated approach aimed at ensuring the most effective and efficient management of ageing. A comprehensive understanding of a CCB, its ageing degradation, and the effects of degradation on the CCB's ability to perform its design functions is a fundamental element in the AMP. This understanding is derived from knowledge of the design basis, including applicable codes and regulatory requirements; the design and fabrication, including material properties and specified service conditions; the operation and maintenance history, including commissioning and surveillance; the inspection results; and generic operating experience and research results.

Figure 7.1 shows the key elements of a concrete containment building AMP as well as their integration. Understanding is the basis for an AMP. Knowledge of the plant, and of the impact of any potential degradation, is fundamental in making decisions about the inspection requirements, evaluating results, and choosing any remedial strategies. Plant specific knowledge is enhanced by drawing on external experience related to concrete behaviour. Definition of an AMP includes documentation of relevant programmes and activities and a description of mechanisms used for programme co-ordination and continuous improvement.

Managing ageing through a systematic AMP consists of the following activities, based on the understanding of CCB ageing:

- *Operation* of plant within design limits to minimize age-related degradation (in particular, error-induced accelerated degradation),
- *Inspection, Monitoring,* and *Condition Assessments* to detect and characterise significant component degradation before fitness-for-service is compromised, and
- *Maintenance* to manage ageing effects.

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[1]The importance of AMPs for concrete structures has been recognised by non-nuclear industries. Appendix II presents an overview of approaches being used by several of these industries.

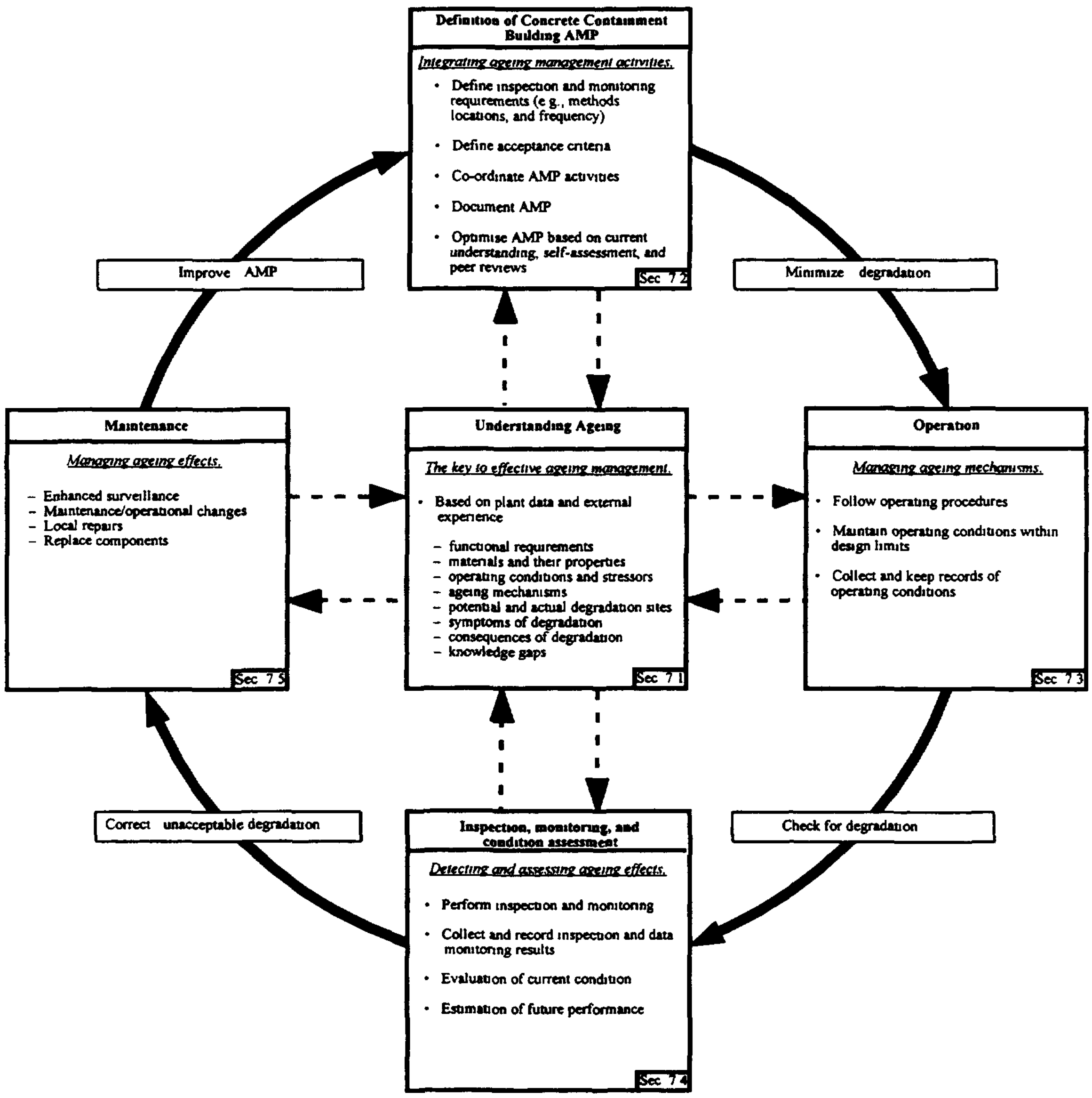


FIG. 7.1. Key elements of a CCB ageing management programme (AMP) and their interfaces.

Such an AMP should be implemented in accordance with guidance prepared by an interdisciplinary CCB ageing management team organised at a corporate or owner's group level.

For guidance on the organisational aspects of a plant AMP and interdisciplinary ageing management teams, refer to Ref. [7.1]. Contained in this document are suggested indicators of AMP effectiveness, stated as results-oriented criteria. Those relevant to the CCB are summarised in Table 7.1.

Existing ageing management programmes at CCBs have generally focused on managing the effects of degradation (i.e., the AMP is based on periodic inspection or monitoring of the structure, with remedial measures being implemented to deal with any observed degradation before serviceability is lost). This reactive approach is recognised as generally being cost effective. However, for inaccessible parts of the structure, where detection of degradation would be difficult, or where repair of any degradation would be costly, it would be appropriate to monitor and, if necessary, control the environment or potential stressors that could lead to degradation.

7.1. UNDERSTANDING AGEING

Understanding relevant ageing mechanisms and their potential impact on the CCB is the key to an effective, optimised AMP. As Figure 7.1 shows, this links to all components of the programme. For example, it helps definition of:

- Parts of the containment susceptible to degradation;
- Key degradation mechanisms, their symptoms, and their potential rate of action;
- Impact of degradation on the containment's ability to perform its safety functions; and
- Appropriate remedial action.

Developing the appropriate level of understanding is a continuous process. It builds on plant experience, in particular the performance of both the containment building and contemporaneous structures at the plant. This is underpinned by a more general awareness of concrete ageing issues.

The following sections review potential sources of data.

7.1.1. Awareness of generic concrete ageing issues

Evaluation of the plant condition always calls for informed judgements. The basis of these judgements is often a review of a generic body of knowledge, supported by detailed understanding of the mechanisms involved, and put into a plant-specific context.

Preliminary, qualitative assessments of the likelihood of degradation in a given plant environment can be made on the basis of background material given in Chapter 3. This requires information on operational conditions and containment design. In addition, feedback of generic NPP experience can help to identify potential ageing mechanisms. Feedback may be achieved through Plant Owner Groups and International Collaboration Programmes [7.2, 7.3]. Problems in sister plants may point to generic issues that need to be addressed. Information on ageing mechanisms and plant experiences world-wide is provided in Chapter 3

TABLE 7.1. INDICATORS OF AMP EFFECTIVENESS FOR A CCB

Results orientated criteria may be used to systematically assess the effectiveness of the Ageing Management Programme (AMP) for a concrete containment building (CCB). Those listed below are derived from the generic indicators given in Ref. 7.1. They should be reviewed and elaborated in accordance with current knowledge, accepted best practice, and relevant national standards.

General Attributes of the AMP
• A clearly defined and documented systematic AMP. This documentation includes: – Overall policy that defines the scope, objectives, activities, and general responsibilities for all relevant organizational units, programmes, and activities. – Methods and procedures for the conduct of activities aimed at understanding, effectively monitoring, and mitigating ageing of the CCB – Performance indicators by which the effectiveness of the AMP can be measured • Staffing and resources that are sufficient to accomplish the AMP objectives • Personnel involved with implementing the AMP should possess: – Clear understanding of their authority, responsibilities, accountabilities, and interfaces with other organisation units and regulatory authorities – Knowledge of relevant ageing phenomena, and of their potential impact on the unctional capability of the CCB and the overall plant safety and reliability – Training and qualifications to perform assigned job functions • CCB records that are accurate, sufficiently comprehensive, and readily retrievable in a form to support ageing management activities • Programme reviews to provide for peiodic evaluations of the effectiveness of the AMP
General Criteria to Evaluate the Quality of an AMP for CCBs
• Containment's safety and structural functions, and components of the containment that play a key role in maintaining these functions, have been identified and documented • Pertinent ageing mechanisms that may impact the containment's safety functions have been identified, evaluated, and documented • Operating conditions that may influence the rate of degradation of CCB components are maintained within design limits • Surveillance programme is sufficient to ensure the timely detection of any ageing process (or processes) and its (or their) potential effects • Acceptance criteria have been established to determine the need for, type of, and timing of corrective actions. (This may include specfied limits for impact of various degradation factors on a component's functional and performance requirements. Although they generally will be somewhat plant specific, design specifications, industry codes and standards, regulatory requirements, and industry experience provide sources for development of the acceptance criteria.) • Methods and criteria have been established to evaluate results obtained from inservice inspection and monitoring that enable a determination of whether: – Current condition of the CCB complies with acceptance criteria – Estimated future performance, based on trending of historical data or application of service life models in conjunction with reliability-based techniques, indicatescontinued compliance with acceptance criteria – Ambient environmental parameters and applied loads, together with their trends, are within specified operating limits • Options for remedial measures are understood
General Criteria to Assess Results Achieved by an AMP for CCBs
• Actual physical condition of the CCB is satisfactory in terms of required safety margins (i.e., integrity and functional capability is retained) – CCB condition and/or functional indicators, provided by surveillance, ISI testing or condition monitoring, and their trends conform to acceptance criteria – Ambient environment and system parameters (e.g., humidity, temperature, and pressure), and their trends are within specified limits • Relevant plant safety indicators such as maintenance preventable failures and CCB contribution to containment system unavailability have been satisfactory

and Table 6.1, respectively. Table 3.1 identifies potential degradation factors for NPP safety-related concrete structures and locations where degradation may occur.

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 data base of material performance with age in a given environment. Such data bases exist, but often contain proprietary information. With sufficient data and an understanding of the degradation processes involved, generic bounds or models may be derived that can be used in quantitative structural assessments. The sources of data include laboratory tests, and experience with both nuclear and non-nuclear structures. Data is often reported in journals and at technical conferences. Published data typically relates to a single mechanism; in applying this to a plant, care should be taken to understand potential synergies between degradation processes.

7.1.2. Assembly of key plant documentation

Details of the containment's design and construction/operational history are required for effective comparison or correlation with external experience. Experience from ageing assessments has shown that the data listed in Table 7.2 are of value. Sources include the plant records, and architect/engineer and consultant files. The data may be grouped into four types: baseline, construction and commissioning, operational history, and inspection and surveillance.

Baseline data identify specific safety and structural functions, the type and properties of materials used, and any assumed operating conditions. This information feeds into a preliminary assessment of potential degradation and locations, and gives insight into the impact that any degradation might have on functional or performance requirements. The design documentation may also include details of provisions made for ensuring the long-term integrity of the CCB (e.g., dealing with effects of creep) or of design limits (e.g., minimum prestressing loads and maximum crack widths). Results from laboratory studies also may be relevant for the purpose of design reviews. For CCBs, these may range from material tests to support concrete mix design through large-scale model tests to validate design methods and assumptions.

Construction and commissioning data enable review of the quality of the containment materials 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, with symptoms often evident at the earliest stages of a structure's life. It is important to identify locations where there were non-conformances with design, or where repairs were necessary.

