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Expanded Materials Degradation Assessment (EMDA)

Volume 4: Aging of Concrete and Civil Structures

Department of Energy · United States of America

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NUREG/CR-7153, Volume 1 – Expanded Materials Degradation Assessment (EMDA): Executive Summary of EMDA Process and Results

NUREG/CR-7153, Volume 4 – Expanded Materials Degradation Assessment (EMDA): Aging of Concrete and Civil Structures

Page 26 in Volume 1 and Page 86 in Volume 4 incorrectly state that there is operational experience indicating the occurrence of alkali-silica reactions in the containment at the Davis Besse Nuclear Power Station. The correct plant should be Seabrook Nuclear Power Plant.

The corrected sentences should read as follows:

Though this degradation is well documented by the operating experience (for bridges and dams in particular) and scientific literature, its high ranking in the EMDA analysis describes the need to assess its potential consequences on the structural integrity of the containment, considering the recent operating experience at Seabrook and other plants.

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Expanded Materials Degradation Assessment (EMDA)

Volume 4: Aging of Concrete and Civil Structures

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ABSTRACT

In NUREG/CR-6923, “Expert Panel Report on Proactive Materials Degradation Assessment,” referred to as the PMDA report, NRC conducted a comprehensive evaluation of potential aging-related degradation modes for core internal components, as well as primary, secondary, and some tertiary piping systems, considering operation up to 40 years. This document has been a very valuable resource, supporting NRC staff evaluations of licensees’ aging management programs and allowing for prioritization of research needs.

This report describes an expanded materials degradation assessment (EMDA), which significantly broadens the scope of the PMDA report. The analytical timeframe is expanded to 80 years to encompass a potential second 20-year license-renewal operating-period, beyond the initial 40-year licensing term and a first 20-year license renewal. Further, a broader range of structures, systems, and components (SSCs) was evaluated, including core internals, piping systems, the reactor pressure vessel (RPV), electrical cables, and concrete and civil structures. The EMDA uses the approach of the phenomena identification and ranking table (PIRT), wherein an expert panel is convened to rank potential degradation scenarios according to their judgment of susceptibility and current state of knowledge. The PIRT approach used in the PMDA and EMDA has provided the following benefits:

- Captured the status of current knowledge base and updated PMDA information,
- Identified gaps in knowledge for a SSC or material that need future research,
- Identified potential new forms of degradation, and
- Identified and prioritized research needs.

As part of the EMDA activity, four separate expert panels were assembled to assess four main component groups, each of which is the subject of a volume of this report.

- Core internals and piping systems (i.e., materials examined in the PMDA report) – Volume 2
- Reactor pressure vessel steels (RPV) – Volume 3
- Concrete civil structures – Volume 4
- Electrical power and instrumentation and control (I&C) cabling and insulation – Volume 5

This volume provides background information on nuclear power plant safety-related concrete structures, their materials of construction, and durability mechanisms and processes that could potentially effect the functional and performance requirements of these structures. This volume also summarizes the results of an expert-panel assessment of the aging and degradation of concrete materials and structures in nuclear power plants. The main objective of the work described herein was to evaluate concrete structures and components in nuclear power plants in which, based on specific operating environments, degradation is likely to occur, or may have occurred; to define relevant aging and degradation modes and mechanisms; and to perform systematic assessment of the effects of these aging-related degradation mechanisms on the future life of those materials and structures. This was accomplished by drawing on the knowledge and expertise of the above-cited expert panel. The approach utilized by the expert

panel was based on the Phenomena Identification and Ranking Table (PIRT) process utilized in the original PMDA report in order to identify safety-relevant phenomena and assess their importance as well as identify and prioritize research needs. The objectives of this report are to determine the degradation mechanisms known for concrete materials and structures, specifically listing the current knowledge on aging degradation of the concrete materials and structures and the confidence level of this knowledge. For areas where there is a lack of knowledge, this report will evaluate the technical gaps in knowledge to identify potential research areas, and prioritize them using the PIRT process.

The PIRT identified a number of mechanisms and degradation modes that may affect the safety function of the concrete and civil structures and components. The highest-ranked mechanism is associated with the corrosion of the reinforcement of cooling towers. This high ranking is the result of significant evidence that the phenomenon will affect many towers beyond 60 years of operation. Though corrosion of reinforced and pre-tensioned concrete elements is well understood and documented, there remain significant knowledge gaps related to the evaluation of the actual state of degradation (inspection) and the evaluation of the structural integrity.

Three of the five high-ranked degradation modes potentially affect the concrete containment, which is the safety-related structure of primary interest.

- The first identified mode is the creep of the post-tensioned concrete containment. Creep is a long-term process associated with sustained loading and moisture transport that affects the internal stress state and, because it adds to tendon relaxation in causing gradual loss of prestress, which is usually restored by periodic re-tensioning thereby introducing a form of cyclic activation of primary creep, can potentially damage the concrete and lead to tertiary (creep-fracture interaction) under accidental loading.

- Related to the creep mode identified above is the interaction between creep and cracking in post-tensioned containments subjected to repair involving prestress modification during the operational life of the containment. While concrete racking is a well understood behavior characteristic of concrete structures in general, and is accounted for in the usual manner in the structural design of reinforced containments, it plays a unique role, (usually unaccounted for in design), in post-tensioned containments. Depending upon the position of the tendons relative to the surface of the containment wall, radially-oriented dilation damage, eventually leading to discrete split cracking, can form on a lamellar surface parallel to the wall surface, which evolves with time as a creep-cracking interaction mechanism. This mode of cracking can potentially occur during initial pre-stressing, during re-tensioning to repair loss of prestress due to concrete creep and tendon relaxation, or during de-tensioning and re-tensioning operations which may be undertaken as part of life extension re-construction work. This type of split cracking can be controlled by radial reinforcement, which generally is not part of the initial design, and because such cracking configuration is internal and is not visible on the surface, it can potentially evolve into an undetectable degradation mode.

- The second mode is the irradiation of concrete. This is due to a lack of sufficient test data to support a clear evaluation of the significance of such mechanism for long-term operations. As a reminder here, the term “concrete containment” is used in a generic sense to describe any concrete part within the containment building. It is obvious here that radiation mainly affects the reactor cavity and the biological shield.

- The third identified mode is the alkali-silica reaction. Though this degradation is well documented by the operating experience (for bridges and dams in particular) and scientific literature, its high ranking in this EMDA analysis describes the need to assess its potential consequences on the structural integrity of the containment.

The fifth identified mechanism is related to boric acid attack of concrete in the spent fuel pool. The knowledge gaps are essentially related to the kinetics and the extent of the attack (role of the concrete mix design) and their consequences on the structural integrity.

The steel components within concrete and civil structures and components were also examined. The two degradation modes of highest priority identified in the PIRT processes for the steel component of the containment are

- the corrosion and stress corrosion cracking of the tendons and
- the corrosion of the inaccessible side of the liner. The lack of knowledge here is associated with the absence of a current in-service inspection technique.

These degradation modes and mechanisms have been identified as having the greatest potential effect on the ability of the concrete and civil structures and components to fulfill their safety related functions during long-term NPP operation. This potential effect may be mitigated by (1) improving the overall level of knowledge about the identified degradation modes in order to better predict and mitigate possible consequences and/or by (2) identifying and implementing acceptable mitigation strategies (replacement, treatments, etc.). Research will be required in either case, and these topics should be the highest priorities for research for concrete and civil structures and components.

The work was conducted via a partnership between the U.S. Nuclear Regulatory Commission's (NRC's) Office of Nuclear Regulatory Research (RES), and the U.S. Department of Energy's (DOE's) Light Water Reactor Sustainability (LWRS) program to extend the NRC's original PMDA report in both time span and scope.

FOREWORD

According to the provisions of Title 10 of the *Code of Federal Regulations* (CFR), Part 54, "Requirements for Renewal of Operating Licenses for Nuclear Power Plants," licensees may apply for twenty-year renewals of their operating license following the initial forty-year operating period. The majority of plants in the United States have received the first license renewal to operate from forty to sixty years and a number of plants have already entered the period of extended operation. Therefore, licensees are now assessing the economic and technical viability of a second license renewal to operate safely from sixty to eighty years. The requirements of 10 CFR, Part 54 include the identification of passive, long-lived structures, systems, and components which may be subject to aging-related degradation, and the development of aging management programs (AMPs) to ensure that their safety function is maintained consistent with the licensing basis during the extended operating period. NRC guidance on the scope of AMPs is found in NUREG-1800 "Standard Review Plan for Review of License Renewal Applications for Nuclear Power Plants" (SRP-LR) and NUREG-1801, "Generic Aging Lessons Learned (GALL) Report."

In anticipation to review applications for reactor operation from sixty to eighty years, the Office of Nuclear Reactor Regulation (NRR) requested the Office of Nuclear Regulatory Research (RES) to conduct research and identify aging-related degradation scenarios that could be important in this timeframe, and to identify issues for which enhanced aging management guidance may be warranted and allowing for prioritization of research needs. As part of this effort, RES agreed to a Memorandum of Understanding with the U.S. Department of Energy (DOE) to jointly develop an Expanded Materials Degradation Assessment (EMDA) at Oak Ridge National Laboratory (ORNL). The EMDA builds upon work previously done by RES in NUREG/CR-6923, "Expert Panel Report on Proactive Materials Degradation Assessment." Potential degradation scenarios for operation up to forty years were identified using an expert panel to develop a phenomena identification and ranking table (PIRT). NUREG/CR-6923 mainly addressed primary system and some secondary system components. The EMDA covers a broader range of components, including piping systems and core internals, reactor pressure vessel, electrical cables, and concrete structures. To conduct the PIRT and to prepare the EMDA report, an expert panel for each of the four component groups was assembled. The panels included from 6 to 10 members including representatives from NRC, DOE national laboratories, industry, independent consultants, and international organizations. Each panel was responsible for preparing a technical background volume and a PIRT scoring assessment. The technical background chapters in each volume summarizes the current state of knowledge concerning degradation of the component group and highlights technical issues deemed to be the most important for subsequent license renewal.

Detailed background discussions, PIRT findings, assessments, and comprehensive analysis for each of these component groups are presented in the following chapters.

CONTENTS

Page

FIGURES

TABLES

ACKNOWLEDGMENTS

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ABBREVIATED TERMS

%, percent

°C, degrees Celsius

°F, degrees Fahrenheit

γ, gamma

γ', gamma prime

Δ, delta; denotes change

$\Delta\sigma_y$, change in yield strength

σ, sigma; denotes variability

τ, UMD recovery time

ϕ, flux

ϕ**t**, fluence

$\langle T_{dam} \rangle$, total average damage energy per atom

0.5T, ½T compact tension specimen

1TC(T), 1T compact tension specimen

3/4-t, three-quarters of the way through the vessel

3DAP, three-dimensional atom probe

41J, 41 joules (absorbed energy level in which Charpy v-notch specimen reaches the ductile-to-brittle transition temperature)

AAR, alkali-aggregate reaction

ADP, annealing demonstration project

AERE, Atomic Energy Research Establishment (UK)

AFCEN, French Society for Design and Construction and In-Service Inspection Rules for Nuclear Islands

AMP, aging management program

AMR, aging management review

ANO-1, Arkansas Nuclear One Unit 1

APT, atom probe tomography

ASME, American Society of Mechanical Engineers

ASTM, American Society for Testing and Materials

at %, atomic percent

ATI, ATI Consulting

ATR, Advanced Test Reactor

B&W, Babcox and Wilcox

BAC, boric acid corrosion

BR3, Belgian reactor 3

BWR, boiling water reactor

C, carbon

C&LAS, carbon and low alloy steels

CASS, cast austenitic stainless steel

CFR, *Code of Federal Regulations*

Cl^-, chloride ion

cm, centimeter

Cr, chromium

CR, cold rolled

CRD, control rod drive

CRDM, control rod drive mechanism

CREEP, thermal creep

CREV, crevice corrosion

CRIEPI, Central Research Institute of Electric Power Industry (Japan)

CRP, Cu-rich precipitates

Cu, copper

CUF, cumulative fatigue usage factor

CVCS, chemical and volume control system

CVN, Charpy V-notch

CW, cold-worked

DBTT, ductile-to-brittle transition temperature

DEBOND, debonding

DH, dissolved hydrogen

DOE, U.S. Department of Energy

dpa, displacements per atom

E, neutron spectrum flux

EBSD, electron backscatter diffraction

EC, erosion–corrosion

ECCS, emergency core cooling system

ECP, electric chemical potential

E_d, displacement threshold energy

EDF, Electricite de France

EDS, energy-dispersive X-ray spectroscopy

EK, Erickson Kirk

Emb., Embrittlement

EMDA, Extended Materials Degradation Assessment

Env., environmental

EONY, Eason, Odette, Nanstad, and Yamamoto

EPMDA, Extended Proactive Materials Degradation Assessment

EPR, electrochemical potentiokinetic reactivation

EPRI, Electric Power Research Institute

eV, electron volt

FAC, flow-accelerated corrosion

FAT, corrosion fatigue

Fe, iron

f_p, volume fraction

FR, fracture resistance

GALL, generic aging lessons learned

GALV, galvanic corrosion

GC, general corrosion

h, hour

HAZ, heat-affected zone

HC, high cycle

HSSI, Heavy-Section Steel Irradiation

HSST, Heavy Section Steel Technology

HWC, hydrogen water chemistry

HWR, heavy water reactor

I&C, instrumentation and controls

IA, irradiation assisted

IAEA, International Atomic Energy Agency

IASCC, irradiation-assisted stress corrosion cracking

IC, irradiation creep

IG, intergranular

IGC, intergranular corrosion

IGF, intergranular fracture

IGSCC, intergranular stress corrosion cracking

IMP, Implementation

IMT, Issue Management Table

in., inch

INL, Idaho National Laboratory

IPA, integrated plant assessment

IVAR, irradiation variables

JAEA, Japan Atomic Energy Agency

JAERI, Japan Atomic Energy Research Institute

JMTR, Japan Materials Testing Reactor

JNES, Japan Nuclear Safety Organization

JPDR, Japan Power Demonstration Reactor

K, stress intensity

keV, thousand electron volt

K_{Ia}, crack-arrest toughness

K_{Ic}, fracture toughness

K_{Jc}, elastic-plastic fracture toughness at onset of cleavage fracture

LAS, low alloy steel

LBP, late-blooming phase

LC, low cycle

LMC, lattice Monte Carlo

LRO, long-range ordering

LTCP, low-temperature crack propagation

LTO, long-term operation

LWR, light water reactor

LWRS, Light-Water Reactor Sustainability

LWRSP, Light Water Reactor Sustainability Program

MA, mill-anneal

MDM, materials degradation matrix

MeV, million electron volts

MIC, microbially induced corrosion

MF, matrix feature

MIG, metal inert gas (welding)

Mn, manganese

MO, Mader and Odette

Mo, molybdenum

MOU, memorandum of understanding

MOY, Mader, Odette, and Yamamoto

MPa$\sqrt{m}$, stress intensity factor; fracture toughness in units of megapascal square root meter