Operational data provide historical information on containment loads and environmental conditions. These data should be compared to the original design basis to check for non-compliances. Operational data are of particular value as input into detailed assessments of the potential future impact of degradation mechanisms.

Inspection and surveillance data provide historical information on containment condition and a baseline against which on-going performance can be judged. This is of great value in trending progress of degradation. The data should be reviewed to confirm that any changes in the containment condition with time are stable and predictable. Also these data are

TABLE 7.2. POTENTIAL SOURCES OF PLANT DATA

Type of data	Sources	Information
Baseline	Design calculations	Design life Design philosophy Design codes/standards Material design properties Design stresses/strains Static design loadings Dynamic design loading Hazard design loading
Construction & 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
	Preoperational test records	Structural integrity test records Leakage test records Polar crane test records
Operational history	Plant operating procedures	Service loadings Environmental conditions Fault loadings Safety procedures Maintenance procedures
Inspection & surveillance	Inspection records	Visual inspection data Leakage-rate tests Ultrasonic thickness tests for liner Prestressing tendon metallurgical tests Prestressing loads
	Plant management/operatives	Plant history Maintenance history

important in monitoring the effectiveness of any remedial measures that have been implemented.

All of the above data types are used in ageing assessment, with the balance of importance between data types specific to the plant and containment type under consideration. However, data are generally limited and it is usually expensive to acquire more data (whether plant specific or generic). In building up a data base, judgement has to be exercised on the amount and type of data likely to be developed in the future. For cost effectiveness, data collection should be based on the requirements of an agreed-upon ageing management programme.

Ensuring that the data are placed in a special file (i.e., making it readily accessible) will save considerable time when evaluating containment condition. Adopting a systematic and consistent approach allows easy comparison with sister plants and also assists the identification of gaps in the data. 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. [7.4].

7.2. DEFINITION OF CONCRETE CONTAINMENT BUILDING AMP

Definition of an AMP includes documentation of relevant programmes and activities (in particular, the inspection and monitoring requirements and appropriate acceptance criteria), and a description of mechanisms used for programme co-ordination and continuous improvement. The continuous AMP improvement or optimization is based on current understanding of component ageing and on results of self assessments and peer reviews.

There are various NPP and external programmes and activities that contribute to the management of CCB ageing. They include NPP operating procedures and practices that affect the service conditions/environment experienced by the CCB (i.e., those to which the CCB is exposed), as well as inspection, monitoring, testing, condition assessment, and maintenance activities. Operating experience feedback and relevant research programmes also are important contributors.

Experience shows that ageing management effectiveness is improved by coordinating and integrating relevant programmes and activities within a systematic structure, system, and component (SSC)-specific AMP [7.1]. Safety authorities of Member States increasingly require licensees to define AMPs for selected SSCs by documenting relevant established programmes and activities and their respective roles in managing SSC ageing, and coordinating mechanisms used.

Some aspects of an AMP (e.g., documentation, record keeping, and personnel requirements) will link to the plant's overall strategy for ageing management. The needs are identified in Table 7.1, and practical guidance for their implementation is given in Ref. [7.1].

In developing the AMP for the CCB, two fundamental needs are to define an inspection/monitoring programme and criteria against which the results can be judged. These aspects are covered in Sections 7.3 and 7.4, respectively. They rely on an understanding of CCB ageing and are supported by an interim ageing assessment of the CCB. An interim ageing assessment allows optimum use of existing information on the structure, and the prioritising of ageing management activities. The process is assisted by the systematic collation of CCB records (see Section 7.1.2). Also, in setting up and implementing an AMP, as well as interpreting results, it is important that this be done under the direction of someone having the appropriate engineering knowledge.

7.2.1. Interim ageing assessment

A recommended approach is to first list all containment building components (e.g., ring girder, walls, and dome) and their function, materials of construction, environmental exposure, and applied loads. A generic list of components for light-water reactors, such as presented in Ref. [7.5], may simplify this activity. Of particular importance is the identification of those components of the containment that play a key role in maintaining its safety functions. For each of these components, identify and document:

- Any operating limits or performance criteria defined in the design.
- Potentially significant degradation mechanisms (see Section 7.1.1 for data sources that may assist, and Chapter 3 which gives a description of mechanisms and typical locations). Significance is primarily measured in terms of potential impact on the containment's ability to perform its safety functions; ability to detect degradation and potential costs of remedial action are also relevant (Chapters 4 and 5 give background information).
- Any known degradation, based on evidence from commissioning tests and any available surveillance data.

Additional details related to component identification are provided in Refs [7.6] and [7.7].

In the absence of historical data on plant condition, an inspection is recommended to gain familiarity with the plant and to assess the current condition. This takes the form of a preliminary visual inspection of all accessible areas and involves the following items.

- A general assessment of the condition of the key structural elements, with emphasis on the detection of symptoms of likely ageing mechanisms.
- An appreciation of any practical or safety-related limitations on access that will impose restrictions and thus influence the inspection strategy. Cycles of plant operation may lead to changing conditions of access.
- Identification of elements or materials to be selected for detailed study.
- Discussion with operatives on history of facility, maintenance history, and other related information.

The objective of this interim ageing assessment is to identify the critical locations in the containment building, based on a systematic assessment of each of the key containment components. This helps to focus subsequent, more detailed activities.

7.2.2. Programme definition

There is a need to define inspection methods, frequency and location. Appendix I provides details on various approaches taken by utilities world-wide. The inspection methods that have been 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, the results from automated monitoring have proven to be very effective in demonstrating continued compliance with design. Where appropriate, integrated leakage-rate tests provide demonstration of the containment's overall leaktightness.

As many of the manifestations of concrete degradation appear as visible indications on exposed concrete surfaces, the general condition of the containment is often assessed using visual inspection. 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 symptoms of ageing, particularly for concrete structures having large exposed surface areas, such as a containment building.

For ease of plant access and reduced radiation and temperature levels, inspections related to the CCBs are generally performed during outages. Inspection intervals or frequencies are determined based on requirements contained in codes and standards (e.g., Refs [7.8] and [7.9]). Early inspections provide confidence in quality of design and construction. Through-life inspections provide valuable data for trending containment behaviour, and give confidence in ageing performance.

Some utilities are using detailed interim inspection of selected areas/components within the containment to confirm plant condition, and reducing the frequency of general inspections to 10 year intervals. The selected locations may include known defects (which are monitored to check activity), or critical locations identified during the interim ageing assessment (See Section 7.2.1). 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, and
- Below-grade wall surfaces.

The more detailed inspections also may involve routine use of nondestructive techniques (see Chapter 4) to provide quantitative data for trending behaviour of the critical areas. For example, regular monitoring of half-cell potentials may be carried out if there is a threat of steel reinforcement corrosion. Remote techniques, linked to data logging equipment, are most useful when supported by software automating the mechanics of data collection/interpretation.

The appropriate inspection frequency will be a function of ease of inspection, and the likely rate of degradation and its potential consequence. Reference [7.10] indicates that a wide range of frequencies exist for conducting more detailed assessments (e.g., leakage-rate tests were conducted at intervals ranging from weekly to ten years). Frequencies also may be varied through plant life, perhaps increased if inspection is being carried out as part of an evaluation programme for an observed defect, or reduced following evidence of stable (predictable) behaviour. The appropriate frequency should be assessed on the basis of engineering judgement, with the overall aim of ensuring detection of degradation before minimum performance requirements are reached.

For inaccessible concrete areas, greater reliance is placed on characterisation of its condition 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 the concrete surfaces, tests are carried out on other components of the containment system. 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 should be included as part of the containment's AMP.

7.3. OPERATION

Plant operation has a significant influence on the rate of degradation of NPP systems, structures, and components. Exposure of the CCB to operating conditions (e.g., temperature, pressure, humidity, radiation, and aggressive chemicals) outside design limits could lead to accelerated aging and premature degradation. Since operating practices influence CCB operating conditions, NPP Operations Staff has an important role within the ageing management programme to minimize age-related degradation of the CCB by maintaining operating conditions within design limits.

Operation of plant systems and testing of the CCB and its components according to procedures, and record keeping of relevant operational data (e.g., environmental conditions, test conditions, and results) also are essential for an effective CCB ageing management programme. In particular, it is prudent to attempt to control and monitor the operating environment of inaccessible parts of the CCB (e.g., basemat and embedded portions of containment liner) where detection and repair of degradation would be difficult and costly.

7.4 INSPECTION, MONITORING AND CONDITION ASSESSMENT

Inspection, monitoring, and condition assessments are essential elements of an effective ageing management programme. Knowledge gained from these activities can serve as a baseline for evaluating the safety significance of any damage that may be present and defining in-service inspection programmes and maintenance strategies.

7.4.1. Inspection and monitoring

The CCB inspection and monitoring activities are designed to detect and characterise significant component degradation before the CCB fitness-for-service is compromised. Together with an understanding of the CCB ageing degradation, the results of the CCB inspections provide a basis for decisions regarding the type and timing of maintenance actions to correct detected ageing effects. Also, these results can impact decisions regarding changes in operating conditions and practices to control significant ageing mechanisms.

Current inspection and monitoring requirements and techniques for CCBs are described in Chapter 4. In general, the rigor and extent of the inspection increases as the CCB develops problems. Normally, a visual inspection of accessible surfaces of the CCB is conducted. Visual inspections are supplemented by nondestructive and destructive tests in areas exhibiting distress.

It is extremely important to know the accuracy, sensitivity, reliability, and adequacy of the nondestructive methods used to identify and evaluate the particular type of suspected degradation. The performance of the inspection method(s) must be evaluated in order to rely on the results, particularly in cases where they are used as part of a fitness-for-service assessment. Inspection methods capable of detecting and quantifying expected degradation are therefore selected from those proven by relevant operating experience. Information on accuracy and capabilities of nondestructive methods for inspection and monitoring of CCB ageing are also presented in Chapter 4.

Systematic and effective record keeping is an important part of the inspection process. It is this data that underpins evaluation of the current condition as well as estimates of future performance.

For visual inspections, permanent records are generally made of the condition of concrete at the time of 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 discoloration, erosion, cavitation, seepage, condition of joint and joint materials, corrosion of reinforcement, 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 extent of degradation should, where possible, be backed up by quantitative measurements.

Quantitative data provided by other testing and monitoring techniques also should be recorded appropriately. 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. [7.4].

7.4.2. Condition assessment

An ageing management programme applies acceptance criteria to results from routine inspections to confirm that the current plant condition is acceptable, and through historical trending of results (plus other data when available) to estimate future performance.

7.4.2.1. Acceptance criteria

Assessment of performance must be carried out against established acceptance criteria. At a fundamental level, this may be identified as the continued ability to meet the original design specification requirements. This would usually involve assessment against original design standards and codes of practice. However the relevance of more recent design standards must also be given consideration, particularly if changes to standards or practice have resulted from improved knowledge of deterioration processes or identification of new failure mechanisms.

Although some acceptance criteria may be found in design codes, these criteria tend to be limited and often wide ranging (see for example Tables 5.2 and 5.3 that present information on ranges of "permissible" crack widths). Determination of a common acceptance criteria is difficult because of differing materials, functional requirements, behaviour characteristics, and other conditions.

A tiered approach to acceptance of an existing condition is often used. This approach uses findings of an evaluation such as a condition survey to trigger various levels of response. In ascending order of severity, responses might be:

- Condition is acceptable and requires no further evaluation;
- Condition requires more frequent inspection to assess whether active;
- Condition requires engineering evaluation, which may lead to mitigation or repair; or
- Condition requires urgent action.

A further refinement of this tiered approach is the development of damage charts for selected mechanisms/test methods. These charts provide a graphical representation of the relationship between pertinent factors (e.g. crack width and environmental exposure) and recommended actions. For example, Fig. 7.3 indicates one form of a damage state chart that has been proposed to relate environmental exposure, concrete crack width, and necessity for additional evaluation or repair [7.11]. Reference [7.6] presents examples of such charts for some of the common degradation mechanisms (e.g., cracking, corrosion, and loss of cover). It should be noted that damage-state charts should not be applied without technical judgement.