MPC, Materials Properties Council

n/cm^2, fluence

n/cm^2·s, flux

NE, DOE Office of Nuclear Energy

NEI, Nuclear Energy Institute

Ni, nickel

NMCA, noble metal chemical addition

NOSY, Nanstad, Odette, Stoller, and Yamamoto

NPP, nuclear power plant

NRC, U.S. Nuclear Regulatory Commission

NWC, normal water chemistry

ORNL, Oak Ridge National Laboratory

P, phosphorous

PA, proton annihilation

PIA, postirradiation annealing

PIRT, phenomenon identification and ranking technique

PIT, pitting

PLIM, Nuclear Power Plant Integrity Management

PMDA, Proactive Materials Degradation Assessment

PMMD, proactive management of materials degradation

PNNL, Pacific Northwest National Laboratory

PRA, primary recoil atom

PRE, Prediction of Radiation Embrittlement

PREDB, Power Reactor Engineering Database

PSF, Poolside Facility

PT, penetration test

PTS, pressurized thermal shock

PWHT, post-weld heat treatment

PWR, pressurized water reactor

PWROG, Pressurized Water Reactor Owners Group

PWSCC, primary water stress corrosion cracking

R&D, research and development

RADAMO, SCK-CEN TR model and corresponding TR database

RCS, reactor coolant system

RES, NRC Office of Nuclear Research

RHRS, residual heat removal system

RIS, radiation-induced segregation

RPV, reactor pressure vessel

RSE-M, Rules for In-Service Inspection of Nuclear Power Plant Components (France)

RT, reference temperature

SA, solution anneal

SANS, small-angle neutron scattering

SCC, stress corrosion cracking

SCK-CEN, Studiecentrum voor Kernenergie—Centre d'Etude de l'Énergie Nucléaire (Belgian Nuclear Research Centre)

SE(B), single-edge, notched bend

SEM, scanning electron microscopy

SG, steam generator

SIA, self-interstitial atom

SIS, safety injection system

SM, Stationary Medium Power

SMF, stable matrix feature

SR, stress relaxation

SS, stainless steel

SSC, system, structure, and component

SSRT, slow strain rate test

SW, swelling

T_0, fracture toughness reference temperature

T_{41J}, ductile-to-brittle transition temperature measured at 41 joules of Charpy impact energy

TEM, transmission electron microscopy

TG, transgranular

Th, thermal

T_i, irradiation temperature

TIG, tungsten inert gas (welding)

TiN, titanium nitride

TLAA, time-limited aging analysis

TMS, The Minerals, Metals and Materials Society

TR, test reactor

TT, reference transition temperature; thermal treatment

TTS, transition temperature shift

UCSB, University of California, Santa Barbara

UK, United Kingdom

UMD, unstable matrix defect

UNS, Unified Numbering System

U.S., United States

USE, upper-shelf energy

UT, ultrasonic test

VS, void swelling

VVER, Voda-Vodyanoi Energetichesky Reaktor (Water-Water Energetic Reactor)

WEAR, fretting/wear

Wstg., wastage

wt %, weight percent

Zn, zinc

1. INTRODUCTION

1.1 BACKGROUND

As concrete ages, changes in its properties will occur as a result of continuing microstructural changes (e.g., slow hydration, crystallization 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 the concrete will not be able to meet its functional and performance requirements. Concrete, however, can suffer undesirable change with time because of improper specification, a violation of specifications, or adverse performance of its cement paste matrix or aggregate under physical or chemical attack. Deterioration of the embedded steel reinforcement, as well as its interaction with concrete, can also be detrimental to the service life of concrete structures.

In general, the performance of reinforced concrete structures in nuclear power plants has been very good. Incidents of degradation initially reported generally occurred early in the life of the structures and primarily have been attributed to construction/design deficiencies or improper material selection. Although the vast majority of these structures will continue to meet their functional or performance requirements during the current and any future licensing periods, it is reasonable to assume that there will be isolated examples where, as a result primarily of environmental effects, the structures may not exhibit the desired durability, (e.g., water-intake structures and freezing/thawing damage of containments), without some form of intervention.

The Expanded Materials Degradation Assessment (EMDA) is the product of a joint NRC-DOE effort to objectively rank the safety significance of materials degradation issues, particularly as they relate to subsequent license renewal. This EMDA provides an expansion of the original Proactive Material Degradation Analysis (PMDA), *NUREG/CR-6923,* by including additional systems, structures, components and materials, in addition to extending to operational periods beyond 60 years. As part of the EMDA, an expert panel was assembled to address aging and degradation of piping and core internals, reactor pressure vessel materials; concrete materials and structures; and cable and cable insulation systems in nuclear power plants (NPPs), with special emphasis on safety-related structures.

This volume summarizes the results of an expert-panel assessment of the aging and degradation of concrete materials and structures in nuclear power plants (NPP). The main objective of the work described herein was to evaluate concrete structures and components in NPPs where, based on specific operating environments, degradation is likely to occur, or may have occurred; to define relevant aging and degradation modes and mechanisms; and to perform systematic assessment of the effects of these age-related degradation mechanisms on the future life of those materials and structures.

1.2 APPROACH

The expert elicitation process conducted in this study is based on the Phenomena Identification and Ranking Table (PIRT), which has been used by NRC in many applications in the last decade. The PIRT process provides a systematic means of obtaining information from experts and involves generating lists (tables) of phenomena where "phenomena" can also refer to a particular reactor condition, a physical or engineering approximation, a reactor component or parameter, or anything else that might influence some relevant figure-of-merit. The process usually involves ranking of these phenomena using some scoring criteria in order to help determine what is most important. That ranking as well as the information obtained to explain the ranking allows users to prioritize research needs for a safety issue or to support some other decision-making process. The PIRT methodology brings into focus the phenomena that dominate an issue, while identifying all plausible effects to demonstrate completeness.

Each PIRT application has been unique in some respect and the current project is, again, a unique application. The approach followed by the civil structures and concrete panel consists of the following steps.

1. First a list of relevant structures and components was prepared, and a hierarchical identification of the various degradation modes was developed and logged in for each. Four classes of structures and components were identified, together with related degradation modes and mechanisms. Descriptions of relevant structures, materials of construction, and durability mechanisms and processes are given in Chapters 2, 3, and 4, respectively. Figures 1.1 through 1.4 display NPP safety-related structures of primary importance and their related degradation modes. Synergistic/coupling effects of degradation mechanisms in general were not addressed as this was considered to be beyond the scope of this study. Crosscutting issues associated with NPP containments are identified in Figure 1.5.

2. Next a spreadsheet reflecting these degradation modes and mechanisms was developed. For each of the identified for classes of structures and components (described below), each panel then provided an assessment of the level of knowledge, susceptibility, confidence, and structural significance for each degradation mode and mechanism. This assessment is detailed in the spreadsheet included in Appendix A.

3. From the spreadsheet, the mean, median, and standard deviation were determined for each potential degradation mode/mechanism.

To remain consistent with the approach adopted for the PMDA, the panel utilized in their assessment the PIRT process. The PIRT process was faithfully applied and was expanded to encompass some of the unique characteristics of concrete structures. The panel defined a fourth category matrix, “Structural Significance,” in addition to the original three, for each combination of component and degradation mode that follows. The assessment thus addressed the following.

- The degree of **Susceptibility** to degradation

- **Confidence** of the expert panel in their assessment of susceptibility

- The extent of **Knowledge** needed to mitigate or “manage” the degradation

- The **Structural Significance** of the degradation to the safe operability of the structure

Panelists independently scored the degradation scored the degradation scenarios in three categories that were originally used in the PMDA report: Susceptibility, Confidence, and Knowledge. The Susceptibility score rates the likelihood that degradation will occur, on a scale from 0 (not considered to be an issue) to 3 (demonstrated, compelling evidence for occurrence, or multiple plant observations). The Knowledge score rates the expert’s current belief of how adequately the relevant dependencies have been quantified through laboratory studies and/or operating experience, on a scale from 1 (poor understanding, little and/or low-confidence data) to 3 (extensive, consistent data covering all dependencies relevant to the component). Finally, the Confidence score measures the expert's *personal* confidence in his or her judgment of Susceptibility, on a scale from 1 (low) to 3 (high).

After completion of scoring and identification of "outliers," the panels were reassembled for discussion of the scoring. In most panels, this was done in a face-to-face meeting, but this was not required in all cases. During this discussion, each degradation mode and related scoring was discussed with the "outliers" being of highest priority. In these discussions, the scoring panelist presented rationale for any scores that differed from the average. The objective was not to develop a consensus score or force conformity among the panelists. The primary goal of this discussion was to foster debate and exchange differing points of view. This debate and discussion among panelists was an important part of the process to ensure all points of view were considered, including consideration of any new information on the subject area which was not previously considered, and accounted for in the final scoring. The PIRT results and EMDA matrices are described in Chapter 5.

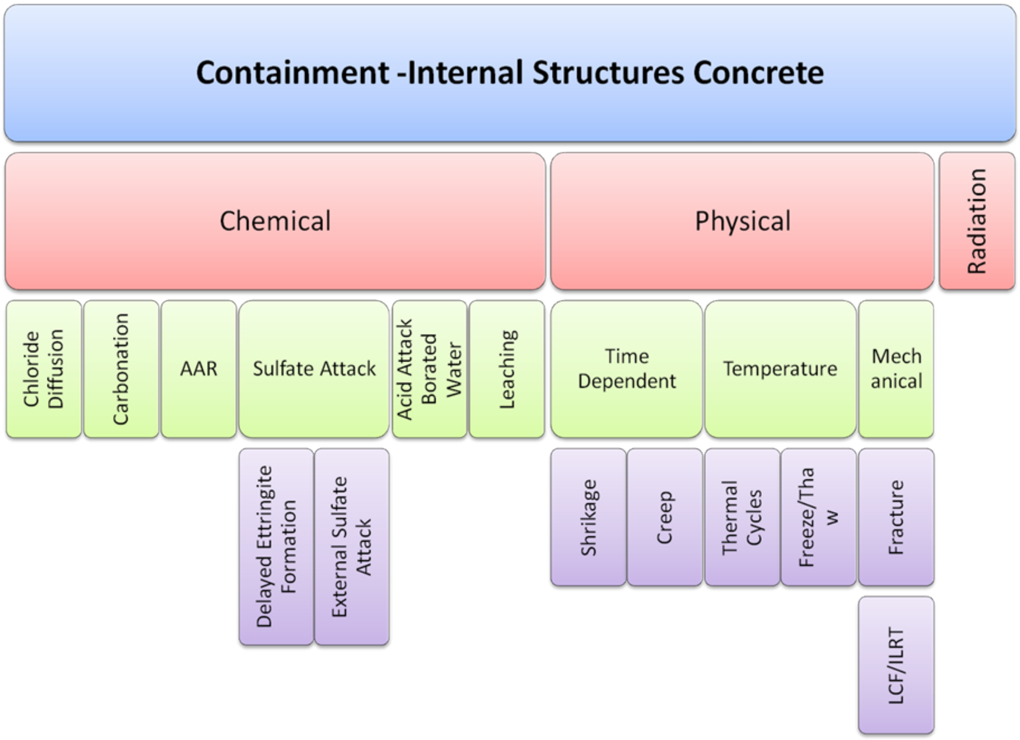

Figure 1.1. Degradation modes in reactor containments: concrete components.

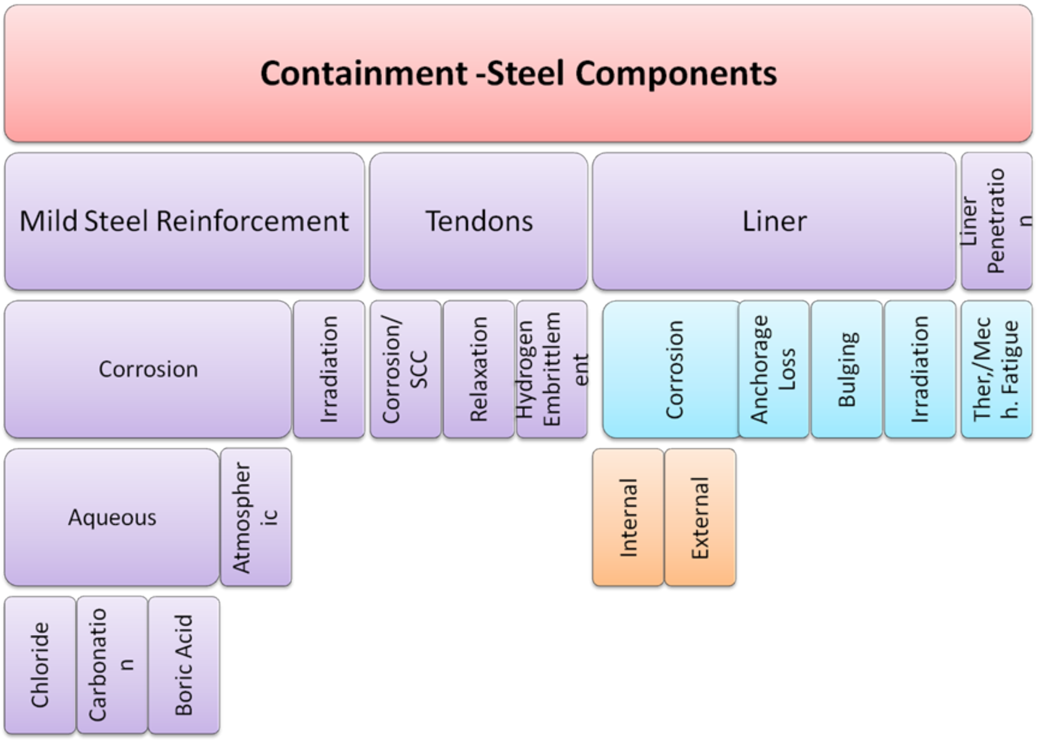

Figure 1.2. Degradation modes in reactor containments: steel components.

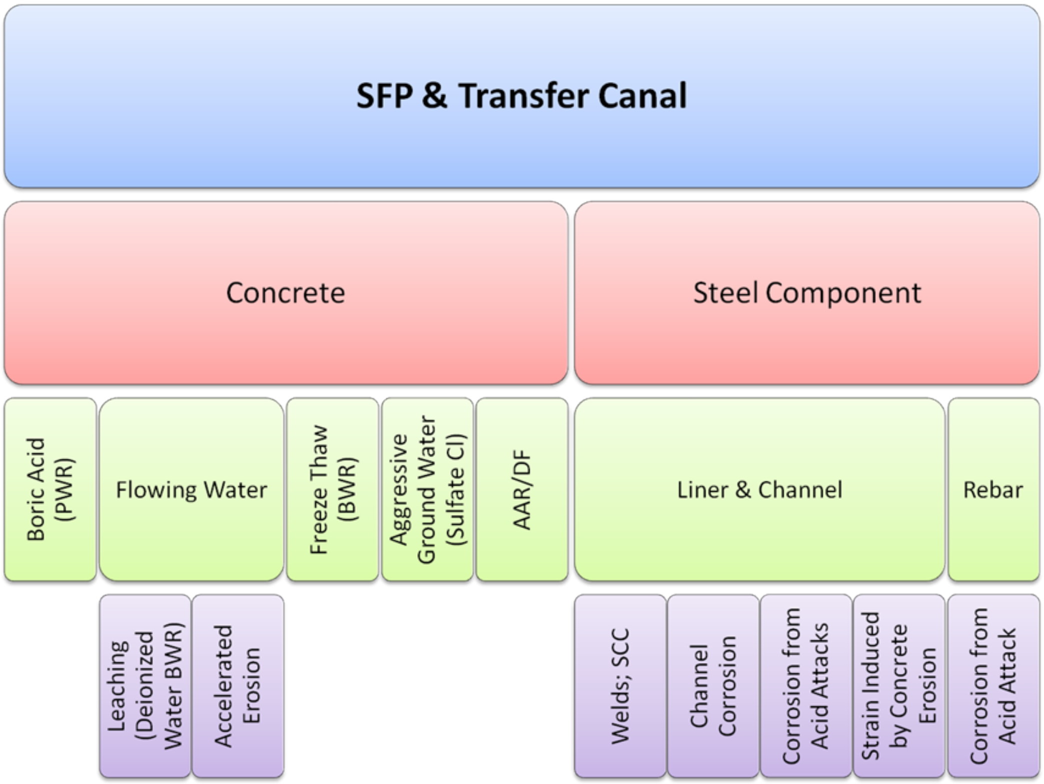

Figure 1.3. Degradation modes in spent fuel pool and transfer canal.

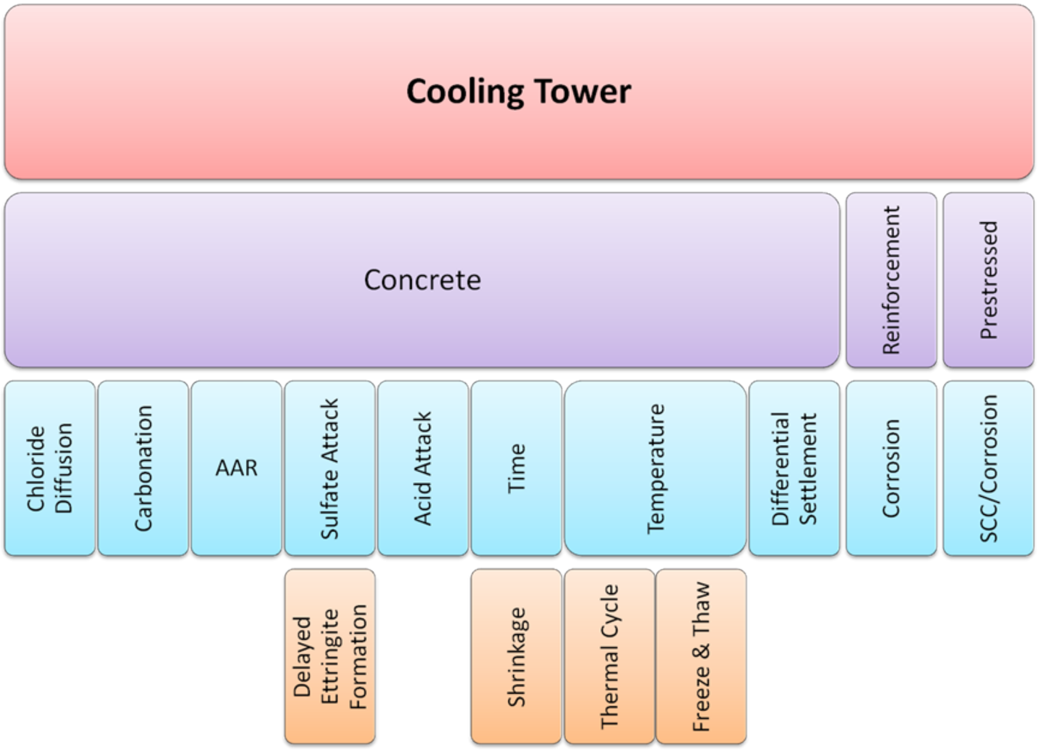

Figure 1.4. Degradation modes in cooling towers.

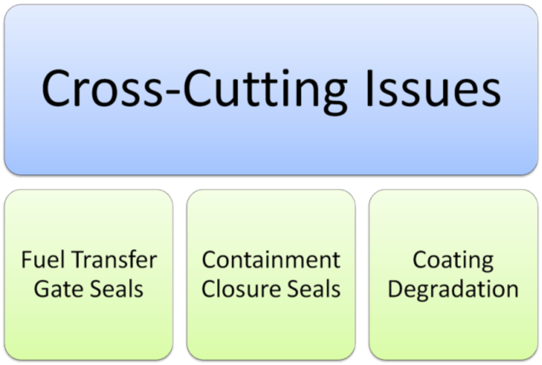

Figure 1.5. Crosscutting issues associated with NPP containments.

2. NUCLEAR POWER PLANT SAFETY-RELATED CONCRETE STRUCTURES

2.1 DESIGN CRITERIA

All commercial U.S. NPPs contain systems, structures, and components (SSCs) whose performance and function are necessary for the protection of the safety of plant operating personnel and the general public as well as for the environment. The basic laws that regulate the design and construction of NPPs are contained in Title 10 of the *Code of Federal Regulations* (10 CFR) [1] that is clarified in documents such as Regulatory Guides (RG) (e.g., RG 1.29, "Seismic Design Classification") [2], NUREG reports, and standard review plans (e.g., *Concrete and Steel Internal Structures of Steel or Concrete Containments*) [3]. For instance, 10 CFR Part 100 and RG 1.29 state that NPP structures important to safety must be designed to withstand the effects of earthquakes without the loss of function or threat to public safety. These "safety-related" structures are designed as Seismic Category I. Seismic Category I structures typically include those classified by the American Society of Mechanical Engineers (ASME) and the American Nuclear Society (ANS) as Classes 1, 2, and 3 (i.e., safety related).

Initially, existing building codes such as the American Concrete Institute (ACI) Standard 318, *Building Code Requirements for Reinforced Concrete* [4], were used in the nuclear industry as the basis for the design and construction of concrete structural members. However, because the existing building codes did not cover the entire spectrum of design requirements and because they were not always considered adequate, the NRC developed its own criteria for the design of Category I structures (e.g., definitions of load combinations for both operating and accident conditions). Current requirements for nuclear safety-related concrete structures, other than concrete reactor vessels and concrete containments, are also based on ACI 318 but have incorporated modifications to accommodate the unique performance requirements of NPPs. These requirements were developed by ACI Committee 349 and were first published in October 1976 [5]. This code has been endorsed by the NRC as providing an adequate basis for complying with the general design criteria for structures other than reactor vessels and containments [6]. Reference [7] provides additional information on the design of seismic Category I structures that are required to remain functional if the Safe Shutdown Earthquake (SSE) occurs (Appendix S to 10 CFR Part 50, Earthquake Engineering Criteria for Nuclear Power Plants). Current requirements for concrete reactor vessels and concrete containments were developed by ACI Committee 359 and were first published in 1977 [8]. Supplemental load combination criteria are presented in Sect. 3.8.1 of the NRC *Regulatory Standard Review Plan* [9]. However, since all but one of the construction permits for existing NPPs have been issued prior to 1978, it is unlikely that endorsed versions of either ACI 349 or ACI 359 were used in the design of many of the concrete structures at these plants. Older plants that used early ACI codes, however, have been reviewed by the NRC through the Systematic Evaluation Program to determine whether there were any safety concerns [10].

Each boiling water reactor (BWR) or pressurized water reactor (PWR) unit in the United States is protected by a large metal or concrete containment that also houses or supports the primary coolant system components. Although the shapes and configurations of the containment can vary significantly from plant to plant, leak-tightness is ensured by a continuous pressure boundary consisting of nonmetallic seals and gaskets and by metallic components that are either welded or bolted together. There are several CFR General Design Criteria (GDC) and ASME Code sections that establish minimum requirements for the design, fabrication, construction, testing, and performance of the LWR containment structures. The GDC serve as fundamental underpinnings for many of the most important safety commitments in licensee design and licensing bases. General Design Criterion 2, “Design Bases for Protection against Natural Phenomena,” requires the containment to remain functional under the effects of postulated natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches. General Design Criterion 16, “Containment Design,” requires the provision of reactor containment and associated systems to establish an essentially leak-tight barrier against the uncontrolled release of radioactivity into the environment and to ensure that the containment design conditions important to safety are not exceeded for as long as required for postulated accident conditions. Criterion 53, “Provisions for Containment Testing and Inspection,” requires that the reactor containment be designed to permit (1) appropriate periodic inspection of all important areas, such as penetrations; (2) an appropriate surveillance program; and (3) periodic testing at containment design pressure of leak-tightness of penetrations that have resilient seals and expansion bellows. Current LWR containments are considered a significant element of the NRC safety policy, which employs a defense-in-depth approach (i.e., successive compensatory measures are exercised to prevent accidents or to mitigate damage if a malfunction, accident, or naturally caused event occurs). The defense-in-depth philosophy ensures that safety will not be wholly dependent on any single element of the design, construction, maintenance, or operation at a nuclear facility (e.g., the facility in question tends to be more tolerant of failures and external challenges).

2.2 TYPICAL PLANT STRUCTURES

From a safety standpoint, the containment is one of the most important components of an NPP because, being independent of the fuel barrier and reactor coolant pressure boundary barrier, it serves as the final barrier to the release of fission products to the outside environment under postulated accident conditions. During normal operating conditions the containment is subject to various operational and environmental stressors (e.g., ambient pressure fluctuations, temperature variations, earthquakes, ingress of hostile ions such as chlorides, and wind storms). In some containment designs, the principal leak-tight barrier is surrounded by another structure (e.g., reactor or shield building) that protects the containment from external events. Ensuring that the structural capacity and leak-tight integrity of the containment has not deteriorated unacceptably from either aging or environmental stressor effects is essential to reliable continued service evaluations and informed aging management decisions. More detailed information on containments is available [11].

In addition to the containment, a myriad of concrete-based structures are included as a part of an LWR plant to provide foundation, support, shielding, and containment functions. Table 2.1 presents a listing of typical safety-related concrete structures that may be included as part of an LWR plant. While there are additional civil structures in an LWR plant, only the safety-related structures are considered here. Relative to general civil engineering reinforced concrete structures, NPP concrete structures tend to be more massive and have increased steel reinforcement densities with more complex detailing. Information pertaining to a particular structure at a plant of interest can be obtained from sources such as the plant’s safety analysis report or docket file. Concrete structures that are considered to be “plant specific” have not been addressed in the discussion below, but some information provided for similar structures may be applicable. The names of certain structures may vary from plant to plant depending on the nuclear steam supply system (NSSS) vendor, architect-engineering firm, and owner preference. Furthermore, cooling towers are not considered safety-related at every plant and this designation depends on the specific design at each site. Typical safety-related concrete structures contained in LWR plants may be grouped into four categories: primary containments, containment internal structures, secondary containment/reactor buildings, and other structures. A listing of typical safety-related concrete structures that fall into these categories at BWR and PWR plants is summarized in Tables 2.2 and 2.3, respectively.

Table 2.1. Typical safety-related concrete structures in LWR plants and their accessibility for visual examination [12]

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

Table 2.2. Typical safety-related concrete structures at BWR plants

Primary containment
Concrete containment
1. Basemat foundation
2. Drywell pedestal
3. Vertical walls (Mark I, Mark II, truncated cone Mark II)
4. Steel liner
5. Suppression chamber (Mark I)
6. Chamber steel liner (Mark I)
7. Concrete dome (Mark III)
8. Polar crane support (Mark III)
Steel containment
1. Basemat foundation
B. Containment internal structures
1. Bottom slab (Steel Mark I and Pre-Mark containments)
2. Reactor pedestal/support structure
3. Biological (reactor) shield wall
4. Floor slabs
5. Walls
6. Columns
7. Diaphragm floor (Mark II)
8. Nuclear steam supply system equipment pedestals/supports
9. Upper and fuel pool slabs (Mark III)
10. Drywell wall (Mark III)
11. Weir/vent wall (Mark III)
12. Crane support structure (Mark III)
C. Secondary containments/reactor buildings
1. Basemat foundation (if isolated from containment building)
2. Walls
3. Slabs
4. Columns
5. Equipment supports/pedestals
6. Sacrificial shield wall (steel containments)
7. Spent/new fuel pool walls/slabs
8. Drywell foundation (Mark I)
D. Other structures (Category I)
1. Foundations[a]
2. Walls[a]
3. Slabs[a]
4. Cable ducts
5. Pipe tunnels
6. Stacks
7. Concrete intake piping
8. Cooling tower basin
9. Dams and intake crib structures
10. Embankments
11. Tanks
12. Water wells

[a] Components of other site buildings such as auxiliary, turbine, control, and diesel generator.

Table 2.3. Typical safety-related concrete structures at PWR plants

A. Primary containment
Concrete Containment
1. Basemat foundation
2. Tendon access gallery
3. Vertical walls (and buttresses)
4. Ring girder (prestressed concrete containment vessel)
Steel Containment
1. Basemat foundation
B. Containment internal structures
1. Bottom floor (steel containment)
2. Floor slabs
3. Walls
4. Columns
5. Nuclear steam supply system equipment pedestals/supports
6. Primary shield wall (reactor cavity)
7. Reactor coolant vault walls
8. Beams
9. Crane support structures
10. Ice condenser divider wall and slab
11. Refueling pool and canal walls
C. Secondary containment building (steel containments)
1. Foundation
2. Walls
3. Slabs
D. Other structures (Category I)
1. Foundations[a]
2. Walls[a]
3. Slabs[a]
4. Cable ducts
5. Pipe tunnels
6. Stacks
7. Concrete intake piping
8. Hyperbolic cooling towers
9. Dams
10. Intake crib structures
11. Embankments
12. Tanks
13. Water wells

[a] Components of other site buildings such as auxiliary, turbine, control, and diesel generator.

2.2.1 Boiling Water Reactors

Of the BWR plants that have been licensed for commercial operation in the United States, approximately 30% utilize a reinforced concrete primary containment. Leak-tightness of each of these containments is provided by a steel liner attached to the containment inside surface by studs (e.g., Nelson studs) or by structural steel members. Exposed surfaces of the carbon steel liner are typically painted to protect against corrosion and to facilitate decontamination should it be required. A portion of the liner toward the bottom of the containment and over the basemat is typically embedded in concrete to protect it from damage (e.g., abrasion, corrosion, and impact). A seal to prevent the ingress of fluids is provided at the interface around the circumference of the containment where the vertical portion of the liner becomes embedded in the concrete.

BWR containments, because of provisions for pressure suppression, typically have “normally dry” sections (drywells) and “flooded” sections (wetwells) that are interconnected via piping or vents (Figures 2.1–2.3).