Specific performance requirement values for plant components may be included in the design documentation (e.g. minimum prestressing loads and maximum crack widths); these values should be clearly identified. Note that these requirements may define both targets (which could be temporarily exceeded) and limits for development of a tiered approach to acceptance of an existing condition. Where no guidance is given in design documentation, the criteria delimiting the tiers should be established on the basis of engineering judgement for groups of structural components (groupings being defined on the basis of either structural/safety function, or of environmental exposure).

7.4.2.2. Evaluate current condition

The results of inspection and monitoring enable the operator to confirm fitness-for-purpose of the containment building. This will be carried out on the basis of comparison with the design requirements and any predefined acceptance criteria.

Symptoms of degradation may require additional evaluation for which various tools and options are available (e.g., test and evaluation techniques). Key input to the assessment of degradation is an understanding of ageing and access to key plant documentation. The rigour of the evaluation will reflect the potential significance of the defect. Key issues that need to be addressed are:

- Whether reactor safety is threatened (depending on the severity of the fault, the reactor may either be shut down to effect a repair or operated at a reduced load while action is taken);
- Possible cause(s) of degradation (Chapter 3 gives an overview);
- Extent of damage (Chapter 4 highlighted the range of nondestructive testing options currently available, these may be supported by selective destructive tests);
- Rate at which damage is progressing, based on knowledge of ageing (Chapter 3) supported by more frequent inspection or monitoring if appropriate; and
- Significance of damage in terms of structural or safety significance (based on review of design requirements).

In general, a documented assessment of an unacceptable condition and its significance for the long-term operation of the reactor would need to be prepared. This would seek to show that reactor operation can safely be continued. It would normally be underwritten by measurements on the plant, calculations (including a revisit of the design calculations), and specialist assessments of potential long-term impact. Where the consequences are far reaching, or the process is not adequately understood, it also may be necessary to commission material test programmes or simulations (e.g., mock ups or accelerated ageing studies). The outcome from this evaluation may include recommendations for maintenance, discussed in Section 7.5.

7.4.2.3. Estimate future performance

A single "one-off" 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). Comparison with recorded or estimated data at the time of construction, or historical data, are needed to deduce the rate of deterioration and to establish trends.

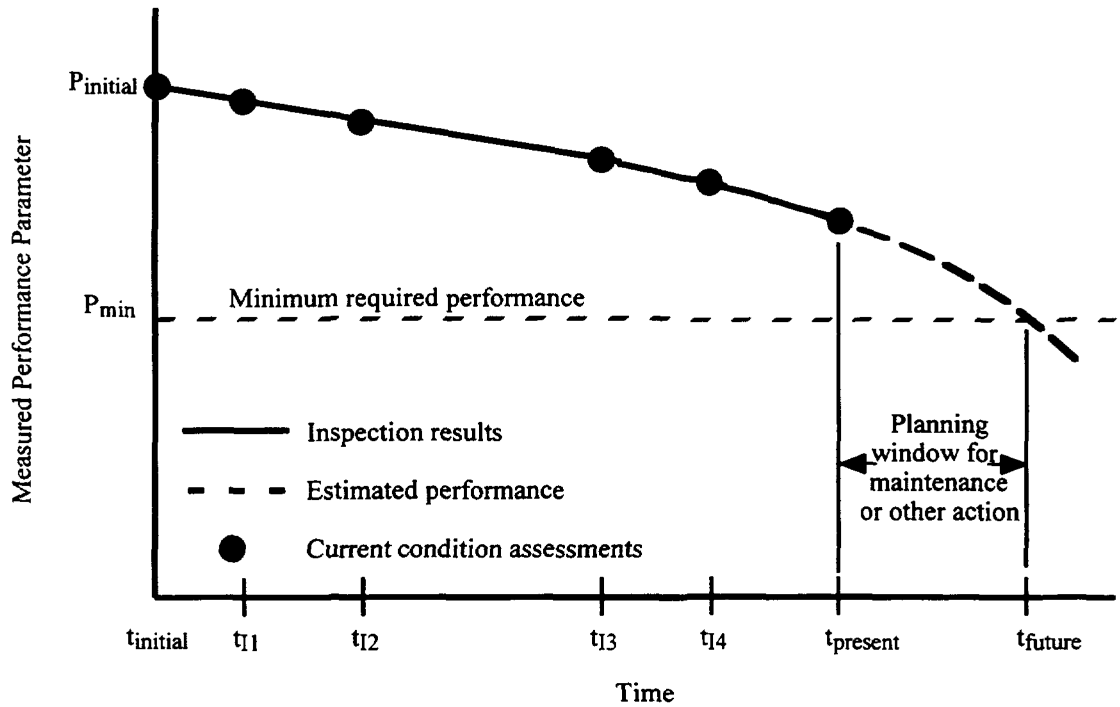
The process of estimating future condition provides an important distinction between an ageing management programme and a series of "ad-hoc" inspections. A comparison of estimated behaviour with performance limits is used to indicate that the containment condition will remain acceptable until the next 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 7.2 presents an example of how performance trending can be used as part of an AMP. Also shown in the figure is an actual example of how maintenance was used to reestablish acceptable performance of a reactor building.

Estimates of future performance are generally based on extrapolation of results from earlier surveys. In the event of degradation being present, an understanding of the processes involved is fundamental as the rate of degradation will vary according to the mechanisms involved (e.g., diffusion or reaction controlled). Further, more frequent inspections may be planned to monitor the actual rate of degradation in the structures.

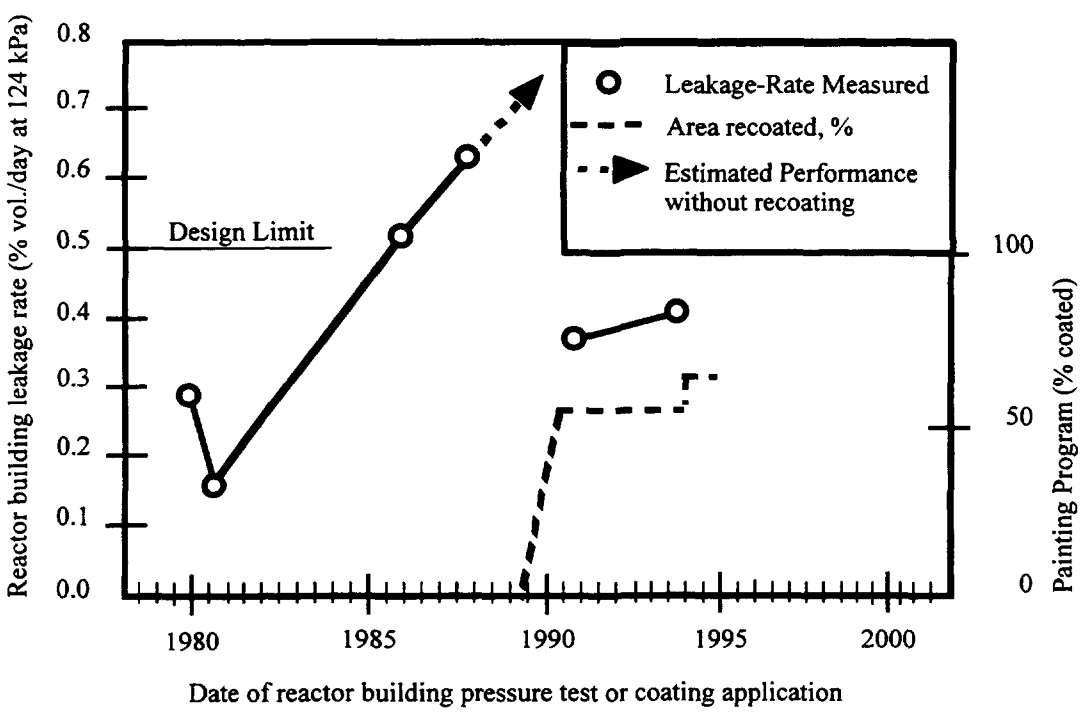
7.5. MAINTENANCE

Most maintenance and remedial work is implemented in response to an identified defect in the structure. Table 6.1 gives some examples of plant degradation and repair experience. Depending on the degree of degradation and the residual strength of the structure, the objective of a remedial measures programme might be any one, or a combination, of structural, protective, or cosmetic. Typical options that would be considered in response to unacceptable plant degradation are:

- Enhanced surveillance to trend progress of deterioration. This is the initial approach adopted as part of the evaluation process during the 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 in the shorter term while repairs are planned).



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. 7.2. Examples illustrating use of performance trending as part of an AMP and use of maintenance to reestablish acceptable performance.

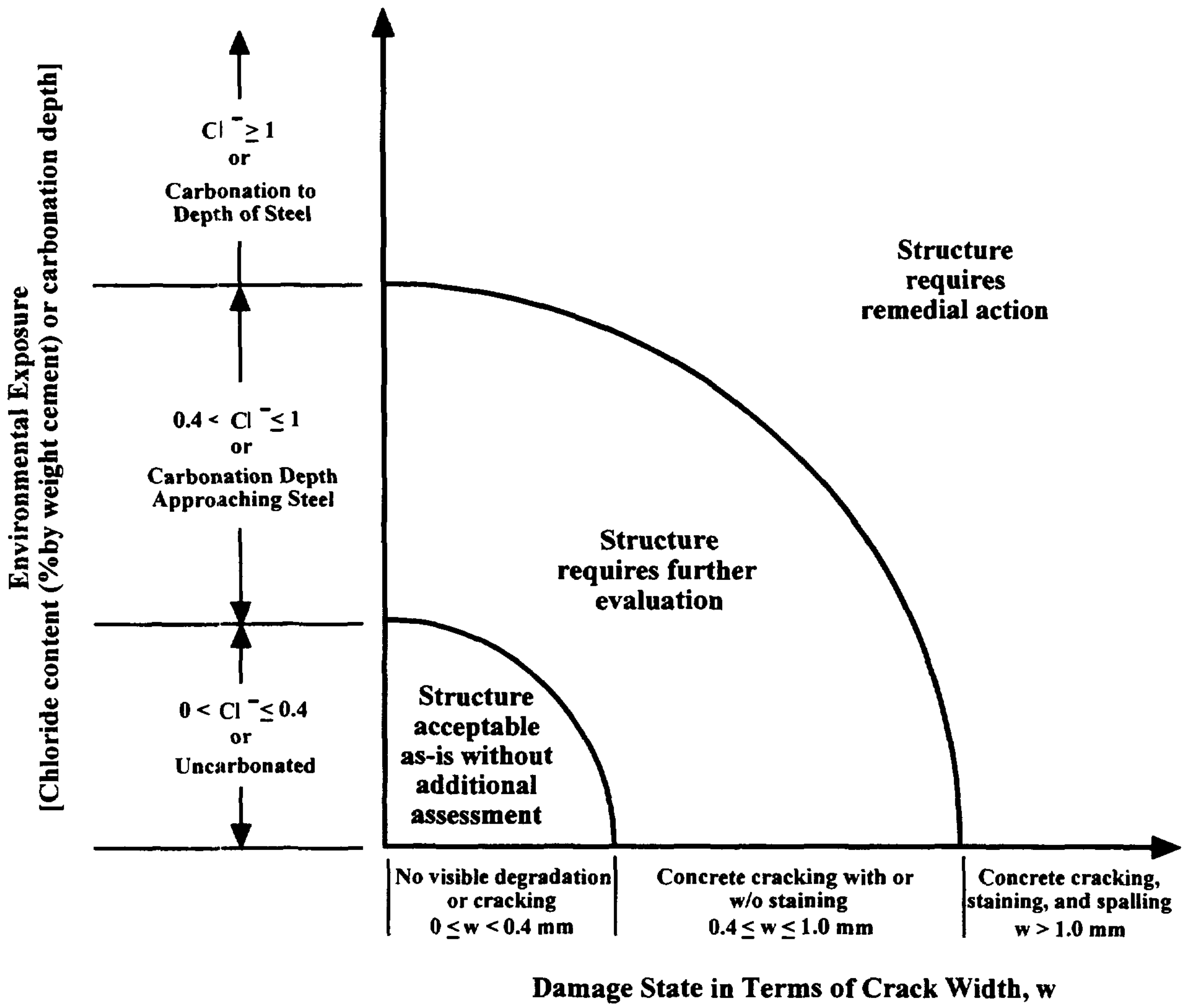


FIG. 7.3. Damage state chart relating environmental exposure, crack width and necessity for additional evaluation or repair [7.6].

- Local repairs to restore parts of a structure to a satisfactory condition.
- Replace component.

Chapter 5 reviews typical repair materials and techniques for concrete structures.

7.6. OPTIMIZING

Section 7.2.2 discussed the use of interim inspections that focused on selected parts of the containment. Generic inspection programmes (e.g., Ref. [7.8]) do not currently give guidance on prioritising inspections for critical locations within the containment. However, a prioritised inspection programme will lead to cost and time savings.

Optimised surveillance relies on an understanding of the ageing mechanisms, which permits the listing of structures that are most susceptible to ageing, the mechanisms involved, and the significance of potential degradation. This may involve the use of a formal quantitative ranking system (e.g., Refs [7.5] and [7.7]), or a simple evaluation of containment materials, exposure conditions, and documented performance.

The activity may be simplified by using generic data such as provided in Chapter 3 to screen degradation processes and containment locations, highlighting 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 also have been developed (e.g., Ref. [7.12]) for use as a tool to optimise inspection frequencies based on the significance of ageing to overall plant risk over the lifespan of the containment. These methods take into account the degradation mechanisms, inspection, and remedial measures. They have the advantage of feeding directly into an overall risk assessment for the plant, and of quantitatively assessing the possibility that degradation of civil engineering structures may impact the performance of other 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. Although not being used routinely today, this work points the way to developments in the near future.

It is important to note that the AMP is not a static programme. As illustrated in Fig. 7.1, it should 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 evolutions in safety thinking and developments in current knowledge and technology relating to both concrete durability and assessment techniques should be incorporated into future versions of the programme.

REFERENCES TO CHAPTER 7

[7.1] INTERNATIONAL ATOMIC ENERGY AGENCY, Implementation and Review of Nuclear Power Plant Ageing Management Programme, IAEA, Vienna (in preparation).

[7.2] DONSEZ, D., FAUDEMAY, P., HOMOND, P., "A Cooperative Database System Based on Workspace Hierarchy," Industrial Information and Design Issues, Springer, Berlin (1966).

[7.3] SENI, C., IANKO, L., "An International Database on Nuclear Concrete Structures Ageing", Industrial Information and Design Issues, Springer, Berlin (1966).
[7.4] INTERNATIONAL ATOMIC ENERGY AGENCY, Data Collection and Record Keeping for the Management of Nuclear Power Plant Ageing, Safety Series 50-P-3, Vienna (1991).
[7.5] HOOKHAM, C.J., Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants, ORNL/NRC/LTR-90/17, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, TN (1991).
[7.6] HOOKHAM, C.J., "In-Service Inspection Guidelines for Concrete Structures in Nuclear Power Plants", ORNL/NRC/LTR-95/14, Lockheed Martin Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, TN (1995).
[7.7] JUDGE, R.C.B., "Classification of Structural Component Degradation Mechanisms for Containment Systems", Proc. Third Int. Conf. on Containment Design and Operation, Toronto (1994).
[7.8] 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 edition with Addenda, New York (1995).
[7.9] CANADIAN STANDARDS ASSOCIATION, N287 Series, CAN/CSA, Toronto, (1991-1994).
[7.10] Summary Results of the Survey on Concrete Containment Ageing, internal IAEA report, 1995.
[7.11] NAUS, D.J., OLAND, C.B., ELLINGWOOD, B.R., Report on Aging of Nuclear Power Plant Concrete Structures, NUREG/CR-6426 (ORNL/TM-13148), Lockheed Martin Energy Research Corp., Oak Ridge National Laboratory, Oak Ridge, TN (1996).
[7.12] ELLINGWOOD, B.R., Aging Effects on Probabilistic Risk Assessment, NUREG/CR-6425 (ORNL/TM-13149, Lockheed Martin Energy Research Corp., Oak Ridge National Laboratory, Oak Ridge, TN (1996).

8. CONCLUSIONS AND RECOMMENDATIONS

Based on results of the Co-ordinated Research Programme which include survey questionnaire results from nuclear power plant owners and operators, experience of CRP participants, and information provided throughout this report, several conclusions and recommendations can be made.

8.1. CONCLUSIONS

- The performance of the reinforced concrete structures in nuclear power plants has been good, with the majority of the identified problems initiating during construction and being corrected at that time. However, as these structures age, incidences of degradation due to environmental stressor effects are likely to increase the potential threat to their functionality and durability. The most commonly observed form of degradation has been concrete cracking. Degradation factors of primary concern would be corrosion of steel reinforcement due to carbonation of the concrete or presence of chloride ions, excessive loss of prestressing force, excessive containment leakage due to failure of the metallic or nonmetallic boundary (lined) or excessive concrete cracking (unlined), and leaching of concrete.

- Techniques for detecting the effects of concrete ageing (i.e. inspection and performance monitoring) are sufficiently developed to provide vital input for evaluating the structural condition of concrete containment buildings. Periodic application of these techniques provide data that can be used to trend performance and form the basis of an ageing management programme. Areas of concern where these techniques require additional development include massive members that contain large quantities of steel reinforcement, such as the basemat, and members that are inaccesible, such as portions of the steel pressure boundary that are embedded in concrete and foundations.

- Methods for conducting condition assessments of reinforced concrete structures are fairly well established and generally start with a visual examination of the structure's surfaces. Despite the usefulness of performing condition assessments in maintaining the containment building's fitness-for-service, with a few notable exceptions, the majority of the responses to the survey questionnaire sent to plant owners and operators indicated that they do only limited evaluations beyond those mandated by codes and standards (e.g. visual examinations, leakage-rate testing, and prestressing tendon assessments). Application of supplemental examinations and testing have primarily been associated with assessments of degradation occurrence or suspected occurrence.

- Maintenance and repair techniques for concrete structures are well established and when properly selected and applied are effective. At present no codes or standards are available for repair of reinforced concrete structures, although some are being developed. Criteria that may be used to determine when a repair action should be implemented are limited (e.g. parameters that relate damage state such as crack width to environmental exposure). Data on the long-term effectiveness or durability of remedial measures are required. Effective implementation of a repair strategy requires knowledge of the degradation mechanisms, the environment of the structure at the macro and micro level, proper preconditioning of the structure to be repaired, correct choice of repair technique and material, and quality workmanship.

- Review of international practice has shown that many utilities worldwide have already responded to the potential for age-related degradation of CCBs and have implemented ageing management programmes. These programmes generally adopt an approach in which any effects of ageing are managed (as opposed to modifying operational environments to control the onset/rate of degradation).

- A characteristic of the most effective AMPs was the clear definition and documentation of a systematic programme of activities aimed at understanding, effectively monitoring, and mitigating ageing effects. A particular feature was the routine trending of surveillance and test data to estimate future performance of the CCB. This has value in ensuring continued CCB's fitness-for-service and hence plant availability.

- Drawing on international experience, a framework for ageing management of CCBs has been defined (Fig. 7.1). The proposed approach is consistent with existing IAEA guidelines. An understanding of the issues involved (based on both plant specific knowledge and external experience on concrete behaviour) is the basis for an effective AMP. The AMP consists of the following key elements: (1) *Definition of the AMP* to co-ordinate and integrate ageing management activities and to identify the inspection and monitoring requirements and acceptance criteria; (2) *Operation* of plant within design limits to minimize age-related degradation, in particular that which is error-induced; (3) *Inspection, Monitoring and Condition Assessment* to characterize significant component degradation before fitness-for-purpose is compromised; and (4) *Maintenance* to correct any unacceptable degradation (i.e. to manage ageing effects). Technical guidance is provided for each of these tasks, together with suggested indicators for measuring the overall effectiveness of an AMP for CCBs.

8.2. RECOMMENDATIONS

- Where effective AMPs have not already been implemented for CCBs, plant owners/operators should seek to integrate and build on their existing ageing management activities (e.g. inspection, testing and maintenance) to develop a systematic AMP that incorporates the key features of understanding ageing, information management, and condition assessment.

- A proactive approach should be included as part of the overall programme to manage ageing of CCBs. A proactive approach is aimed at preventing premature degradation of CCB components that can be caused by high temperature, pressure, humidity and radiation; aggressive chemicals and mechanical wear resulting from poor plant operation and maintenance.

- Utilities and regulators should work together to develop guidelines that provide a generic framework for AMPs of CCBs. Results of this CRP may be used as the basis for such guidelines.

- Individual plant AMPs can be enhanced by drawing on wider experience of plant ageing. Exchange of information at the international level that has been initiated by organizations such as the IAEA and OECD/NEA should be combined and augmented. This may involve developing and implementing international data bases on ageing experience utilizing a common format and covering both CCBs and other safety-related concrete structures.

- Material developed under this CRP should be extended to address other safety-related concrete structures. Much of the information developed under this activity is applicable to these structures.

- Issuance of new or revised design guidelines should address ageing considerations (e.g. durability, maintenance, inspection and "safe store") through incorporation of information such as provided under this CRP.

Appendix I

CURRENT AGEING MANAGEMENT PRACTICES AND OPERATING EXPERIENCE OF SEVERAL MEMBER STATES

The inspection and monitoring programmes used to assess concrete containment buildings (CCBs) by Member States vary significantly from country to country. Examples of several of these programmes and associated ageing management experience are summarised below.

I.1. BELGIUM

The seven PWR nuclear power units of Belgium are operated by Electrabel S.A. The units were built between 1960 and 1985 according to requirements of US regulations [e.g., American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code]. Each unit has its own inservice inspection programme that includes visual inspection of the prestressed concrete and liner, lift-off tests of the tendons, and monitoring the evolution of strain in the prestressed concrete. These programmes have been updated to take into account inservice inspection requirements contained in subsections IWE and IWL of the ASME Code. Due to a defect of several of the strain gauges of the Doel 3 containment, new vibrating wire strain gauges are being installed on the face of the concrete prestressed concrete shell.

I.2. CANADA

The Canadian vendor of nuclear power stations is AECL which is known as Atomic Energy of Canada Ltd. AECL is a crown corporation that is wholly owned by the Canadian government. The Canadian authority that regulates and licences NPPs is the Atomic Energy Control Board (AECB). The CANDU stations are owned and operated by a group of Canadian utilities — Ontario Hydro, New Brunswick Power Corporation, and Hydro Quebec. Currently there are 22 nuclear reactors in Canada. Activities addressing the ageing management of concrete structures are being carried out by the Canadian Utilities, AECL, and AECB.

I.2.1. Ontario Hydro

Ontario Hydro operates 20 CANDU reactors in 5 multi-unit stations. The oldest unit has been in service for over 25 years. Containments at these stations are subjected to regular visual inspections (5- to 10-year intervals) and pressure tests to verify integrity, meet operating standards, and satisfy regulatory requirements. Current requirements for inspection of post-tensioned structures are specified in "Inspection Requirements for CANDU (draft)," CAN/CSN-281.7-M94 Standard. Appendix A to this standard addresses prestressed concrete containments with grouted tendons and Appendix B covers prestressed concrete containments with ungrouted tendons. The Code requires that prestressing systems used as principal reinforcement in concrete structures be evaluated to determine the effects on the structural integrity of time-dependent shrinkage and creep of concrete, corrosion of tendons, and stress relaxation.