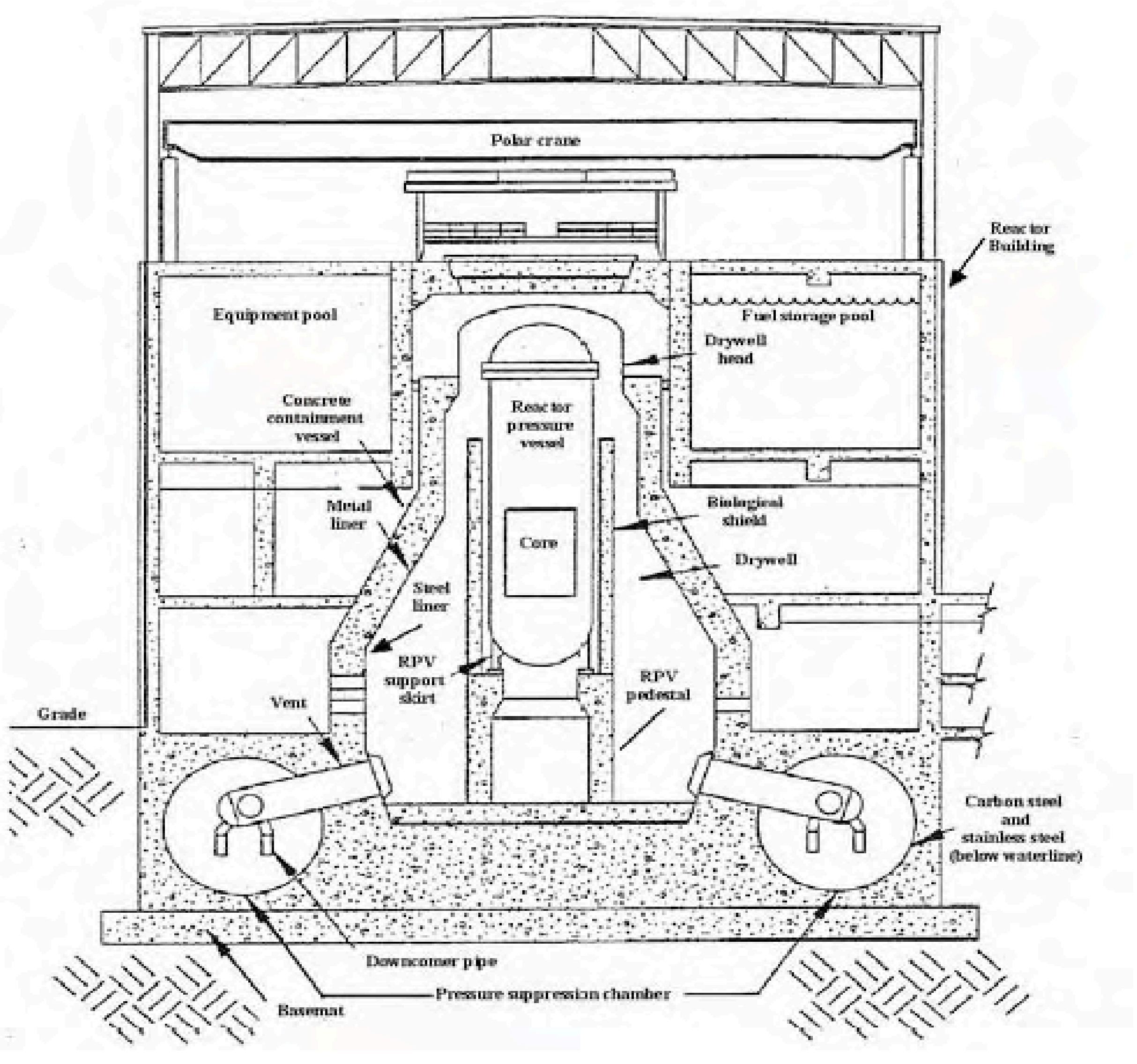

Figure 2.1. BWR Mark I type reinforced concrete containment [13].

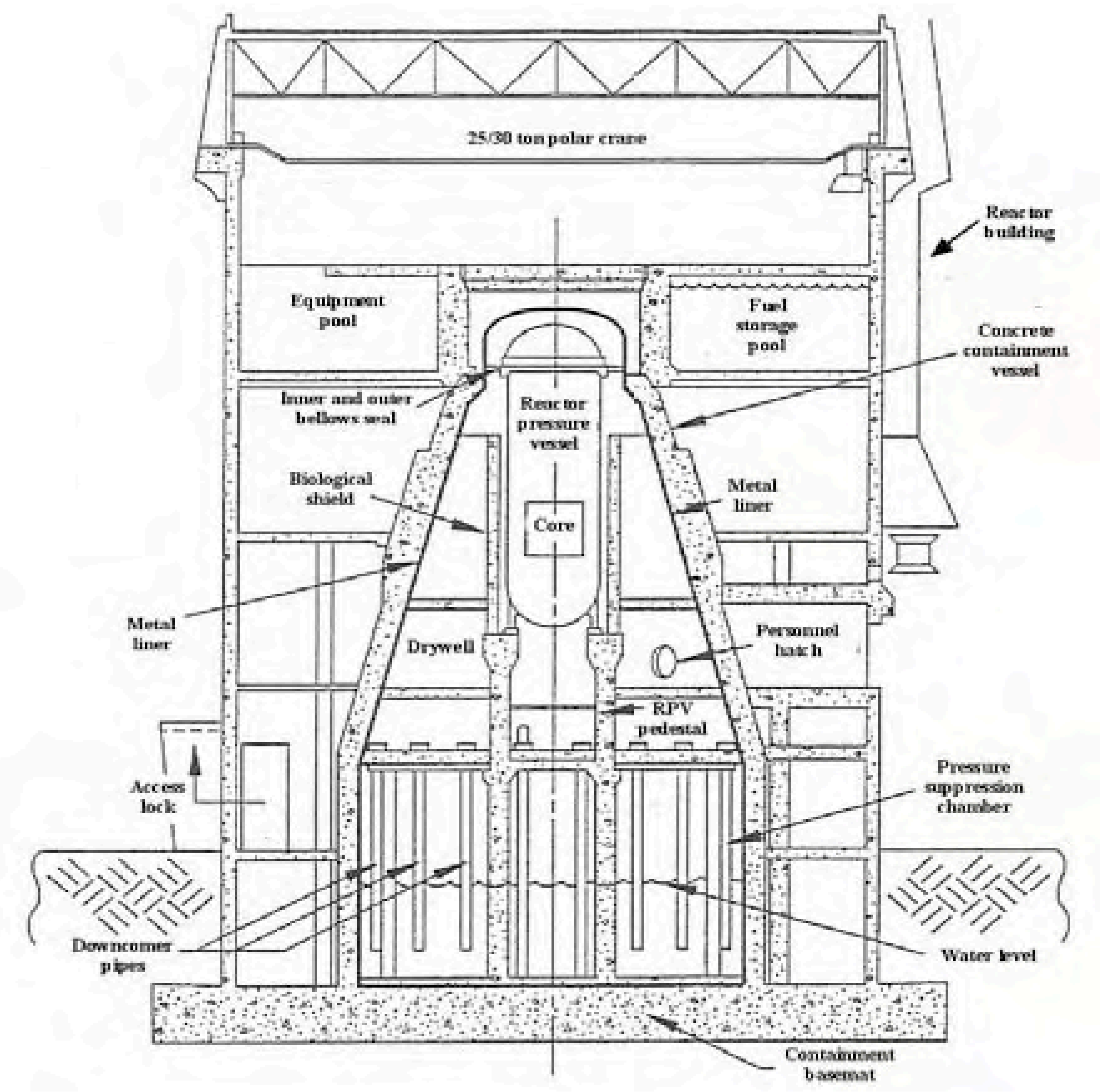

Figure 2.2. BWR Mark II type reinforced concrete containment [13].

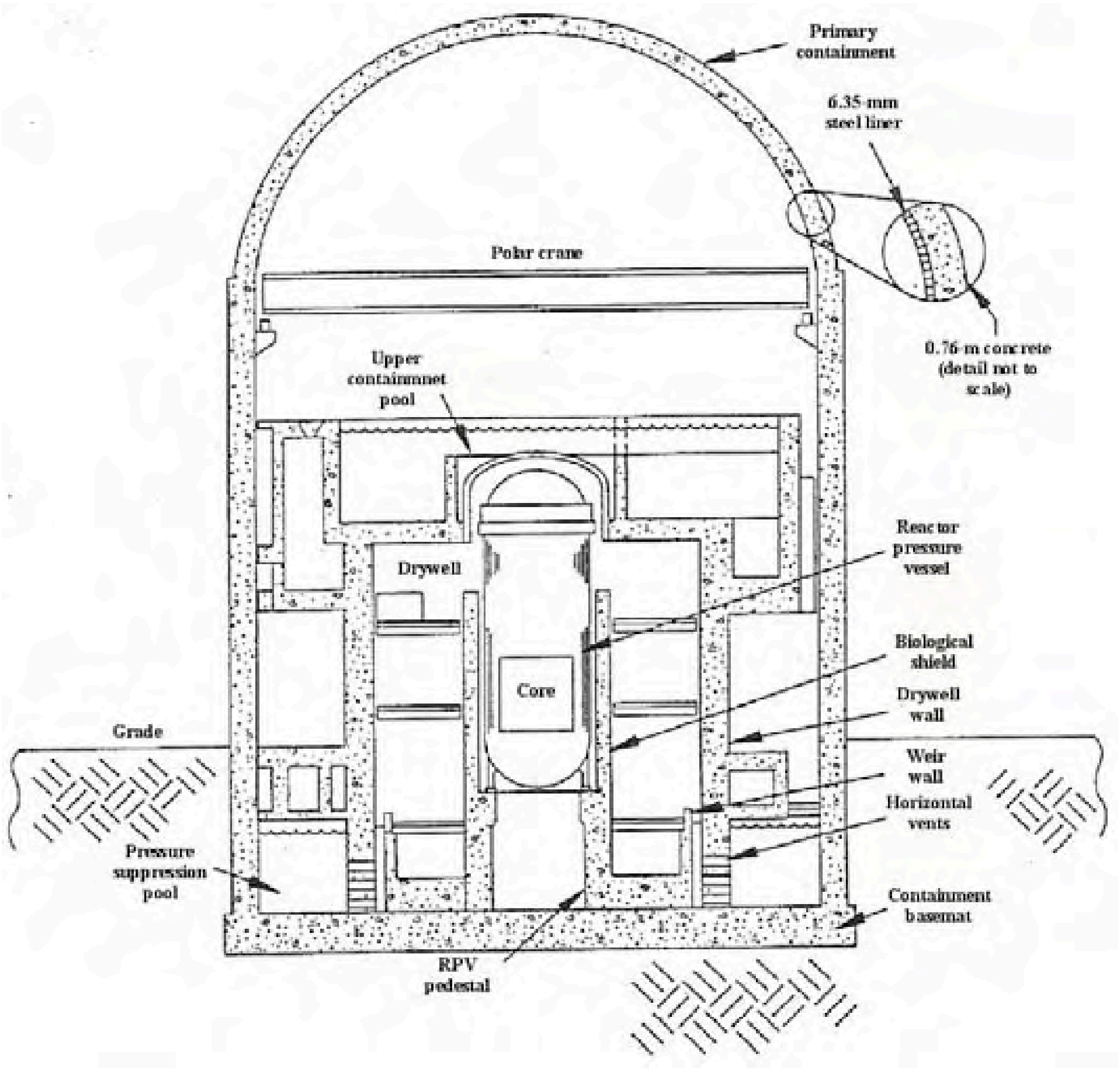

Figure 2.3. BWR Mark III type reinforced concrete containment [13].

Requirements for BWR containments include the following:

1. provide an “essentially” leak-tight barrier against the uncontrolled release of radioactivity to the environment for all postulated design basis accident conditions;

2. accommodate the calculated pressure and temperature conditions resulting from a loss-of-coolant accident;

3. withstand periodic integrated leakage-rate testing at the peak calculated accident pressure that may be at levels up to and including the containment design pressure; and

4. permit appropriate periodic inspection of all important components and surfaces and the periodic testing of the leak-tightness of containment penetrations.

The containment vessel also can provide structural support for the NSSS and other internal equipment. The containment foundation, typically a basemat, provides the primary support and transfer of load to the earth below.

Each of the three BWR primary plant types (Mark I, Mark II, and Mark III) incorporate a number of reinforced concrete containment internal structures. These structures may perform singular or several functions including the following:

- Radiation shielding
- Human accessibility provisions
- NSSS and other equipment anchorage/support/protection
- Resistance to jet, pipe whip, and other loadings produced by emergency conditions
- Boundary of wetwells and pool structures, which allows communication between drywell and wetwell (Mark II and III)
- Lateral stability for containment
- Transfer of containment loads to underlying foundation
- Transfer of fuel to reactor (Mark III)

As many of these functions are interrelated with the required containment functions, these structures are considered safety related.

Of the BWR plants that utilize steel primary containments, all but the pre-Mark plant type have reinforced concrete structures that serve as secondary containments or reactor buildings and provide support and shielding functions for the primary containment. Although the design parameters for the secondary containments of the Mark I and Mark II plants vary somewhat, the secondary containments are typically composed of beam, floor, and wall structural elements. These structures typically are safety related because they provide additional radiation shielding; provide resistance to environmental and operational loadings; and house safety-related mechanical equipment, spent fuel, and the primary metal containment. Although these structures may be massive in cross section in order to meet shielding or load-bearing requirements, they generally have smaller elemental thicknesses than primary containments because of reduced exposure under postulated accident loadings. These structures may be maintained at a slight negative pressure for collection and treatment of any airborne radioactive material that might escape during operating conditions.

Other structures include such components as foundations, walls, slabs, and fuel/equipment storage pools. The spent- and new-fuel storage pools, and the pools for reactor internals storage, typically have a four-wall-with-bottom-slab configuration. The walls and slab are composed of reinforced concrete members lined on the interior surface with stainless steel. Cross sections of these members are generally large because they must support a large pool of water and heavy fuel/component loads produced by high-density fuel storage considerations. The fuel storage pool in Mark III plants is located within the primary containment.

2.2.2 Pressurized Water Reactors

Of the PWR plants that have been licensed for commercial operation in the United States, approximately 80% utilize either reinforced or prestressed concrete primary containments. In meeting the same basic functional and performance requirements as noted for BWR containments, the concrete containments in PWR plants are of three different functional designs (Figures 2.4–2.6): subatmospheric (reinforced concrete), ice condenser (reinforced concrete), and large/dry (reinforced and prestressed concrete). The primary differences between these containment designs relate to volume requirements, provisions for accident loadings/pressures, and containment internal structure layout.