Ontario Hydro's overall approach to life management has objectives of ensuring that appropriate maintenance, inspection, or surveillance programmes are in place for station systems, structures, and components (SSCs); assuring the ongoing quality and effectiveness of the programmes implemented; incorporating operating experience to continuously improve the ageing management programme and respond to new concerns as they occur; and carrying out periodic programme reviews and initiating modifications in programmes not fulfilling their objectives. In 1991 Ontario Hydro initiated a Nuclear Plant Life Assurance (NPLA) Programme with objectives of (1) maintaining the long-term reliability, availability, and safety of their nuclear plants during the normal service life of 40 years (life assurance and life management); and (2) preserving the option of extending the life of the plants beyond the currently assumed service life of 40 years (life extension) [I.1]. The programme focuses on understanding and managing degradation of a relatively few major components that are most critical to the long-term reliability, safety, and life of the plant since they can not easily and economically be replaced. Major civil structures that were identified under the NPLA programme include the vacuum building, calandria vault, reactor building, and cooling water intake structure. Scoping reports to identify key components, potential degradation mechanisms, the current condition, and required activities to ensure continued performance, have been comp eted for the Pickering and Bruce stations. While the performance of the civil structures generally has been good, there have been incidences of localised material deterioration and increases in leakage rate during pressure tests [Ref. I.2]. These incidences primarily have been associated with construction flaws, unanticipated environments, material discontinuities, and short-lived materials (e.g., coatings). Examples include cracks in Pickering A vacuum building floor due to premature exposure to cold weather, reactor building dome cracks noted immediately after construction (probably shrinkage cracks), and leaks from reactor shield tanks due to weld flaws and deteriorated sealants. Other incidences of degradation include elastomeric seal failures due to uneven clamping or construction flaws, corrosion of intake structure steel components by lake water, cracked concrete near boilers due to elevated temperature, and attack of sealants and coatings inside the vacuum building by sulfate-reducing bacteria from lake water used in safety systems. These problems are being addressed through research programmes on concrete repair materials for cracks and overlays, concrete behavior (leaching, elevated-temperature response, carbonation rates), ageing of elastomers, and analysis of inspection information. Procedures have been written for standardised inspection programmes, and testing of crack repair materials has been completed.

I.2.2. AECL

The Canadian single-unit stations are Gentilly 2 and Point Lepreau, both being in operation for 13 years; however, the containments at these stations were constructed 20 and 22 years ago, respectively. New Brunswick Electric Power Corporation, Hydro Quebec, and AECL in 1994 initiated a common comprehensive plant life management study addressing these stations. Included in this study are activities evaluating the ageing of the concrete containments. Enhancements are being proposed to the current life management programme.

Ageing concerns at AECL are addressing in particular methods to control the leak tightness of the containments. Two materials are being considered to enhance the performance with respect to leaktightness of containment buildings – polyurethane and fiber reinforced mortar. The reactor building concrete containment walls at the Gentilly 2 NPP and Point Lepreau Nuclear Generating Station initially had an epoxy liner on the inner surface to provide a leaktight barrier and assist in decontamination. Although the initial containment leakage-rate test results at these stations were satisfactory, subsequent results have shown a significant increase with time (e.g., 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 in 24 hours). The majority of leakage was attributed to leakage through the containment walls where cracks have occurred and increased in number and magnitude with time. The initial epoxy liner was considered rigid and not capable of bridging the cracks and is being replaced with an elastomeric polyurethane liner. This replacement has resulted in a significant reduction in the containment leakage rate.

AECL also is carrying out safety research programmes as part of the CANDU Owners Group (COG). COG is a partnership of the Canadian nuclear utilities and AECL. The programmes are developing effective repair materials for concrete containments (1995–1996), evaluating the effects of elevated temperatures on concrete materials (1993–1997), evaluating potential structural/functional issues associated with reactor building containment leakage-rate test intervals (i.e., does repeated pressure testing accelerate ageing) (ends 1997), and developing a rating and assessment of the ageing of NPP concrete structures/components (1995–1999).

I.2.3. AECB

The AECB is addressing ageing of concrete structures through development of a reliability-based inspection programme for reinforced concrete structures at CANDU stations. It is required that the prestressing systems used as principal reinforcement be evaluated to determine the effects on structural integrity of time-dependent shrinkage and creep of concrete, corrosion of tendons, and tendon stress relaxation. The systematic approach for appraisal of the structural reliability of concrete containment structures being developed is based on information collected from grouted prestressed concrete beams, inspection of ungrouted prestressed concrete structures, and visual inspections during pressure tests of containments. The inspection and repair approach is aimed at maintaining the target reliability of the containment at an acceptable level throughout its service life. The technical approach involves four steps: (1) development of the time-dependent reliability analysis, (2) updating the reliability analysis based on inspection data (Bayesian approach), (3) factoring into the analysis the effects of sample size (statistics), and (4) determination of the optimal inspection frequency intervals. The approach has been used to define the number of prestressing tendons in a civil engineering structure that need to be inspected to assure, for a given coefficient of variation, that the percentage of failed tendons in the entire structure will be less than a predicted percentage of the entire population. The approach is currently being applied to nuclear power plant-specific concrete structures.

I.3. FRANCE

Currently France has 54 PWR and 2 fast reactor nuclear power plants that supply over 75% of France's electricity. In France no limit is set on the plant operating licence period but the utility has to obtain a permanent renewal of its licence subject to numerous and continuous justifications (e.g., safety re-evaluation) [Ref. I.3]. Research and development programmes have been conducted in support of continuing the service of the French nuclear power plants by Electricité de France (EdF), which is the operator, and also by the French Atomic Energy Commission (CEA). The Institut de Protection et de Sûrete Nucléaire (IPSN) is the technical support body of the safety authority that analyses documentation, in particular that provided by the licensees to determine if operating conditions are acceptable.

Starting in 1986, a systematic work programme known as the "Lifetime Project" was implemented to (1) evaluate as accurately as possible the potential for technical and economic longevity of each of France's NPPs, and (2) take the necessary measures to provide improvements wherever possible and to do so effectively while maintaining a satisfactory safety level [I.4]. The project is run at EdF and addresses four topics: (1) ageing of materials,

(2) influence of changes in operating conditions, (3) safety aspects, and (4) economic aspects. Activities primarily address a limited number of items considered to be most critical from a safety or economic viewpoint. The basic approach is to characterise ageing of each of the sensitive components and to address problems as they occur. This approach is supplemented by a large data base on the in-service performance of the components and experience derived from standardised designs. The CCB is a sensitive component.

Prior to commissioning a unit, a containment is subjected to leakage rate and structural integrity tests to the design pressure. These tests are repeated after the first refueling (≈ two years) and every 10 years thereafter. Also, the containments, with the exclusion of base mats, are inspected every 10 years for cracks, with recent inspections being recorded on video tape. The recordings provide a more efficient method for documenting concrete distress, such as cracks, and can be used as a reference for future examinations to indicate changes since the previous inspection. The first unit of each plant at a site is instrumented extensively to obtain results for comparisons to prior analytical studies, to verify that the concrete globally remains in a state of compression (essentially crack free), and to account for changes in material properties with time, (e.g., concrete creep and shrinkage, and relaxation of prestressing steel). Instrumentation utilised includes vibrating-wire strain gauges cast into the concrete in the dome, near penetrations, near the junction between the wall and base mat, at different elevations in the cylinder, and in the base mat itself; thermocouples to provide information on thermal gradients and temperatures where vibrating-wire strain gauges are located (i.e., make temperature corrections for the strain gauges); 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 4 tendons that are ungrouted in the first plant unit; 12 to 20 hydraulic leveling pots (optical levels) to determine tilt of the base mat; and bench marks for determination of base mat distortions. Instrumentation measurements are made at three-month intervals unless an abnormal event occurs. Readings are then taken at weekly, bi-weekly, or monthly intervals. This large data base, that has been accumulated for up to 15 years at some plants, not only enables the performance of these structures to be demonstrated, but also provides information for use in trending analyses should life extension of these structures be a consideration. Recent developments relative to the containment instrumentation systems have been directed at more automated data collection and analysis. Two ageing-related problems have been detected as a result of testing and monitoring of instrumentation: corrosion of the steel liner and larger than anticipated loss of prestressing force.

Corrosion of the 6-mm-thick liner of the 900 MW(e) prestressed concrete containment vessels has occurred in several of the containments in the CP Series. The corrosion occurred in two areas: general area along the entire circumference of the conical portion of the liner adjacent to the upper portion of the concrete base mat, and beneath the construction joints of the 1-m-thick concrete-base floor slab. Corrosion of the steel liners was first noticed at a plant age of 10–15 years. Corrosion had occurred over a 20-cm section of the liner extending from the location of the waterstop between the wall and the 1-m-thick concrete slab over the liner toward the base mat (inaccessible region). Corrosion had progressed to the state that 1-cm-diameter holes penetrated the liner in this region. Although the containments passed the integrated leakage-rate tests, it was observed at test conclusion that water containing corrosive substances was stagnating in some of the pressurisation channels that were used during construction for inspection of the welds used to join the liner plate sections over the foundation raft. Corrosion occurrence was attributed to a breakdown in the waterstop in conjunction with the presence of high humidity during construction and operation. The 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 pressurisation channels were filled with cement grout, the space between the liner and floor slab was filled with a corrosion inhibitor (wax), and a new waterstop installed. The new waterstop consisted of a composite elastomeric material that was shielded by a metallic sheet attached by bolts so it periodically could be removed for inspections. Liner corrosion also has been observed at a location at the bottom of a joint in the base floor slab (CP units only). The corrosion had penetrated through about 3 mm of the 6-mm-thick liner and was attributed to decomposition of the joint seal and the presence of water with a pH ≈ 5.

In two of the prestressed concrete containments, tendon force vs time curves obtained from the control ungrouted tendons are showing larger than expected losses in prestressing forces. The losses are attributed to creep and shrinkage of the concrete, with one plant exhibiting twice the design value. Although the prestressing force vs time results from these plants are approaching the design curves (lower tendon force limit), the forces remain at acceptable levels, and the slope of the curves is following the same shape as obtained from plants having prestressing forces in the normal range [i.e., the force vs time curves are flattening out (embedment instrumentation is showing concrete creep to be leveling off) so they should 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, modeling studies are being conducted to address the delayed behavior of the PWR containments (i.e., creep and shrinkage). Prediction of the time-dependent response of concrete (creep in particular) is important because of its potential impact on the prestress level that in turn possibly affects the leaktightness of containments. Since all but the four monitoring tendons are grouted in the French containments, the possibility of re-establishing the required prestressing forces is unlikely. Considerable efforts are underway at EdF to develop methods to predict concrete creep and shrinkage.

I.4. INDIA

Nuclear safety-related civil engineering structures are required to be designed, constructed, tested and maintained to quality standards such that they perform safety functions under the effect of postulated accidents and normal operating conditions throughout their service life [I.5]. Suitable measures need to be taken relative to durability and ageing management at all major stages of engineering for these structures to withstand the effects of various deteriorating processes, including those associated with extreme conditions (e.g., prolonged elevated temperature and prolonged irradiation). The major stages of engineering include design, construction, and operation. In addition, organisational measures need to be taken for effective implementation of the engineering measures through a suitable quality assurance programme. Relative to the design stage, instrumentation for monitoring the structural integrity or other serviceability-related parameters should be identified so that suitable provisions can be made for their inclusion. Also, at this stage proper access and working space should be planned for inspection and maintenance, durability should be addressed based on both the internal and external environments, and proper detailing should be used to avoid things such as wide crack widths. Factors considered at the construction stage include constituent material selection, concrete mix design, minimisation of local climatic effects on the concrete properties and construction operations, and quality assurance and inspection measures. An effective maintenance programme is to be developed to ensure that the structures are capable of fulfilling their safety functions. Relative to ageing effects, some localised cracking and spalling of cover concrete due to steel reinforcement corrosion has been observed on the exterior faces of the reinforced concrete secondary containment at the Tarapur Atomic Power Station. Also there have been some fine shear cracks observed in these wall with evidence of leaching. The station management has been requested to conduct a condition survey at the plant and to monitor the cracks.

I.5. JAPAN

Currently Japan has 28 BWRs (2 BWRs with reinforced concrete containment vessel), 23 PWRs (5 PWRs with prestressed concrete containment vessel) and 1 GCR which are operating. In April 1996, The Ministry of International Trade and Industry (MITI) published a report showing the basic concept for nuclear power station ageing management [Ref. I.6]. A summary of MITI report on concrete structures follows.

Ageing phenomena to be considered are strength deterioration and shielding capability reduction caused by various factors. It is recognised that heat, neutron irradiation, neutralisation, salinity infiltration, and alkali-silica reaction are the major factors that cause strength deterioration; and it also is recognised that neutron irradiation (exothermic by gamma ray) is the major factor that causes shielding capability reduction. Concrete 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.

I.6. SWITZERLAND

Operating licences for the five nuclear power reactors (2 BWR, 3 PWR) in Switzerland are, according to Swiss law, not formally restricted by time-limited licences. However, the licensing authority (Swiss Federal Nuclear Safety Inspectorate - HSK) requires that the utilities report periodically on their plant's condition in order to demonstrate that safety requirements are continuing to be met. The approach used by the utilities to meet this requirement has focused on preventative measures and immediate repair of any visible degradation. For the reinforced concrete structures, this has included (1) periodic visual inspections, (2) periodic testing of special structural elements (e.g., anchorages and joint sealants), and (3) early and complete repair of any detected degradation. Examples of problems that have been experienced at the Swiss NPPs include leakage of borated water from the fuel pool, failure of waterstops, cracking of coating materials, and cracking in the roof or dome of reactor buildings. Because the older plants in Switzerland have been in commercial operation for over 20 years, HSK in 1991 required that all utilities develop ageing management programmes for their plants that address the structural condition as well as electrical and mechanical components [I.7].

In developing ageing management programmes, the HSK recommended the following actions (1) identification of potential degradation modes and areas of the plant subject to degradation, (2) listing of periodic testing and maintenance programmes that are currently in effect, (3) assessment of data provided by state-of-the-art inspection techniques, and (4) cataloging additional measures for use in surveillance and assessment of the ageing process. Initiation of these actions was through preparation of a state-of-the-art report that defined the basic problem being addressed, reviewed world-wide experience in this area, and provided the basic technical requirements for the ageing management programme that is to be developed [I.8]. Conclusions of the report were that the most important degradation mechanism was corrosion of steel reinforcement, with structural elements not accessible for inspection being of most concern (e.g., foundation and outer walls embedded in soil); inspection methods used for general civil engineering reinforced concrete structures are also applicable to the NPP structures; plant service life is not likely to be limited by degradation of the concrete structures; anchorage elements embedded in concrete should receive special attention; and decommissioning needs to be included in service life predictions. Draft ageing management programmes based on these guidelines are in progress for each of the five NPPs.

I.7. UNITED KINGDOM

Nuclear Power Stations in the United Kingdom (UK) are operated by British Energy plc (Nuclear Electric Ltd and Scottish Nuclear Ltd), Magnox Electric, and British Nuclear Fuels plc. In the year ended 31 March 1996, 27% of UK electricity generation was from nuclear stations that included 8 Magnox reactors, 7 AGRs, and 1 PWR.

The Nuclear Installations Act 1965 (as amended) lays down the legal requirements that must be satisfied by any UK utility as a condition of their being allowed to operate a nuclear reactor. The licensing and inspection of sites falls within the jurisdiction of the UK Health and Safety Executive (HSE). With certain exceptions, no site may be used for the purpose of installing or operating any nuclear installation unless a nuclear site licence has been granted by the HSE. This site licence is administered by the Nuclear Installations Inspectorate (NII), which is part of the HSE's Nuclear Safety Division. The efforts of the NII are directed toward ensuring that appropriate expert and detailed assessment of design and construction is undertaken and that the necessary precautions are taken to ensure that the plant is operated safely. The rules are not prescriptive, however, and the Licensee retains absolute responsibility for nuclear safety. The Inspectorate retains the power to close down a station, or prevent its start-up, if it is not satisfied with regard to any item of plant safety. A regulatory view of nuclear containment on UK licensed sites is given in Ref. [I.9].

A set of 35 Standard Conditions is attached to each nuclear site licence. They specify all aspects that the NII require to be satisfied, by provision of appropriate documentation, before a site licence is granted. Site Licence Conditions relate to all phases of operation, from initial design to final decommissioning. 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."*

The UK operating licence, unlike many other countries, is not issued for a fixed period of time. The basis of the licence is therefore continually reviewed to consider the plant's suitability for continued operation. A system of Long Term Safety Reviews (LTSR) was established for the Magnox reactors when the earliest reactors were reaching ages of 20 years. The main objectives of these reviews are to [Ref. I.10]:

- confirm that the plant is adequately safe for continued operation by examination of the original design standard, operational history, and plant modifications;

- identify and evaluate any factors that may limit safe operation of the plant in the foreseeable future (i.e., plant ageing effects over the projected operating period and any "cliff-edge" features just beyond that period); and

- assess the plant's safety standards and practices against modern standards and introduce any reasonably practicable improvements.

The principle of LTSR has been extended to the AGR stations. It is proposed that the documents be reviewed every ten years. A recent change in terminology means that further reviews, and those carried out for reactors other than Magnox, will be called Periodic Safety Reviews (PSRs).

The principal civil engineering structures in the UK power reactors are the:

- concrete biological shield in the early Magnox stations,
- prestressed concrete pressure vessel in the later Magnox and the AGRs, and
- the prestressed containment vessel in the PWR.

A number of other structures may impact, perhaps indirectly, the overall safety of the plant, however, and this influences their design and assessment requirements. Such structures are often referred to as Safety Related Civil Structures. They would include buildings and structures housing critical plant components, where failure of the structure could lead to consequential damage or active waste contaminants. These buildings and structures include such things as spent fuel cooling ponds and supporting structures (e.g., crane platforms).

The continued fitness for purpose of the containments and other safety-related concrete structures has been demonstrated by an extensive system of monitoring, inspection and maintenance. The routine reports and assessments provided have enabled a pattern of behaviour to be built up over the years. This is used to help underwrite the long term operation (thus supporting the PSR), and gives confidence in a continued ability to perform required safety functions. Reference [I.11] provides details of in-service monitoring of AGR and PWR nuclear safety-related structures.

Current research activities target the UK's ageing stock of nuclear facilities, where it is recognised that continued operation relies on effective plant life management aimed at maintaining the integrity and safety function of safety-related civil structures (see Ref. [I.9] for further detail). This includes work in the following areas:

- the 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 the progress of any ageing mechanism (e.g., nondestructive testing 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 recognised 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.

I.8. UNITED STATES OF AMERICA

There are 109 nuclear power reactors that have been licensed for commercial operation in the USA with 1 reactor still under construction and 5 reactors partially completed, but under a deferred construction schedule [I.12]. The concrete containment vessel is generally the only structure included in a defined in-service inspection programme. This has resulted from the requirement contained in General Design Criteria 53, "Provisions for Containment Testing and Inspecting," to Title 10 of the *Code of Federal Regulations*, Part 50 (10CFR50), that the reactor containment be designed to permit periodic inspection of all important areas and an appropriate surveillance programme [I.13].

Regulations for preservice and subsequent periodic containment leakage-rate testing are provided in Appendix J to *10CFR50.* This regulation contains requirements pertaining to 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 are designed to measure the overall leakage rate of the complete containment system. Type B tests are conducted to detect local 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. Type C tests measure isolation valve leakage rates. Both Type B and C tests are conducted by local pressurisation. The purpose of the leakage-rate tests is to periodically verify that leakage through the containment system and pressure-retaining components does not exceed allowable leakage-rate values specified in the Plant Technical Specifications and to ensure that the integrity of the containment structure is maintained during its service life.

In September 1995, the NRC amended Appendix J to provide a performance-based option for leakage-rate testing as an alternative to the existing prescriptive requirements. Option A (prescriptive approach) requires that tests be conducted at approximately equal time intervals during each 10-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. A general visual inspection of accessible interior and exterior surfaces of the containment structure and components is performed prior to each Type A test. For conventionally-reinforced (non-post-tensioned) concrete containments, the general visual inspection currently is the primary in-service inspection that is required for the concrete structures.

Examinations of the unbonded post-tensioning systems are conducted at regular intervals. These examinations are mandated by the Plant Technical Specifications. Detailed examination procedures and acceptance criteria are provided in Regulatory Guide 1.35 [I.14] and the ASME *Boiler and Pressure Vessel Code* Section XI, Subsection IWL [I.15]. These examinations typically consist of a visual examination of accessible hardware; measurement of tendon end anchorage force; measurement of prestressing wire (or strand) strength and ductility; and analysis of sheathing filler for water content, concentration of corrosive ions (i.e., chlorides, sulfides, and nitrates), and reserve alkalinity. The visual assessment considers cracks in the bearing zone concrete; cracking, corrosion, and deformation of load bearing metallic hardware; the presence of water within the end caps or ducts; and the degree of sheathing filler coverage as well as sheathing filler leakage. The examinations are conducted at one, three, and five years following the initial structural integrity test, and at five-year intervals thereafter.

Although instrumentation systems are used in conjunction with the preservice structural integrity test of NPP containments, use of these systems during plant operating life is generally not done in the USA. Responses to a questionnaire sent to US utilities [I.16] indicate that the majority of the plants perform inspections of the concrete structures only to the extent that the above requirements are met. Plants that perform nondestructive evaluations generally do so to investigate a problem or potential problem area (e.g., corrosion of steel reinforcement).

Most of the instances related to degradation of NPP concrete structures in the USA occurred early in their life and have been corrected. Causes primarily were related either to improper material selection and construction/design deficiencies, or environmental effects. Examples of some of the problems attributed to these deficiencies include low 28-d compressive strengths, voids under the post-tensioning tendon bearing plates resulting from improper concrete placement (Calvert Cliffs); cracking of post-tensioning tendon anchorheads due to stress corrosion or embrittlement (Bellefonte, Byron, and Farley); and containment dome delaminations due to low quality aggregate materials and absence of radial steel reinforcement (Crystal River), or unbalanced prestressing forces (Turkey Point 3) [I.17, I.18]. Other construction-related problems have included occurrence of excessive voids or honeycomb in the concrete, contaminated concrete, cold joints, cadweld (steel reinforcement connector) deficiencies, materials out of specification, higher than code-allowable concrete temperatures, misplaced steel reinforcement, post-tensioning system buttonhead deficiencies, and water-contaminated corrosion inhibitors [I.18, I.19].

Although continuing the service of a NPP past the initial operating licence period is not expected to be limited by the concrete structures, several incidences of age-related degradation have been reported [I.17–I.20]. Examples of some of these problems include corrosion of steel reinforcement in water intake structures (San Onofre), corrosion of post-tensioning tendon wires (Fort St. Vrain), leaching of tendon gallery concrete (Three Mile Island), low prestressing forces (Ginna, Turkey Point 3, Zion, and Summer), and leakage of corrosion inhibitors from tendon sheaths (Palisades, Trojan, and Fort Calhoun). Other related problems include cracking and spalling of containment dome concrete due to freeze-thaw damage, low strengths of tendon wires, contamination of corrosion inhibitors by chlorides, and corrosion of concrete containment liners.

REFERENCES TO APPENDIX I

[I.1] CHANDRA, J., et al., "Canadian Programmes for Understanding and Managing Nuclear Power Plant Aging Degradation," in Proc. of ASME Pressure Vessel and Piping Conference, San Diego, California, 1991, American Society of Mechanical Engineers (ASME), New York (1991) 57–62.

[I.2] CRAGG, C.B.H., Overview of Aging Management Programmes for Concrete Containment Structures at Ontario Hydro, IAEA Co-ordinated Research Programme Meeting in Vienna, Austria, June 30 – July 2, 1993, IAEA, Vienna (1993).

[I.3] BACOT, J., "Maintenance of Major EdF Components,"Proc.of the 13th Int. Conf. on Structural Mechanics in Reactor Technology, Vol. I, Porto Alegre (August 13–15, 1995) 231–235.

[I.4] COMBES, J.-P. et al., "Presentation of Works Related to the French PWR Nuclear Power Plant Lifetime," in Proc. of ASME Pressure Vessel and Piping Conference, San Diego, California, June 23–27, 1991, American Society of Mechanical Engineers, New York (1991) 79–83.

[I.5] BISHNOI, L.R., BASU, P.C., "Durability and Ageing Management of Nuclear Safety Related Concrete Structures," Proc. Int. Conf. on Maintenance and Durability of Concrete Structures (4-6 March 1997)", JNT University, Hydrabad (1997).

[I.6] MINISTRY OF INTERNATIONAL TRADE AND INDUSTRY, Basic Concept for Nuclear Power Station Ageing Management (Japanese Version), Japanese Government, Tokyo (1996).