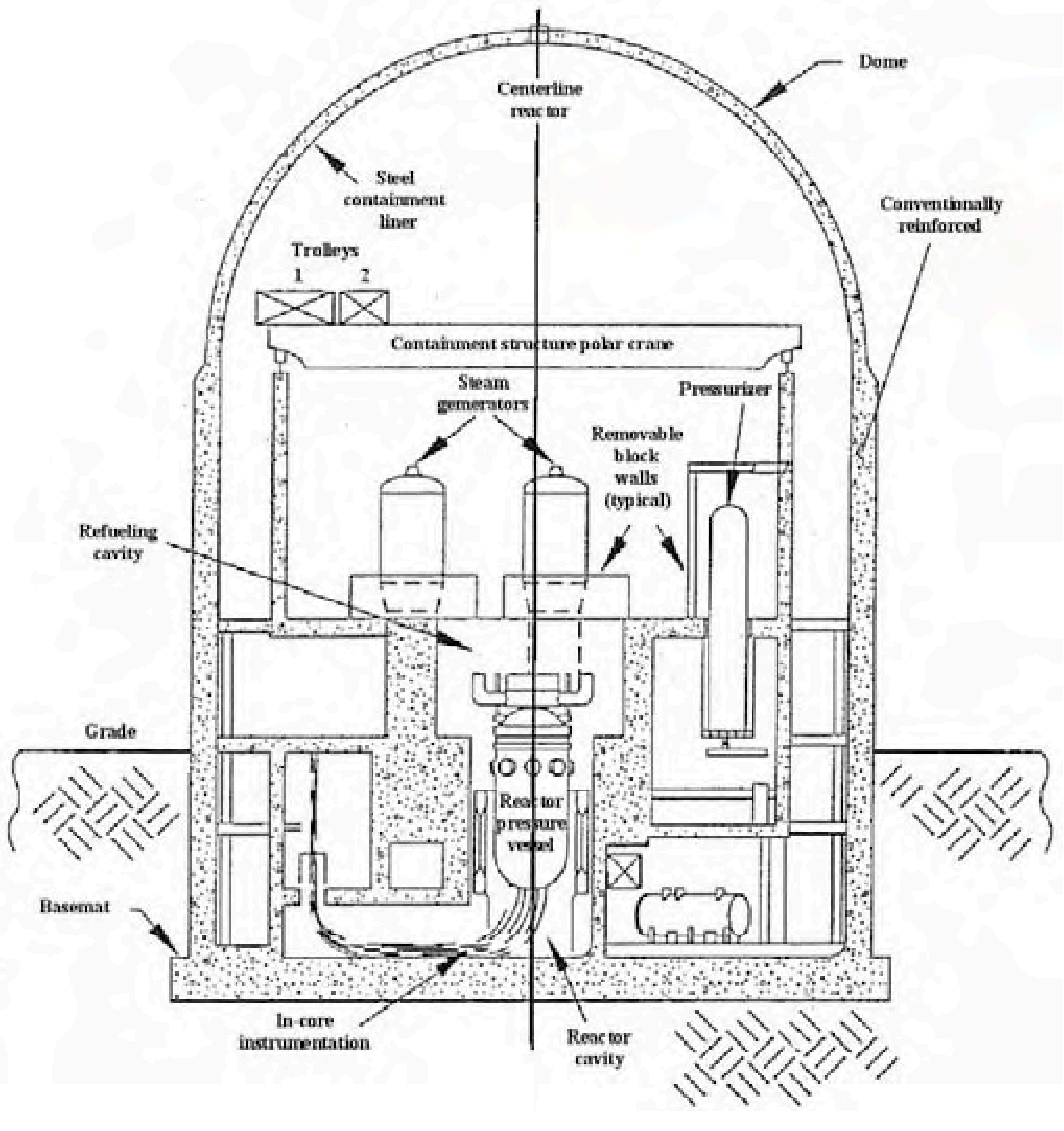

Figure 2.4. PWR subatmospheric type reinforced concrete containment [14].

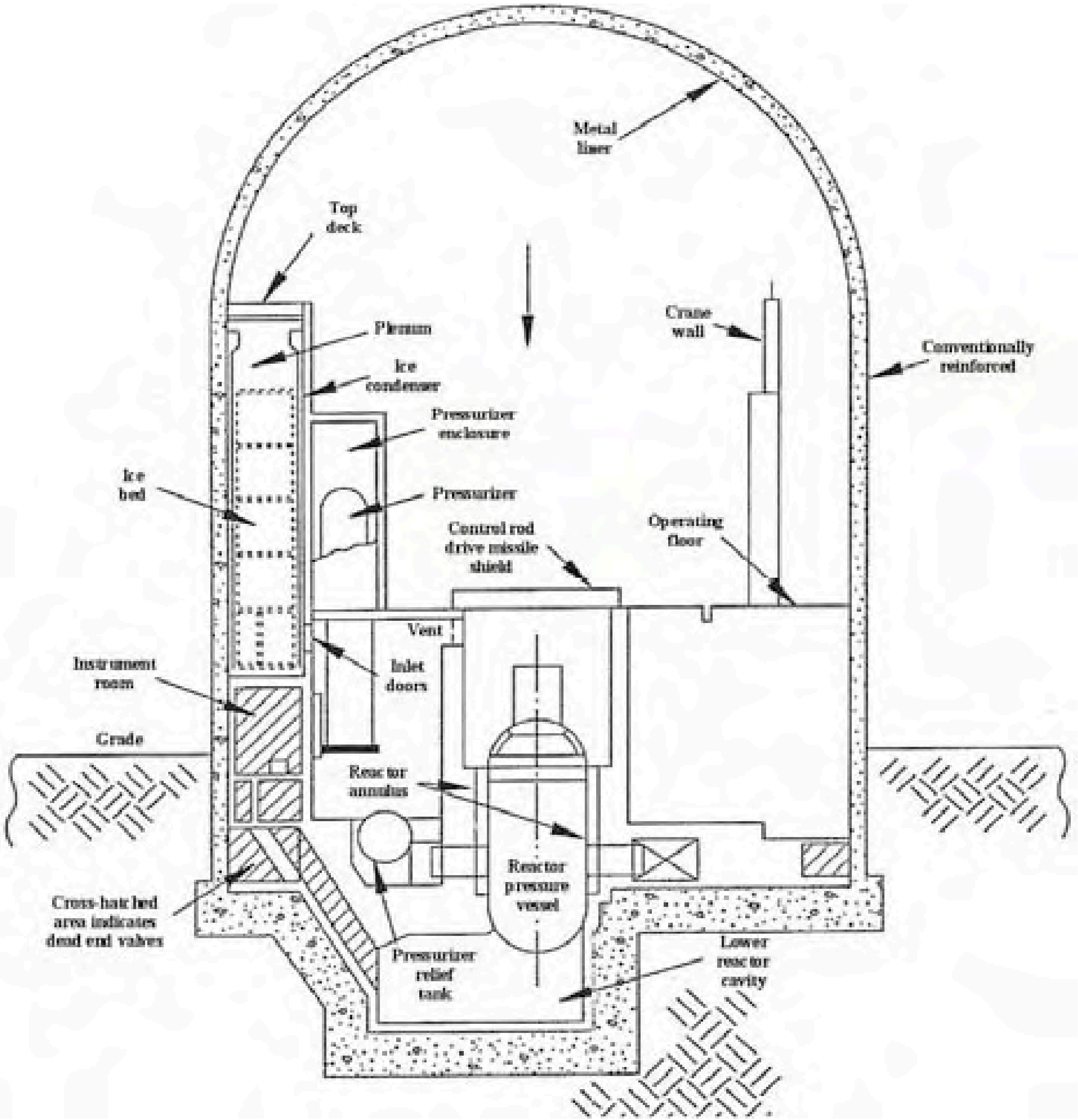

Figure 2.5. PWR reinforced concrete containment with ice condenser [14].

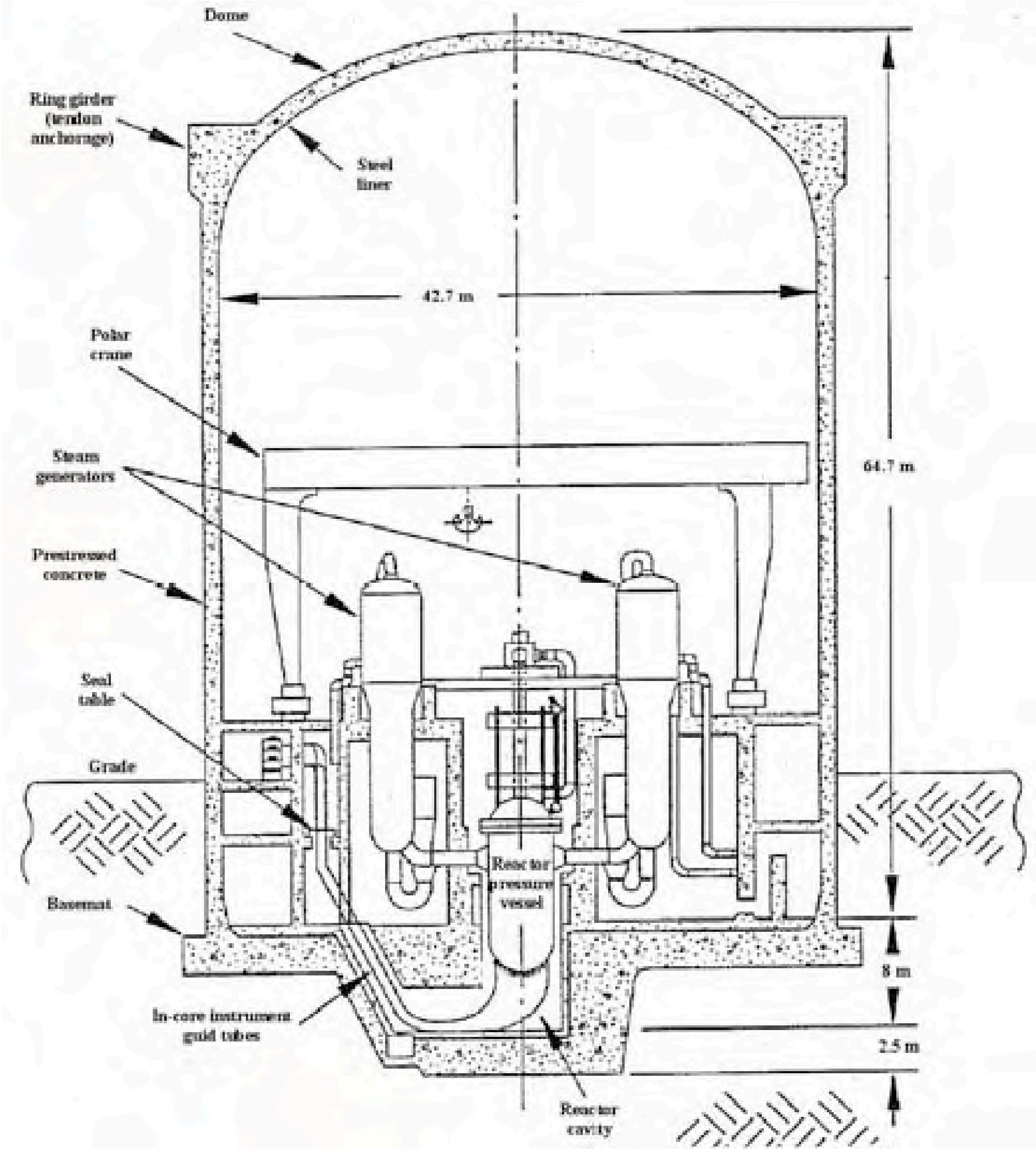

Figure 2.6. PWR large dry prestressed concrete containment [14].

The PWR containment structure generally consists of a concrete basemat foundation, vertical cylindrical walls, and a dome. The basemat may consist of a simple mat foundation on fill, natural cut or bedrock, or may be a pile/pile cap arrangement. Most of the plants have utilized the simple mat on fill or bedrock design. Interior containment surfaces are lined with a thin carbon steel liner to prevent leakage. Exposed surfaces of the carbon steel liner are typically painted to protect against corrosion and to facilitate decontamination should it be required. Depending on the functional design, the concrete containments can be on the order of 40 to 50 m in diameter (131 to 164 ft) and 60 to 70 m high (196 to 230 ft), with wall and dome thicknesses from 0.9 to 1.4 m (3 to 4.6 ft), and base slab thicknesses from 2.7 to 4.1 m (8.9 to 13.5 ft).

The containment internal structures in PWR plants are typically constructed of conventionally reinforced concrete and tend to be more massive than the internal structures in BWR plants because they typically support the reactor pressure vessel, steam generators, and other large equipment and tanks. In addition, these structures provide shielding of radiation emitted by the nuclear steam supply system. Some of the specific functions that these structures (typically floor slabs, walls, and columns) are required to perform include

- provision of human accessibility,
- support and separation of various plant equipment,
- resistance to emergency loading conditions,
- transfer of containment loads to containment foundation,
- missile protection, and
- channeling/routing steam and air through ice condensers (PWR ice condenser containments).

PWR plants that utilize a metallic primary containment (large dry and ice condenser designs) are usually contained in reinforced concrete “enclosure” or “shield” buildings. The secondary containment consists of a vertical cylinder wall with a shallow dome (Figure 2.7) and is often supported by the containment basemat. In addition to withstanding environmental effects, the secondary containment provides radiation shielding and particulate collection and ensures that the freestanding metallic primary containment is protected from the natural environment.

Except for differences in the spent- and new-fuel storage pools, structures that fall into the other structures category are essentially the same at the PWR and BWR plants. The spent- and new-fuel storage pools for PWR plants are typically located in an auxiliary building proximate to the containment. These reinforced concrete wall and slab structures are generally massive in cross section to support a large pool of water and the fuel elements and are lined on the water side with stainless steel. The pools are connected to the reactor/refueling cavity (inside containment) via a transfer channel that is also a safety-related structure since it must provide radiation shielding and support for the fuel transport mechanism and fuel.

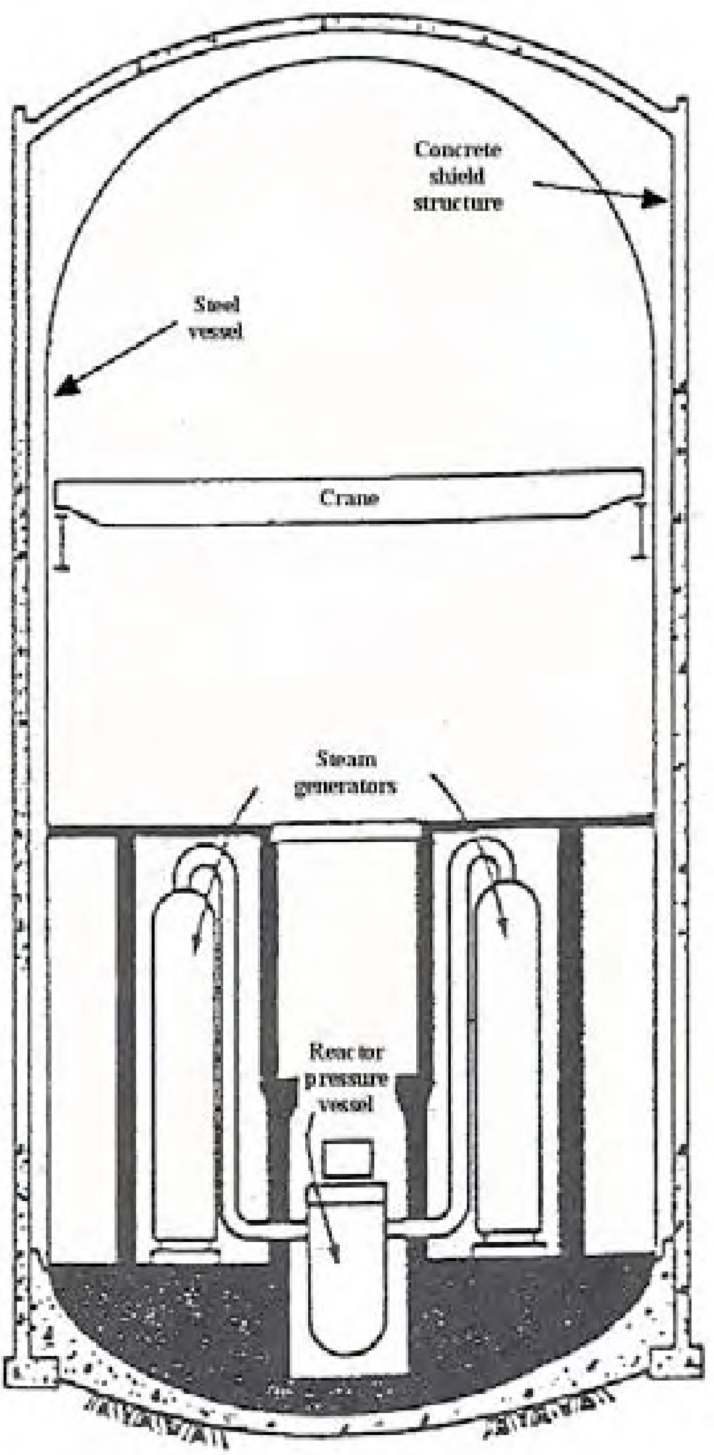

Figure 2.7. PWR freestanding steel containment with elliptical bottom [15].