[I.7] ZWICKY, P., KLUGE, D., "Aging Management of Safety Related Concrete Structures in Switzerland," in Proc. 12th Int. Conf. on Structural Mechanics in Reactor Technology, Stuttgart, August 15–20, 1993, Elsevier Science Publishers, Amsterdam (1993).

[I.8] HSK and BASLER & HOFMANN, Alterung der Baustrukturen von Kernkraftwerken (Aging of Nuclear Power Plant Structures), Technical Report B 1712-2, Zurich, July 1992 (draft, internal report).

[I.9] BRADFORD, P.M., MCNAIR, I.J., "A Regulatory View of Nuclear Containment on UK Licensed Sites," Institution of Nuclear Engineers, Int. Conf. on Nuclear Containment, Cambridge (1996).

[I.10] 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).

[I.11] MCFARLANE, J.P., SMITH, L.M, DAVIES, D.R., MCCLUSKEY, D.T, "In-Service Monitoring of AGR and PWR Nuclear Safety Related Structures," Institution of Nuclear Engineers, Int. Conf. on Nuclear Containment, Cambridge (1996).

[I.12] UNITED STATES NUCLEAR REGULATORY COMMISSION, Nuclear Regulatory Commission Information Digest, NUREG-1350, Vol. 7, Division of Budget and Analysis, Office of the Controller, Washington, DC (1995).

[I.13] NATIONAL ARCHIVES AND RECORDS ADMINISTRATION, Code of Federal Regulations, Title 10–Energy, Office of the Federal Register, Washington, DC (1995).

[I.14] UNITED STATES. NUCLEAR REGULATORY COMMISSION, "Inservice Inspections of Ungrouted Tendons in Prestressed Concrete Containment Structures," Regulatory Guide 1.35, Rev. 3, Office of Standards Development, Washington, DC (1990).

[I.15] 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 edition with Addenda, New York (1995).

[I.16] KRAUSS, P.D., Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants, ORNL/NRC/LTR-93/28, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, TN (1994).

[I.17] ASHAR, H., NAUS, D.J., TAN, C.P., "Prestressed Concrete in US Nuclear Power Plants (Part 1)", in Concrete International, **16** (5), American Concrete Institute, Detroit MI (1994) 30–34.

[I.18] ASHAR, H., TAN, C.P., NAUS, D.J., "Prestressed Concrete in US Nuclear Power Plants (Part 2)", in Concrete International, **16** (6), American Concrete Institute, Detroit, MI (1994) 58–61.

[I.19] 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, Toronto (1990).

[I.20] NAUS, D.J., Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants, NUREG/CR-4652 (ORNL/TM-10059), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, TN (1986).

Appendix II

AGEING EXPERIENCE OF NON-NUCLEAR CONCRETE STRUCTURES

II.1. INTRODUCTION

In considering the likely performance of reinforced or prestressed concrete containments in nuclear power plants (NPPs), 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 mitigative action, this has been seen to progress until, ultimately, failure has occurred.

In making comparisons, it is necessary to recognise 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 equally 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 (on the order of several hundred thousand US dollars per day).

II.2. NON-NUCLEAR INDUSTRY PRACTICES

Design, specification and construction in reinforced and prestressed concrete since the 1950's 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 of the more recent guides also include requirements for monitoring construction quality and through-life performance of concrete structures.

It is useful to consider monitoring practice for civil structures in NPPs 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):

- Water industry
- Offshore industry/marine structures
- Transportation industry
- Process industry.

Table II.1 summarizes their ageing management practice. It may be seen that:

- All four industries named above have routine inspection regimes in place, which triggers appropriate reactive maintenance if a fault is found.
- The frequency of visual inspection is between 4 months and 10 years; the frequency reflects the detail of inspection (more detail, less frequent).
- Prescriptive acceptance criteria for any defects found during inspection are not defined. Assessments of defects are based on engineering judgment.
- Although assessment and inspection frequency historically has been based on a determiistic approach, the trend is towards probabilistic assessments. These

TABLE II.1. SUMMARY OF INSPECTION PRACTICE 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 months (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

assessments are used to (1) define inspection periods for structures where a large data base exists (e.g.,bridges), and (2) assess risk from structures with significant consequence of failure (e.g., dams).

Summarized below is additional information related to ageing management practices of the relevant industries noted above. Also information is provided relative to practices associated with cooling towers utilised by the power industry at nuclear or thermal plants.

II.2.1. Water industry

The condition of reinforced concrete dams, locks, and spillways is routinely inspected for structural safety, stability, and operation adequacy [II.1]. A condition survey of these structures generally includes a comprehensive review of the design of the structures, construction techniques and materials, and operational and maintenance history. Information is obtained from available engineering data on the structure as well as an on-site investigation. Data are analyzed and an evaluation report is written that includes conclusions and/or repair recommendations. The frequency of the condition assessments and the approach followed vary somewhat from country to country.

In the USA, organizations such as the Army Corps of Engineers, Bureau of Reclamation, and Tennessee Valley Authority set standards for their civil works structures. Although there is great diversity of facilities due to factors such as intended purpose and use, amount and type of building material, and geographical location, most of the procedures utilise 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 somewhat subjective in that responses are of the form – yes/no, satisfactory/unsatisfactory, high/medium/low, or excellent/good/fair/no change/bad/critical. Some use a computer data bank to assist in maintenance operations, and photographs are required as part of some of the technical evaluations. Most of the organizations require professional engineering services for performance of the inspection and maintenance and have a time schedule or overall frequency of inspections for their structures.

In Canada, guidelines for evaluation of dams have been developed by the Canadian Dam Safety Association [II.2]. Ontario Hydro Technologies is currently conducting seven-year comprehensive reviews of their dam structures in accordance with the policy of the International Commission on Large Dams (France). The reviews of dam safety includes a site inspection; hazard classification; design review; surveillance, monitoring, and dam performance; maintenance adequacy; operation history; and emergency response procedures. The 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 the 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.

Practice in the United Kingdom, which is typical of Europe, involves two levels of inspection [II.3]. At the lower level, routine visual inspections are performed, typically at a four month cycle, by an engineer employed by the particular company. 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 nondestructive 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.

II.2.2. Offshore industry/marine structures

II.2.2.1. Offshore oil/gas production structures

The oil industry has built some 4000 offshore and coastal platforms in the last 50 years, with the construction of deepwater rough sea platforms becoming routine in the last 15 years [II.4]. The in-service performance of offshore platforms is monitored through a combination of inspection and instrumentation programmes.

Inspection of platform substructures is undertaken on a regular basis to monitor the 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 [II.5] notes that the same standards of inspection are not required in all areas of the structure and three classes of visual and nondestructive testing method requirements are provided for underwater and atmospheric inspections. It is generally recommended that the structures be surveyed annually for damage or deterioration paying particular attention to areas that required previous repairs or modifications, and parts of the structure exposed or subjected to fatigue loadings or alternate wetting and drying [II.6]. In addition, results of the surveys should be reviewed in detail every five years and should cover visual inspection of the general conditions, concrete deterioration or cracking, condition and function of corrosion protection system (if present), condition of exposed metal components, condition of foundation and of scour protection system, and amount of marine growth and presence of debris. Visual signs that indicate 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. [II.7].

The in-service performance of the offshore structures is also monitored by instrumentation systems. The primary objective of these systems is to monitor the response of the structures to loadings (e.g., wave and impact). Results also have application to the design of future structures.

II.2.2.2. Marine structures

Concrete structures in a marine environment are exposed simultaneously to action of a number of physical and chemical deterioration processes. Four environmental zones are usually considered: underwater, tidal, splash, and atmospheric [Ref. II.8]. The tidal and splash zones have many similarities and are often combined. The splash (and tidal) zone is considered to be most susceptible to degradation because it is subjected to repeated wetting and drying by sea water. The atmospheric zone follows in susceptibility ranking. The most significant threat to these structures is corrosion of the steel reinforcement. The durability of these structures is covered through minimum design requirements in terms of concrete quality and cover to steel reinforcement. Concrete quality is addressed in terms of minimum cement contents, and concrete cover is specified based on the exposure classification and concrete strength. Assessments of the condition of the concrete in marine structures is based primarily on visual inspections and follows the general guidance provided above for offshore oil/gas production structures.

As noted above, the main cause of deterioration of marine structures is the ingress of chloride ions from the sea water which in time will cause corrosion of the steel reinforcement. In some countries (e.g., Scandinavia) the tidal variation is quite small with the result that the 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. A low risk of reinforcement corrosion is normally associated with submerged structures due to low oxygen levels. If, however, the concrete is of poor quality (e.g., a high water-cement ratio and low resistivity) a macrocell may be created between above- and below-water reinforcement. The reinforcement above the water will receive oxygen from the atmosphere and fuel the corrosion process at the submerged anodic reinforcement.

Many nuclear power plants 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 constitute another unique type of marine structure and should be given special consideration when assessing corrosion risk. It has been found that the corrosion risk to these structures is high wherever turbulence occurs or flow levels are quite high. This increased risk is the result of a higher rate of oxygen diffusion to the reinforcement. A cooling channel cannot be considered to be completely submerged since the outside surfaces of the structure (i.e., culvert or pipe) may be surrounded by relatively dry ground or may even be exposed to the atmosphere. The reinforcement near the outer and inner concrete surfaces normally will be electrically in contact, with the former acting as a cathode. Corrosion also may occur in these structures where there is an imbalance in the electro-chemical potential. This can occur at locations where reinforcing bars intersect or at construction features that interrupt the uniformity of the concrete cover. Construction joints between structural elements, such as between floor and walls or walls and roof slabs, are areas at particular risk. Some experience in Scandinavia has shown that the risk of steel reinforcement corrosion in homogeneous concrete is high if cover thickness to the reinforcement is less than 30 mm and the chloride levels exceed 0.2% Cl^- (with total chloride ion content expressed as a percentage of concrete weight).

A visual inspection of marine structures can be effectively carried out provided the concrete surfaces are thoroughly cleaned, preferably by high-pressure water. It should be noted that corrosion under water will not manifest itself in the manner normally associated with atmospheric corrosion (i.e., cracking and spalling of cover concrete). In the case of underwater corrosion, the corrosion products tend to be less expansive, partly due to the leaching effects of the flowing water. Corrosion tends to occur as a point attack resulting in severe local pitting of the steel. This may be evidenced by rust staining and hard crusts of corrosion products on the concrete surface. By relating the pattern of corrosion points to the steel reinforcement 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.

II.2.3. Transportation industry

The transportation industry primarily encompasses highway bridges, pavements, and railway and transit structures.

II.2.3.1. Highway bridges

Both reinforced and prestressed concrete have been used extensively in the construction of highway bridges. Service life design for these structures generally involves a combination of (1) design strategy (protection against deterioration), (2) type and composition of materials utilised, (3) workmanship, (4) maintenance strategy, and (5) level of quality assurance [II.9]. Although this basic 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 United Kingdom, problems have occurred within 10 years or less of construction. Factors contributing to premature deterioration of bridges include (1) corrosion of metallic embedments (e.g., steel reinforcement) due to environmental conditions and use of deicing salts, (2) fatigue of structural members, (3) inadequate maintenance, and (4) changing performance requirements for older bridges as a result of increased vehicle sizes and frequency of use. Given the magnitude of this problem, both in terms of number of bridges effected and costs that would be associated with repair or replacement, a systematic approach is required to effectively address the problem. Some activities toward development of such a systematic approach to bridge management and structural assessment have occurred.

Bridge management and structural assessment systems

Systems or guidelines for the 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. Individual countries also have developed systems for their use (e.g., United Kingdom and USA).

Basic recommendations for a bridge management system that includes optimisation 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 have been developed by the OECD [II.10]. Bridge inspections are effected 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 optimisation of the inspection. An automated archive (e.g., electronic data base) is the preferred medium for data storage and retrieval. Optimisation of maintenance requires the 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 should 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 the overall bridge management process and requires the 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 of the 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.