2.2.3 Auxiliary Structures

Auxiliary structures are considered to be those concrete structures in an NPP that may or may not perform safety-related functions, depending on the plant-specific design and licensing or operating criteria. These structures typically house important plant equipment or control room facilities or provide additional radiation shielding/containment to meet 10 CFR requirements. They may be located immediately adjacent to the secondary containment (e.g., auxiliary building, diesel generator building) or may be separated on site (e.g., intake structures, off-gas stacks). Although these reinforced concrete structures may take many different physical configurations in meeting their functional and performance requirements, they typically fall into two broad categories: (1) common structures and (2) plant-specific structures.

Common building structures are typically configured in a rectangular box shape and consist of reinforced concrete floor slabs, walls, and mat foundation. These subelements are typically of lighter construction (thinner sections with reduced conventional reinforcing) than the plant containment structures. They may also be composite with structural steel framing and may contain shear walls for vertical and horizontal load resistance. Primary functions of these structures are to provide an enclosure for equipment important to plant safety and to provide secondary radiation containment.

Plant-specific concrete structures include components such as intake canal liners, off-gas stacks, and emergency cooling pathways. Although these structures are typically constructed of conventional reinforced concrete, their configuration and methods of construction differ from that of general building construction because the structures must meet specific design loading conditions dictated by their function as well as that of potential extreme environmental conditions (e.g., earthquake, flood, tornado). In addition, these structures may be required to resist the effects of the natural environment, and may be exposed to cooling water (e.g., river, ocean, and lake). Typically, the plant-specific structures contribute to plant safety by serving to dissipate heat and radiation or to protect other safety-related components.

2.2.4 Cooling Towers

Cooling towers are used in closed-cycle water systems to remove waste heat from the main condenser loop. Cooling towers used as the primary heat sink are considered Class I structures and are designed to withstand natural phenomena (e.g., earthquakes and tornadoes).

Concrete cooling towers are divided into two categories: atmospheric or natural draft cooling towers (NDCTs) and mechanical draft cooling towers (MDCTs). Forty-nine cooling towers are reported in operation in the United States: 23 are NDCTs and 26 are MDCTs.

2.2.4.1 Natural Draft Cooling Towers

NDCTs rely on a chimney effect such that air circulation is naturally driven by the change in density between heated air (less dense) and cool air (more dense). Therefore, these structures operate better in areas of high relative humidity. These structures tend to be large and are constructed on site [16]. Figure 2.8 presents an example of an NDCT.

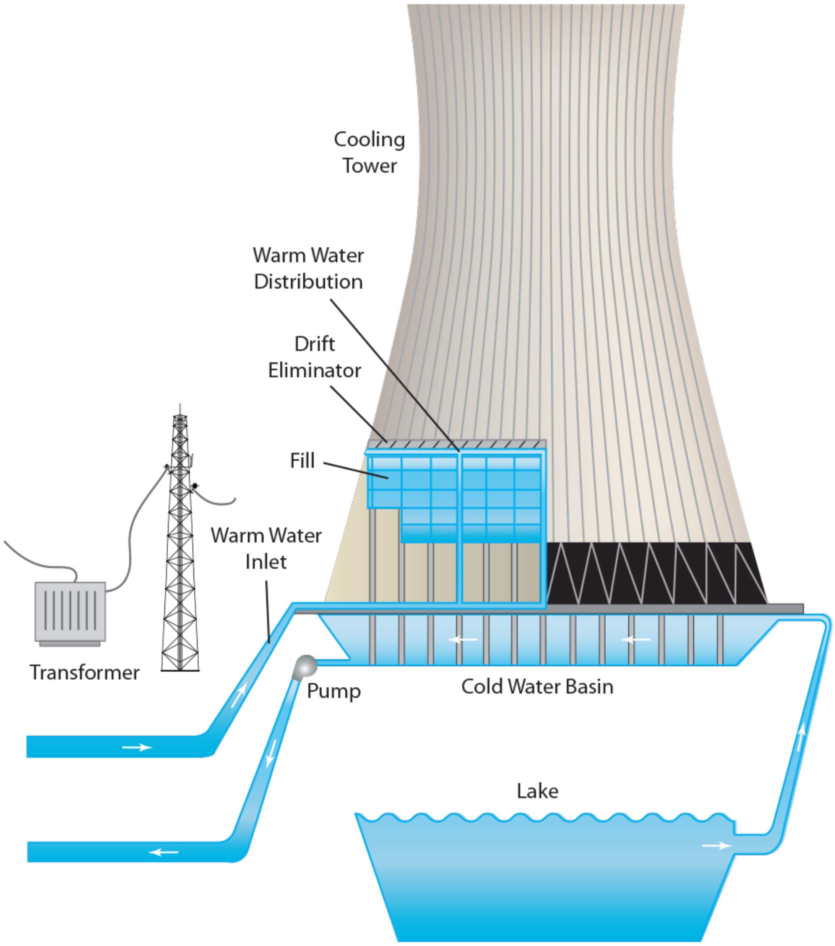

Figure 2.8. Example of natural draft cooling tower [16].

Concrete structures in a generic NDCT include:

- **hyperbolic shell**: reinforced concrete structures (minimal steel ratio as specified by design code), typical height between 120 and 175 m (in France, can go up to 200 m) (393 and 574 up to 656 feet), shell thickness from 25 (neck) to 40 cm (0.82 to 1.3 ft), and crown/cornice diameter about 80 m (262 ft). The crown is generally wider both for mechanical stiffening and for allowing a walking access. The shell lintel is supported by X/W/A-shaped columns and usually a shallow annular foundation.

- **packing support**: reinforced beams supporting dispersion fills.

- **pool**: reinforced structure used to collect the intake water and dispatch it through a network of metallic piping that sprays the water through sprinklers.

- **intake structures including piping**: piping leading to the tower may be a variety of materials from concrete to steel or fiber-reinforced piping (FRP).

- **cold-water basin**: after the water is cooled, it must return to a reservoir for routing back to the condenser. This reservoir is called the cold-water basin. It is usually a concrete reservoir.

2.2.4.2 Mechanical Draft Cooling Tower

Mechanical draft cooling towers are called mechanical because they have fans, usually located in the top of the cooling tower structure, that move air across the hot water. The tower may be either a cross-flow type or a counter-flow type. The cross-flow type allows for the air to travel in a direction essentially perpendicular to the air flow direction. This is accomplished by water being pumped to an open inlet basin. Metering orifices (i.e., holes) in the inlet basin allow water to fall by gravity onto cooling tower fill. The metering orifices may incorporate shapes at the outlet of the orifices to better distribute the water for cooling. Fans are used to induce air flow through the fill. Typically, for power plants, these fans are rather large axial fans. The air enters horizontally, often directed by louvers over and through the wet surface of the fill. Drift eliminators are used to prevent excessive amounts of water droplets from being carried through the fan. The air then turns vertically and is exhausted through the fans [17].

Mechanical draft cooling towers are usually prefabricated and can be either assembled on site or at the factory. Different materials are used for these towers, with concrete being the preferred material for the water basins in all of them. Additionally, field-erected towers (typically large units) are increasingly using concrete for columns, beams, supports, and decks because of its higher resistance to fire and its load-carrying capacity. Different configurations can be seen in these towers depending on the direction of the air flow (cross-flow vs. counter-flow); the location of the fan (forced draft vs. induced draft); and the shape of the tower (rectilinear vs. round) [16]. Figure 2.9 presents an example of an MDCT cross section and related information.

Concrete structures in a generic round-shaped MDCT include the following.

- fan deck: located at about 15.2 m (50 ft) and supported by a network of beams and slender columns. The fan deck may support 16 different fans and mechanical equipment.

- distribution flume and basin: located at the periphery of the fan deck, they distribute the water before dropping through the fills.

- radial and circumferential panels (frames) support the distribution flume and basin. They also support the fills (splash bars) and the concrete fill beams.

- cold water basin: similar to that for an NDCT.

Concrete elements can be reinforced or prestressed precast elements. One plant reports the use of epoxy-coated reinforcement.

Operating Conditions

During operation, concrete inside the MDCT shell is subjected to a temperature around 40 °C (104 °F) and relative humidity of 100%.

The “natural” chemistry of the intake water varies from one plant to the other. It is notable that some plants use municipal retreated water containing chlorides or brackish water.

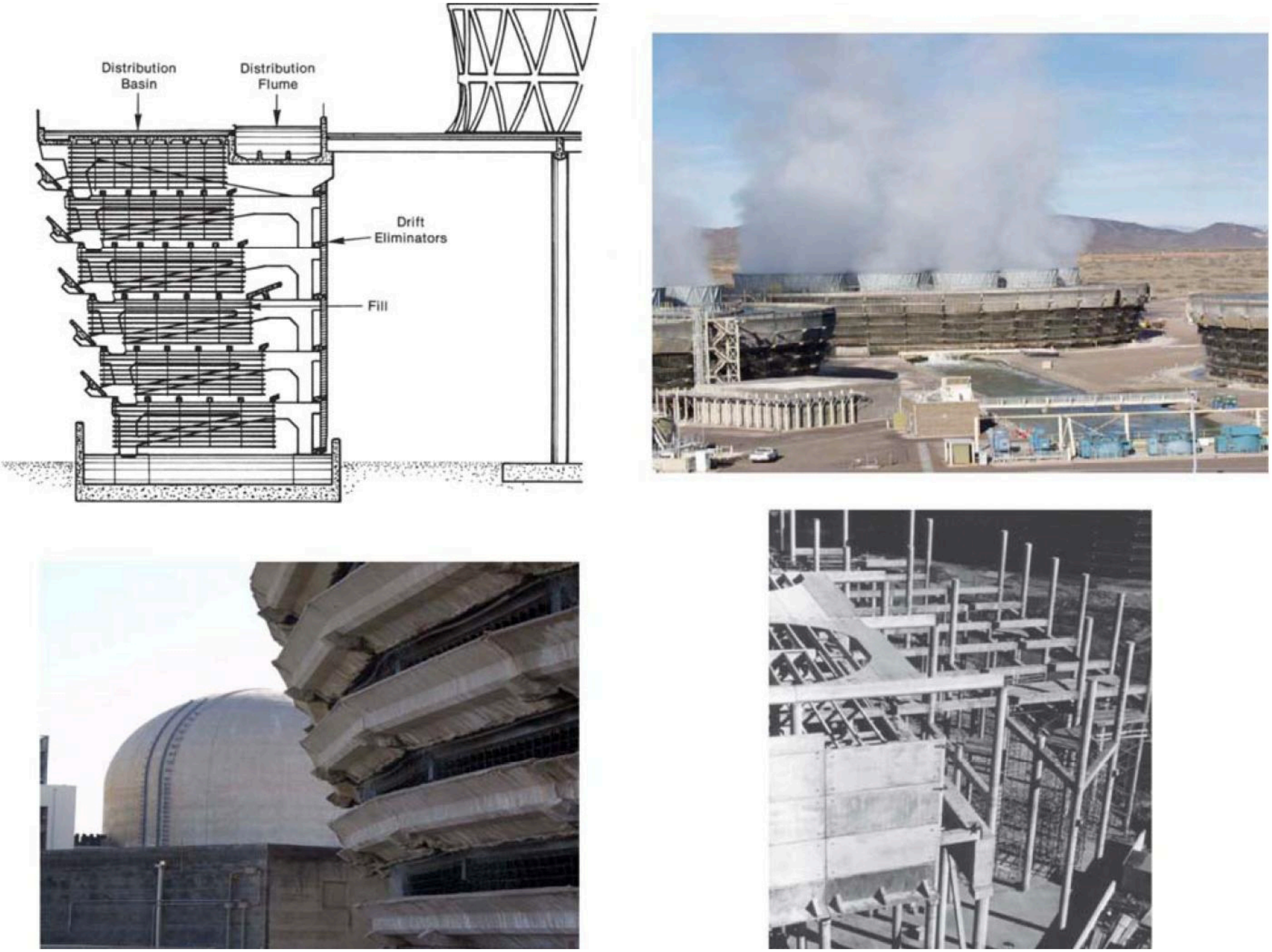

Figure 2.9. Example of mechanical draft cooling tower: cross section (upper left), tower in operation (upper right), louver (bottom left), and modular construction (bottom right). Source: APS and SPX Cooling Technologies.

To clean the condenser, chemicals are often added to the water. In the United States, acid/phosphate/ oxidizing biocides, such as sodium hypochlorite, are used to dissolve the packing deposit or are used as a biocide. Calcium deposits are often observed on packing beams. The deposit threshold triggering a cleaning operation is about 120 kg/m^3 in the United States and 30 kg/m^3 in France (7.5 and 1.8 lb/ft^3, respectively).

3. MATERIALS OF CONSTRUCTION

Nuclear safety-related concrete structures are composed of several constituents that, in concert, perform multiple functions (e.g., load-carrying capacity, radiation shielding, and leak tightness). Primarily, these constituents can include the following material systems: concrete, conventional steel reinforcement, prestressing steel, steel liner plate, and structural steel. The quality of these materials is established through regulations, qualification tests, and certification followed by checking throughout construction.

3.1 CONCRETE

Concrete is a composite material consisting of a binder (cement paste) and a filler of fine or fine and coarse aggregate particles that combine to form a synthetic conglomerate.

Portland cements are primarily composed of four chemical compounds: tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium aluminoferrite (C_4AF). The type of Portland cement produced (e.g., general purpose, moderate sulfate resistance and heat of hydration, high early strength, low heat of hydration, and sulfate resistant) depends on the relative amounts of the four basic chemical compounds and particle fineness (contributes to high early strength). The calcium silicate hydrates (C-S-H) constitute about 75% of the mass. The C-S-H gel structure is made up of three types of groups that contribute to bonds across surfaces or in the interlayer of partly crystallized tobermorite material: calcium ions, siloxanes, and water molecules. Bonding of the water within the layers (gel water) with other groups via hydrogen bonds determines the strength, stiffness, and creep properties of the cement paste.

There are also a number of alternative cementing agents that have been used in conjunction with Portland cement, such as pulverized fly ash, ground granulated blast furnace slag, and silica fume. Fly ash is collected from the exhaust flow of furnaces burning finely ground coal and reacts with calcium hydroxide in the presence of water to form cementing compounds consisting of calcium silicate hydrate. Ground granulated blast-furnace slag (GGBS) is a by-product of the iron-making process and is formed by taking the hot slag, rapidly chilling or quenching it, and grinding into a powder. When mixed with water in the presence of an alkaline environment provided by the Portland cement, GGBS hydrates to form cementing compounds consisting of calcium silicate hydrate. Silica fume is the condensed vapor by-product of the ferro-silicon smelting process that reacts with calcium hydroxide in the presence of water to form cementing compounds consisting of calcium silicate hydrate. High-alumina cement, consisting mainly of calcium aluminates, has been utilized as a cementitious material because of its rapid set and rapid strength gain characteristics and resistance to acidic environments, seawater, and sulfates. Several NPPs have utilized high-alumina cement to produce porous concrete subfoundations. Although some erosion of the cementitious materials has occurred in at least one plant, the amount of material removed has been insignificant and the plants are monitored for signs of settlement [18]. However, because under certain conditions of temperature and humidity the cement converts over time to a different hydrate having increased porosity and reduced strength, it is recommended that calcium aluminate cements not be used for structural applications, particularly in wet or humid conditions above 27 °C (80.6 °F) [19].

Selection of the proper water content of concrete is critical since too much water reduces the concrete strength, and if insufficient water is added, the concrete will be unworkable. Hardening of concrete occurs as a result of hydration, which is a chemical reaction in which the major compounds in the cement form chemical bonds with water molecules and become hydrates.

The hardened cement paste consists mainly of calcium silicate hydrates, calcium hydroxide, and lower proportions of calcium sulphoaluminate hydrate either as ettringite or monosulphate. About 20% of the hardened cement paste volume is calcium hydroxide. The pore solution is normally a saturated solution of calcium hydroxide within which high concentrations of potassium and sodium hydroxides are present. Proper curing of the concrete is essential as it affects the concrete's durability, strength, watertightness, abrasion resistance, volume stability, and resistance to freezing and thawing.

Since cement is the most expensive ingredient in concrete, it is desirable to utilize the minimum amount necessary to produce the desired properties and characteristics. Aggregate typically occupies 60% to 75% of the volume of concrete, and therefore its characteristics strongly influence the chemical, physical, and thermal properties of concrete; its mix proportions; and economy. Aggregates thus are important with respect to the concrete durability. The aggregates come in various shapes, sizes, and material types, ranging from fine sand particles to large coarse rocks. Selection of the aggregate material is determined in part by the desired characteristics of the concrete. The available aggregate materials range from ultra-lightweight (e.g., vermiculite and perlite) to lightweight (e.g., expanded clay shale or slate-crushed brick) to normal weight (e.g., crushed limestone or river gravel) to heavyweight (e.g., steel or iron shot). Sometimes chemical or mineral admixtures are added during the mixing process to enhance durability (air entrainment), improve workability (enhanced placement and compaction), modify hardening and setting characteristics, aid in curing, reduce heat evolution, or provide other property improvements [20].

The concrete typically used in nuclear-safety-related structures consists of Type II Portland cement, fine aggregates (e.g., sand), water, various mineral or chemical admixtures for improving properties or performance of the concrete, and either normal-weight or heavyweight coarse aggregate. Although eight types of Portland cement are recognized by ASTM C 150 [21], Type II Portland cement typically has been used because of its improved sulfate resistance and reduced heat of hydration relative to the general-purpose (Type I) Portland cement. Both the water and fine and coarse aggregates are normally acquired from local sources and are subjected to material characterization testing prior to use. Coarse aggregate can consist of gravel, crushed gravel, or crushed stone. Chemical (e.g., air entraining or water reducing) or mineral (e.g., fly ash or GGBS) admixtures have been utilized in many of the mixtures to impart improved characteristics or performance. For those concrete structures in NPPs that provide primary (biological) radiation shielding, heavyweight or dense aggregate materials, such as barytes, limonites, magnetites, and ilmenites, may have been used to reduce the section thickness and meet attenuation requirements.

The constituents are proportioned and mixed to develop Portland cement concrete that has specific properties. Depending on the characteristics of the specific structure, the concrete mix may be adjusted to provide increased strength, higher durability, or better workability for placement. The hardened concrete typically provides the compressive load-carrying capacity for the structure. Specified concrete unconfined compressive strengths typically have ranged from 13 to 55 MPa (1,885 to 7,977 psi), with 35 MPa (5,076 psi) being a typical value achieved at 28 days.

3.2 CONVENTIONAL STEEL REINFORCEMENT

Concrete tensile strength is about one-tenth to one-fifth its compressive strength, so concrete cannot be relied upon to withstand tensile stresses. This limitation is overcome by embedding steel reinforcement in the concrete. The concrete and steel thus work in concert. In addition to resisting tensile loads, the bonded steel reinforcement is used to control the extent and width of cracks, resist inclined tensile stresses caused by shear forces, and assist in resistance of compressive forces, especially where it is desirable to reduce member cross sections. Steel reinforcement also is used in compression members to safeguard against the effects of unanticipated bending moments that could crack the member or even cause it to fail, and contributes to a reduction in creep deflections. The effectiveness of reinforced concrete as a structural material depends on the interfacial bonding between the steel and concrete so that it acts as a composite material, the passivating effect of the highly alkaline concrete environment to inhibit steel corrosion, and the nearly identical coefficients of thermal expansion of the concrete and steel.

Reinforced concrete has been utilized in all LWR facilities. Most of the mild, or conventional, reinforcing steels [22] used in NPPs to provide primary tensile and shear load resistance/transfer consist of plain carbon steel bar stock with deformations (lugs or protrusions) on the surface. These bars typically conform to ASTM A 615 [23] or A 706 [24] specifications (initial plants may contain bars conforming to ASTM A 432 [25] or A 305 [26] specifications that have been either replaced by the above or withdrawn). The minimum yield strength of the steel reinforcement ranges from 280 to 520 MPa (40,600 to 75,400 psi), with the 420 MPa (60,900 psi) strength material being most common and is available in bar size designations from #3 to #18.

3.3 PRESTRESSING STEEL

Post-tensioning is a method of effectively reinforcing (or strengthening) concrete structures with high-strength steel wire, strands, or bars, typically referred to as tendons. The tendons are installed, tensioned to prestress the concrete, and then anchored to the hardened concrete forming the structure. The post-tensioning system resists tensile loadings and is used to apply compressive forces to the concrete to provide increased resistance to concrete cracking. The post-tensioning system is generally utilized in conjunction with conventional steel reinforcement. Whereas conventional steel reinforcement is considered passive, a post-tensioning system can be considered as active.

A number of NPP concrete containment structures utilize post-tensioned steel tendons to provide primary resistance to tensile loadings. Three major categories of post-tensioning systems exist and are identified according to the type of material utilized to fabricate the tendons: wire, strand, or bar. The materials used to fabricate the tendons for these systems conform to ASTM specifications A 421[27], A 416 [28], and A 722 [29], respectively. Minimum tensile strengths range from 1,620 to 1,725 MPa (235,000 to 250,000 psi), for the A 421 material and 1,725 to 1,860 MPa (250,000 to 270,000 psi) for the A 416 material. The A 722 material has a minimum tensile strength of 1,035 MPa (150,000 psi). Typical NPP tendon systems group sufficient numbers of wires, strands, or bars to have minimum ultimate strengths ranging from 2,000 to 10,000 kN (450,000 to 2,240,000 lbf). The trend has been to increase the strength of the tendons to reduce the total number (e.g., in the early 1970s the typical tendon had a capacity of 3000 kN (674,000 lbf) and since has progressed to capacities of 15,300 kN or 3,400,000 lbf) [11].

Typically, the tendons are installed within preplaced ducts in the containment structure and post-tensioned from one or both ends after the concrete has achieved sufficient strength. After being tensioned, the tendons are anchored by buttonheads, wedges, or nuts. Corrosion protection is provided by filling the ducts with wax or corrosion-inhibiting grease (unbonded), or Portland cement grout (bonded). Supplemental conventional reinforcing is also used to minimize shrinkage or temperature effects and to provide local load-carrying capacity or load transfer. With the exception of Robinson 2 (bar tendons) and Three Mile Island 2 (strand tendons), plants that have post-tensioned containments utilize unbonded tendons so that the tendons can be inspected and replaced (if necessary). Bellefonte and Ginna each have grouted tendons (rock anchors) to which tendons are attached.

3.4 LINER PLATE AND STRUCTURAL STEEL

Leak-tightness of reinforced and post-tensioned concrete containment vessels is provided by a steel liner plate. A typical liner is composed of steel plate stock less than 13 mm thick (0.04 ft), joined by welding and anchored to the concrete by studs (headed anchors or similar conforming to ASTM A 108 [30]), structural steel shapes, or other steel products. The drywell portions of BWR containments and PWR containments are typically lined with carbon steel (ASTM A 36 [22] or A 516 [31]). The liners of LWR fuel pool structures are typically constructed of stainless steel plates (ASTM A 276 [32] or A 304 [33]). The liners of wetwells also have used carbon steel materials such as ASTM A 285 [34], A 516 [31], and A 537 [35]. Certain LWR facilities also have used carbon steel clad with stainless steel weld metal as liner members. Although the liner’s primary function is to provide a leak-tight barrier, it acts as part of the formwork during concrete placement and may be used in the support of internal piping/equipment. The liner is not considered to contribute to the strength of the structure; however, the interaction of the concrete and liner is important (e.g., prevention of liner buckling due to differential creep).

Structural steel has been used as a material of construction for BWR and PWR internal structures (e.g., operating and intermediate floors). The steel components of internal structures are typically fabricated of carbon steel materials such as noted above.

4. DURABILITY MECHANISMS AND PROCESSES

4.1 INTRODUCTION

The safety-related concrete structures in NPPs are designed to withstand loadings from a number of low-probability external and internal events, such as earthquake, tornado, and loss-of-coolant accident. Consequently they are robust and not subjected to high enough stresses during normal operation to cause appreciable degradation. In general, this has proven to be the case because the performance of reinforced concrete structures in NPPs has been good. Initially, the reported incidents of degradation occurred early in the life of the structures and primarily were attributed to construction or design deficiencies and improper material selection. However, as the NPPs age, degradation incidences are starting to occur at an increasing rate, primarily owing to environment-related factors. Although the vast majority of these structures will continue to meet their functional and performance requirements during the initial licensing period (i.e., nominally 40 years) as well as during periods of continued service (i.e., 20 years), it is reasonable to assume that with the increasing age of the operating reactors, there will be isolated examples where the structures may not exhibit the desired durability without some form of intervention.

Degradation of a reinforced concrete structure can result in the inability of the structure to meet its functional or performance requirements (e.g., ability to retain a fluid or loss of load-carrying capacity). Degradation can affect a component or member within a structure (local) or the entire structure (global). Whether or not a concrete structure will degrade is a function of many factors, including the constituent materials, its location (e.g., coastal or inland), climatic conditions (e.g., temperature and moisture), and the presence of external agents (e.g., sulfates and other chemicals). When the concrete mix design and reinforcement cover meet the prescriptive requirements of standards and codes, and the concrete is properly mixed, placed, compacted, and cured, durability problems attributable to concrete as a material are relatively rare. However, concrete may deteriorate as a result of violation of one of the previous items, inadequate design or construction practices, lack of maintenance, or because an inadequate concrete was specified [36].

4.2 DESIGN, CONSTRUCTION, AND MAINTENANCE CONSIDERATIONS

Design errors that can lead to subsequent deterioration of concrete structures can be placed into two categories: inadequate structural design and lack of attention to details [37]. Inadequate structural design occurs when the structure is exposed to a load greater than it is capable of carrying or it sustains greater strain than its strain capacity. Inadequate considerations of temperature change or concrete creep and accidental impact can also result in damage. Typical symptoms of inadequate design include spalling or expansion and cracking of concrete. Poor detailing of a structure may result in localized concentration of stresses that results in cracking, which in turn can permit water or chemicals to access the concrete, or ponding of water to produce saturated concrete. Poor detailing does not generally lead directly to concrete failure but can contribute to the action of one of the other specific causes of concrete failure [37]. Examples of inadequate structural design include insufficient concrete cover over steel reinforcement, lack of shear reinforcement, improper sizing and placement of steel reinforcement, inadequate section geometry, inadequate provision for drainage, abrupt changes in section, material incompatibility, and inadequate provision for deflection.

Poor construction practices and negligence can result from not following specified procedures or carelessness. Poor construction practices do not lead directly to failure or deterioration of concrete but can cause defects that lead to concrete cracking. Examples of concrete cracks that can result from poor construction practices include plastic shrinkage, plastic settlement, early thermal contraction, crazing, and long-term drying shrinkage. The resulting concrete cracking then can enhance the adverse effects of mechanisms such as described in the next section and can lead to further concrete degradation. Poor construction practices and negligence are best addressed through adequate quality assurance/quality control in conjunction with an aggressive inspection program. Examples of poor construction practice include adding more water to concrete to facilitate placement or finishing, improper mixing and curing, improper consolidation/vibration of materials, and improper location of steel reinforcement. Lack of knowledge about the importance of careful selection and specification of materials and use of admixtures can also result in durability issues. This can include improper cement contents, use of aggregates having poor quality or being contaminated with materials that can degrade the concrete properties or affect setting, incorporation of additives that can produce corrosion (such as calcium chloride accelerators), and incorrect water-cement ratios.

Improper or inadequate maintenance can contribute to the deterioration of concrete structures. Examples of inadequate maintenance include moisture exposure and penetration caused by unrepaired cracks, improper application of coatings, failure to clean drains and drain pathways, and improper monitoring of the forces in tendons of prestressed concrete components.

4.3 ENVIRONMENTAL STRESSOR CONSIDERATIONS

The longevity or long-term performance of safety-related concrete structures is primarily a function of the durability or propensity of these structures to withstand the potential effects of degradation. Table 4.1 presents a summary of the degradation factors that can affect the performance of the basic constituents that compose safety-related concrete structures in NPPs (i.e., concrete, reinforcing steel, post-tensioning system, and liner steel) [38]. Also contained in the table for each degradation factor is a listing of the aging mechanism, aging effect, potential degradation sites, and remarks (e.g., significance).