In the United Kingdom, inspection procedures for concrete bridges are primarily contained in Refs [II.11] and [II.12]. General inspections are carried out every two years by a Bridge Inspector. Principal inspections involving a complete examination of all accessible elements are carried out every six years by a Chartered Engineer with the results provided in a detailed report. In addition, as a result of the general or principal inspection, there may be special inspections. Guidance on defect detection (e.g., chloride ion attack, carbonation, alkali-silica reactions, and cracking), testing methods, and repair is provided in Ref. [II.12]. Any defect present is generally repaired. If the bridge is assessed to be in poor condition, the Authority undertakes a detailed assessment that may result in weight restrictions, strengthening, or replacement.

Under a bridge inspection programme that began in March 1968, a biennial inspection of most of the bridges in the USA is conducted. Although each of the states has its own approach to bridge inspection, contained as a part of each is a Structural Integrity & Appraisal (SI&A) sheet that is used to develop the National Bridge Inventory (NBI). The NBI provides the only national centralised 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, superstructure, etc. The 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 to require extensive rehabilitation, while others require only minor repairs throughout their service lives which may be 50 years or more. In general, the 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 predict future performance (service life) [II.13]. In order 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 [II.14]. The most recent and comprehensive development related to bridge management systems is "Pontis" which was developed for the Federal Highway Administration [II.15]. Pontis is a network optimisation system developed to address the improvement and maintenance of bridge networks. Several of the objectives that "Pontis" has been designed to achieve 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 the updating of predictive probabilities as data became available; considering the relative importance of various bridges in terms of safety, risk exposure, and public convenience; and providing the basis for short-term and long-term MR&R and improvement in budget planning and resource allocation. A set of interrelated predictive, optimisation, and economic models forms the basis of "Pontis."

II.2.3.2. Highway pavements

Requirements for ageing management of highway pavements are in large measure the same as for highway bridges (e.g., nomenclature, rating system, photographs, and survey form) [II.16]. Central to the concrete pavement damage classification systems is the visual assessment that relies on photographs as a reference and permanent record medium. 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 [II.17, II.18]. The use of these manuals is for identification of distress modes and their severity level using suggested methods for measurement. Data and historical information are obtained at each site by photographic survey vehicles. Results are stored in a central performance data base. Surface distress data include number of potholes and severity, and length or area of surface cracking, spalling, and edge cracking. A separate study [II.19], also based on photography, is mapping and recording highway features in the USA and Canada for use in determining road conditions, prioritising repairs, making cost predictions, planning traffic flow, and coordinating work crews.

II.2.3.3. Railway and transit structures

The bridge is the most common railway structure fabricated of reinforced concrete. In the United Kingdom, the railway bridges are subjected to a wide variety of environments that have produced many concrete deterioration mechanisms (e.g., steel corrosion, alkali-silica reactions, sulfate attack, and frost attack). Occurrence of steel corrosion due to the presence of chlorides or carbonation has been so prevalent that the British Rail Research has developed a portable on-site test for rapid assessments of chloride levels and carbonation depths [II.20]. Criteria for interpretation of results from the chloride content and half-cell potential tests are essentially the same as used by the general concrete industry.

In the USA, a structural monitoring system has been developed for the Miami Metrorail System that encompasses a 37 kilometer elevated rapid transit system and 10 open-air stations [II.21]. The system was developed to address the long-term durability and reliability of the transit system which has a design life of 60 years. The inspection programme is designed to provide procedures for (1) inspecting structural subelements (routine format); (2) evaluating changing conditions and adding new ones; (3) maintaining and updating a data base system; (4) identifying key items that require frequent monitoring, repair, or maintenance; and (5) using the data collected in analysis routines in the event of derailment or other incidents that cause damage to the structure. Guidelines for data collection were prepared. Structural and cosmetic defects observed by the inspectors are recorded in the field to scale on drawings. Data generated in the field are managed by a data base system that was developed. Data evaluation is performed by the responsible engineer who also defines repair procedures and stipulates which items are to be repaired and establishes priorities. An item requiring repair is monitored for at least one additional inspection cycle to ensure effectiveness of the material and technique utilised. A complete inspection of the metrorail system requires two years.

II.2.4. Process industry

Inspection procedures and acceptance criteria used by the chemical and petrochemical industries generally follow internal company guidelines. One organisation, for which information was provided in Ref. [II.3], inspects their concrete structures every five years. Either an "in-house" engineer or an outside consultant performs the 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.

II.2.5. Cooling towers

In France the concrete shell of the cooling tower is considered as a sensitive component under their "Lifetime Project" (see Section I.3). The cooling towers are potentially 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) [II.22, II.23]. An assessment of the failure margins of the cooling towers is being conducted through analysis of aging factors, evaluation of safety and durability factors, and improvements in inspection. Aging factors were analysed with respect to the outside surface (rate of concrete carbonation and penetration of aggressive ions), internal surface (attack by algae and condensate), and 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. The Centre Experimental de Reserches et d'Etudes Bâtiment et des Travaus Publics (CEBTP) and Electricité de France (EdF) have designed and developed a device called "The Lezard" to inspect cooling towers [II.24]. To allow continuous movement over the surface of a cooling tower the mobile device is positioned on cables. It is equipped with a video camera, magnetic sensors to measure concrete cover, and electrode wheels to make potential measurements and determine concrete carbonation. Estimated corrosion rates and cover thickness measurements are used to predict the lifetime of the structure. Also, in 1992 a digital structure mapping (or CNS) system was developed that enables automated visual inspections of the EdF cooling towers and improved diagnosis capabilities [II.25]. The inspection methods and monitoring programmes have been compiled into a large data base. This data base has become an important tool for judging cooling tower behaviour because of the analysis of the level of degradation of each tower and its progression, and the ability to identify structures that may be more vulnerable to degradation so that maintenance activities can be optimised. This method is undergoing an evolution based on results developed from cooling tower applications and it is being modified for applications to other structures.

In Belgium the objective of the ageing management programme for cooling towers is to obtain an early detection of any abnormal changes so as to be able to proceed with reparations as limited as possible. Inspections of the Belgian cooling towers is conducted in collaboration with EdF and includes topographic examinations, visual inspections, and other inspections. The topographic inspections are conducted to monitor any displacement of the foundation system or deformation of the shell. Rings of reference benchmarks are glued to the top of the foundation system at five levels of the shell. The change in elevation position of each of these benchmarks shows any differential settlement and allows an estimation of its structural effects on the shell. The measurement of the horizontal movement of each benchmark is performed by planimetry and the deformed shape of each level is drawn and analysed. Indeed, due to the shape and height/thickness ratio of the shell, its global stability and dynamic behaviour are very sensitive to the top and bottom ring rigidity. A deformation of these rings from the circular shape can induce an excess of flexibility in the structure. Visual inspections are primarily associated with mapping of cracks and other defects present in the shell outer face are mapped. In order to emphasise the defects the cooling tower is in operation during the mapping and thus the inner surface is not accessible. Numerous photographs covering the entire shell surface are taken from the ground level. Computer-aided interpretation of the photographs permits such things as mapping of cracks and defects and their evolution; development of statistics about crack distributions, lengths, and orientations; and cataloguing of photographs showing the change in the shell with time. Additional inspections include measurements to determine presence of steel reinforcement corrosion and depth of concrete carbonation, and laboratory tests on samples removed from the cooling tower. Results of the inspections (e.g., deformation of the shell and degradation in material properties) are used to perform analyses to demonstrate that the required safety factors are being maintained.

REFERENCES TO APPENDIX II

[II.1] STOWE, L., THORNTON, H.T., Jr., Engineering Condition Survey of Concrete in Service, Technical Report REMR-CS-1, US Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS (1984).

[II.2] Dam Safety Guidelines for Existing Dams, Canadian Dam Safety Association, draft, 1993.

[II.3] WILLIAMS, M.C., Review of International Acceptance Criteria Used when Inspecting Concrete Structures, AEA-ESD-0167, AEA Technology Engineering Services, Risley, Warrington (1995).

[II.4] SMALL, R.O., "Offshore and Marine Structures," The Design Life of Structures (G. Somerville, Ed.), Blackie and Son, London (1992) 94–104.

[II.5] BROWNE, R.D., DOYLE, V.J., PAPWORTH, F., Inspection of Concrete Offshore Structures, J. Pet. Tech. (1981) 2243–2251.

[II.6] ACI COMMITTEE 357, Guide for the Design and Construction of Fixed Offshore Concrete Structures, ACI 357R-84, American Concrete Institute, Detroit, MI (1984).

[II.7] SLELTEN, R., FJELD, S., ROLAND, B., "In-Service Inspection of North Sea Structures," Proc. 9th Annual Offshore Technology Conference, Houston (1977).

[II.8] LIU, P.C., "Damages to Concrete Structures in a Marine Environment", Proc. 4th Meeting of RILEM TC-104-DDC, die Technische Universität, Munich (1990).

[II.9] ROSTAM, S., "The European Approach to Design Life", The Design Life of Structures (G. Somerville, Ed.), Blackie and Son, London (1992) 132–143.

[II.10] Bridge Management, Organisation for Economic Co-operation and Development, Paris (1992).

[II.11] DEPARTMENT OF TRANSPORT, The Inspection and Repair of Concrete Highway Structures, BA35/90, HMSO, London (1990).

[II.12] DEPARTMENT OF TRANSPORT, Trunk Road and Motorway Structures — Records and Inspection, TRMM 2/88, Trunk Road Maintenance and Management Notice (1988).

[II.13] SABNIS, G.M., SOROKO, R., DOSHI, K., "Numerical System for Rating of Load Carrying Members", Concrete International **12** (12), American Concrete Institute, Detroit, MI (1990) 63–65.

[II.14] SHIROLE, A.M., WINKLER, W.J., HILL, J.J., Bridge Management Systems — State of the Art, Transportation Research Records 1290, Transportation Research Board, National Research Council, Washington, DC (1991).

[II.15] GOLABI, K., THOMPSON, P., HYMAN, W.A., Pontis Technical Manual, Optima, Inc./Cambridge Systems, Inc., prepared for Federal Highway Administration, Washington, DC (1992) (pre-release copy).

[II.16] LAUER, K.R., "Comments on Concrete Damage Classification System", Proc. 6th Meeting of RILEM Technical Committee 104-DCC, Haifa (1992) 69–77.

[II.17] NATIONAL RESEARCH COUNCIL, Distress Identification Manual for Long-Term Pavement Performance Studies, SHRP-LTPP/FR-90-001, Washington, DC (1990).

[II.18] NATIONAL RESEARCH COUNCIL, Handbook for the Identification of Alkali-Silica Reactivity in Highway Structures, SHRP-C/FR-91-101, Washington, DC (1990).

[II.19] AMERICAN SOCIETY OF CIVIL ENGINEERS, Instrumentation System Maps Nation's Highways, Civil Engineering **61** (8), New York (August 1991).

[II.20] DIXON, J.F., "Railway Structures", Chapter 9 in Durability of Concrete Structures, (G. Mays, Ed.), E.&F.N. Spon, London (1992).

[II.21] SMITH, B.J., "An Application of Structural Monitoring",. Proc. Structural Congress of ASCE, Orlando, Florida, 1987, American Society of Civil Engineers, New York (1987) 204–304.

[II.22] COMBES, J.-P. et al., "Presentation of Works Related to the French PWR Nuclear Power Plant Lifetime", Proc. ASME Pressure Vessel and Piping Conference, San Diego, 1991, American Society of Mechanical Engineers, New York (1991) 79–83.

[II.23] BOLVIN, M., CHAUVEL, D., "Lifetime Cooling Towers", Proc. ASME Pressure Vessel and Piping Conference, New Orleans, Louisiana, 1992, American Society of Mechanical Engineers, New York (1992) 103–110.

[II.24] VIÉ, D., "LEZARD – A Control Device for Cooling Towers", Natural Draught Cooling Towers, (Wittek & Krätzig, Eds), Balkema, Rotterdam (1996) 403–408.

[II.25] GUILLOTEAU, T., SAGEAU, J.K., LANCON, H., "Automated Visual System of Structures (CNS): Application to Electricité de France Cooling Towers", Natural Draught Cooling Towers, (Wittek & Krätzig, Eds), Balkema, Rotterdam (1996) 397–401.

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