Table 4.1. Degradation factors that can affect the performance of reinforced concrete safety-related structures [38]

Aging stressors/service conditions	Aging mechanism	Aging effect	Potential degradation sites	Remarks (e.g., significance)
Concrete				
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 ground water	Sulfate attack	Expansion and irregular cracking	Subgrade structures and foundations	Sulfate-resisting cements or partial replacement of cements used to minimize 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
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/vibration	Fatigue	Cracking; strength loss	Equipment/piping supports	Localized damage; fatigue failure of concrete structures unusual
Exposure to flowing gas or liquid carrying particulates and abrasive components	Abrasion; erosion; cavitation	Section loss; loss of cover to expose rebar to corrosion	Cooling water intake and discharge structures	Unlikely to be an issue for containment structures; intake structures at most risk
Exposure to thermal cycles at relatively low temperatures	Freezing and thawing	Cracking; spalling	External surfaces where geometry supports moisture accumulation	Air-entrainment utilized to minimize potential occurrence

Table 4.1. Degradation factors that can affect the performance of reinforced concrete safety-related structures [38] (continued)

Aging stressors/service conditions	Aging mechanism	Aging effect	Potential degradation sites	Remarks (e.g., significance)
Concrete (cont.)				
Thermal exposure/thermal cycling	Moisture content changes and material incompatibility due to different thermal expansion values	Cracking; spalling; 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 loss
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 (e.g., $<10^{19}$ neutrons/cm^2 or $<10^{10}$ rads dose)
Consolidation or movement of soil on which structure founded	Differential settlement	Equipment alignment; cracking	Compacted structures on independent foundations	Allowance made in design; soil sites generally include settlement monitoring instrumentation
Exposure to water containing dissolved salts (e.g., seawater)	Salt crystallization	Cracking and scaling	Surfaces subject to salt spray; intake structures; foundations	Minimized through use of low-permeability concretes, sealers, and barriers
Mild steel reinforcement				
Depassivation of steel due to carbonation or presence of chlorides	Composition or corrosion 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	Irradiation levels likely to be below threshold levels to cause degradation

Table 4.1. Degradation factors that can affect the performance of reinforced concrete safety-related structures [38] (continued)

Aging stressors/service conditions	Aging mechanism	Aging effect	Potential degradation sites	Remarks (e.g., significance)
Mild steel reinforcement (cont.)				
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
Prestressing systems				
Localized pitting, general corrosion, stress corrosion, or hydrogen embrittlement	Corrosion due to specific environmental exposure (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 for corrosion than mild steel reinforcement
Elevated temperature	Microcrystalline changes	Reduction of strength; increased relaxation and creep	Near hot process and steam piping	Thermal exposure not likely to reach levels that can produce aging effects in prestressing
Irradiation	Microstructural transformation	Increased strength; reduced ductility	Structure 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 4.1. Degradation factors that can affect the performance of reinforced concrete safety-related structures [38] (continued)

Aging stressors/service conditions	Aging mechanism	Aging effect	Potential degradation sites	Remarks (e.g., significance)
Containment Liners				
Electrochemical reaction with environment (metallic liners)	Composition or concentration cells leading to general or pitting corrosion	Loss of cross section; reduced leak tightness	Areas of moisture storage/ accumulation, exposure to chemical spills, or borated water	Corrosion has been noted in several containments near where the liner becomes embedded in the concrete
Elevated temperature (metallic liners)	Microcrystalline changes	Reduction of strength; increased ductility	Near hot process and steam piping	Thermal exposure not likely to reach levels that can produce aging effects in metal liners
Irradiation (metallic and nonmetallic liners)	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 nonmetallic liners)	Fatigue	Cracking; reduced leak tightness	Inside surfaces of concrete containment building	Not likely as cyclic loadings are generally small in number and magnitude
Localized effects (nonmetallic liners)	Impact loadings; stress concentrations; physical and chemical changes of concrete	Cracking; reduced leak tightness	Inside surfaces of concrete containment building	Potential problem in high-traffic areas

4.3.1 Concrete Material Systems

The durability of concrete can be limited as a result of adverse performance of its cement-paste matrix or aggregate constituents due to either physical or chemical processes. In practice, these processes may occur concurrently to reinforce each other. In nearly all physical and chemical processes influencing the durability of concrete structures, dominant factors involved include transport mechanisms within the pores and cracks, and the presence of water. Transport mechanisms important in the consideration of durability of concrete include [39]

- diffusion of gases (e.g., CO_2, O_2, and water vapor) through empty pockets, microcracks, and the interfaces between components;

- diffusion of ions (e.g., chlorides and sulfates) in the concrete pore solution and dissolved gases;
- permeation of water or aqueous solutions under hydraulic head (submerged concrete or water-control structures) [40]; and
- capillary suction of water (water absorption) or aqueous solutions in empty or unsaturated capillaries.

Transport characteristics, however, do not provide information on the rate or extent of reaction, or the total amount of substance reacting with the aggressive material, and therefore only provide an indication of a material's durability [39]. Table 4.2 provides an indication of the influence of moisture (relative humidity) on several deterioration processes in concrete [41].

Table 4.2. Influence of moisture state on selected durability processes [41]

Ambient relative humidity	**Relative severity of deterioration process**[a]				
	Carbonation of concrete	**Frost attack on concrete**	**Chemical attack on concrete**	**Risk of steel corrosion**	
				In carbonated concrete	**In chloride-rich concrete**
Very low (<40%)	1	0	0	0	0[b]
Low (40–60%)	3[c]	0	0	1	1
Medium (60–80%)	2[d]	0	0	3	3
High (80–90%)	1	2	1	2	3
Saturated (>98%)	0	3	3	1	1

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

4.3.1.1 Physical Processes

Physical attack involves the degradation of concrete from external influences and generally results in cracking due to exceeding the tensile strength of the concrete, or loss of surface material. Although load-induced cracking is not considered an aging mechanism, it is included in the discussion below because it can affect future durability.

Cracking

Microcracks, or microscopic cracks, are important from the standpoint that they are considered to be the major cause of concrete's nonlinear stress-strain behavior in compression. Also, under load the microcracks can coalesce into macrocracks that can affect the durability of concrete structures.

Cracking occurs in virtually all concrete structures and, because of concrete's inherently low tensile strength and brittleness (e.g., lack of ductility), can never be totally eliminated. Cracking plays an important role in concrete's response in both tension and compression. Cracks and crack patterns have different characteristics depending on the underlying cause. Macrocracks are significant from the standpoint that they can indicate major structural problems (active cracks), provide an important avenue for the 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 leak-tightness or shielding capacity). Figure 4.1 presents examples of the most common types of intrinsic cracks that form in concrete as well as an indication of their potential time of occurrence [42]. Additional information on cracking and its classification with respect to damage is available [43].

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

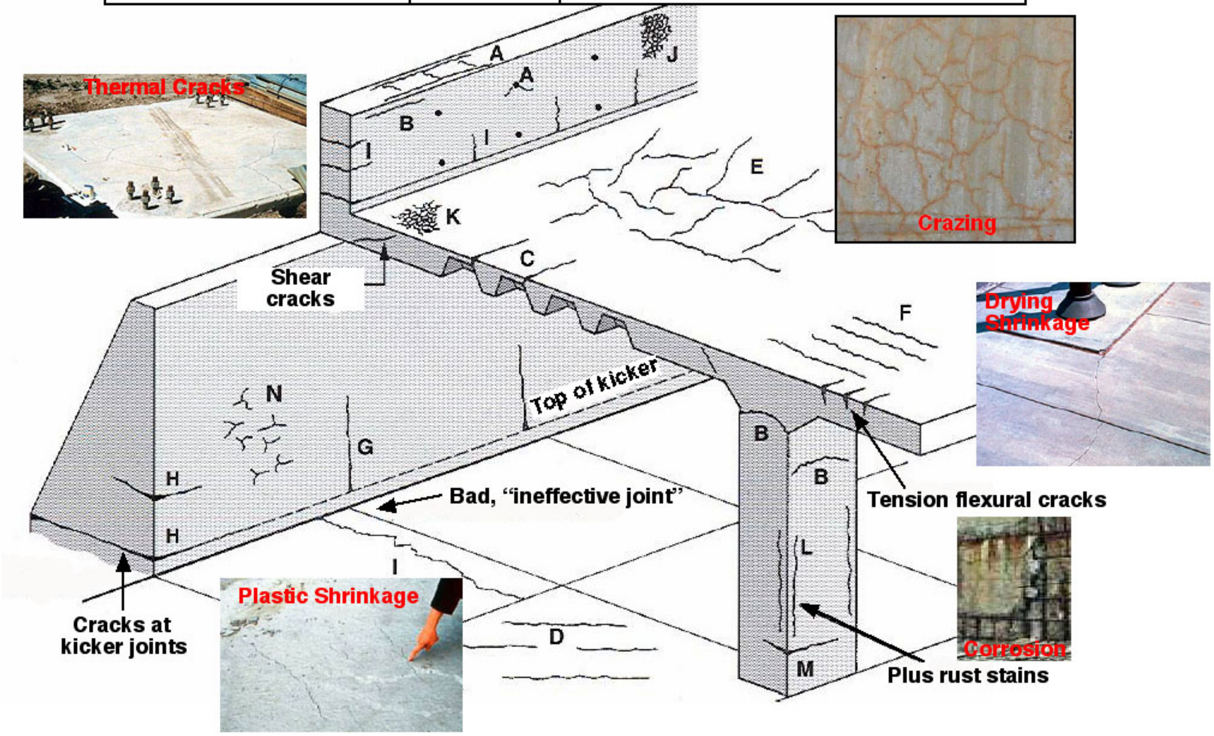

Figure 4.1. Examples of intrinsic cracks in a hypothetical structure [42].

Salt Crystallization

Physical salt attack is caused by the movement of salt solution by capillary action through the concrete and subsequent crystallization through drying. The process is repeated through cycles of wetting and drying. Crystallization and recrystallization of certain salts (e.g., NaCl, $CaSO_4$, and $NaSO_4$) can generate expansive forces that result in the physical breakdown of the concrete. The mechanism is somewhat similar to freezing and thawing of water in concrete. Structures in contact with fluctuating water levels or in contact with groundwater containing large quantities of dissolved salts are susceptible to this type of deterioration. Above ground level the moisture is drawn to the concrete surface, where it evaporates, leaving crystals of salt growing in the near-surface pores. The result is an area of deterioration just above ground level. The problem of salt crystallization is minimized for low-permeability concretes and where sealers or barriers have been effectively applied to prevent water ingress or subsequent evaporation.

Freezing and Thawing Attack

Concrete, when in a saturated or near-saturated condition, can be susceptible to damage, visible on the surface, during freezing and thawing cycles produced by the natural environment or industrial processes. Damage can occur in both the cement paste and aggregate phases of the concrete. Structures constructed without adequate air entrainment and portions of structures where moisture can accumulate are at greatest risk. The damage is incurred after an extended number of cycles and is observed on exposed surfaces of affected structures. One hypothesis is that the damage is caused by hydraulic pressure generated in the capillary cavities of the cement paste while critically saturated as the water freezes (when the water freezes, it expands about 9%). When the pressure in the cavity due to expansion of the water as it freezes exceeds the tensile strength of the concrete, cracking occurs.

Damage to concrete resulting from freezing and thawing attack can take several forms:[*] expansion, internal cracking, and spalling; scaling associated with application of salt; and pop-outs [44, 45]. Internal damage is confined primarily to the mortar and is associated with freeze-thaw damage of young concrete or mature cement paste that does not have a pore structure capable of resisting stresses that develop from freeze-thaw occurrence. Scaling is delamination of the concrete surface. Weakness may exist at the surface because of an inherent weakness of the concrete as a result of excessive water, excess mortar, or treatment during construction. The surface layer may become detached if the stresses occurring exceed the tensile or bond strength holding the layer to the substrate. Scaling may develop from a shallow surface feature into internal damage. It often is associated with application of de-icing chemicals that cause the temperature of the concrete surface to change rapidly, which induces a thermal shock that can cause cracking and surface scaling. Also, if salts are present in the pore solution, the osmotic pressure is increased since moisture tends to move toward zones of higher salt concentrations. A pop-out is a small volume of concrete that has separated from the body of the concrete to leave a roughly conical depression. The most common cause of pop-outs is stress resulting from freeze-thaw action within the coarse aggregate particle that causes cracking of the particle and simultaneous fracture of the concrete between the particle and nearest concrete face. Aggregates that produce pop-outs are generally sedimentary (e.g., cherts, sandstones, shales, and limestones) but can be calcareous or siliceous or gravel or crushed rock and have a high porosity [46]. Internal damage occurs in the form of cracking, is confined to mortar, and is associated with freeze-thaw damage to cement paste that does not have a pore structure capable of resisting the stresses that develop. Figure 4.2 presents a schematic of the types of freeze-thaw damage as well as examples.

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 to evaluate if the concrete was produced to provide resistance to freezing and thawing attack are available [4, 47, 48].

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[*] Pattern or “D” cracking can also occur, but it is predominately associated with pavements and is characterized by cracks developing toward the base of a slab at the edges and joints under permanent high-moisture conditions. The cracks may then spread inward and upward, eventually reaching the concrete surface.

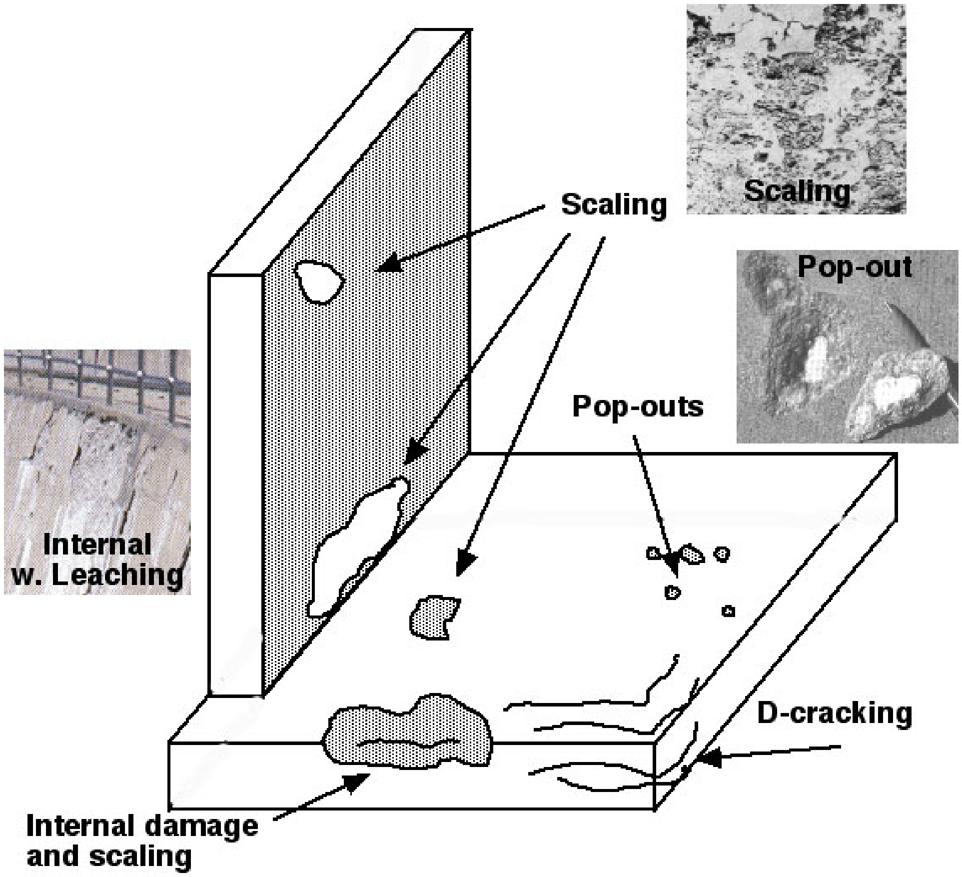

Figure 4.2. Types of freeze-thaw damage. Source: Adapted from [44].

Abrasion, Erosion, and Cavitation

Progressive loss of material at the concrete surface can occur from abrasion, erosion, or cavitation. Abrasion generally refers to dry attrition due to rubbing or grinding of aggregate or other debris on the concrete surface, while erosion is normally used to describe wear by the abrasive action of fluids containing solid particles in suspension. Mechanical abrasion is usually characterized by long shallow grooves in the concrete surface and spalling along monolithic joints. Concrete surfaces abraded by waterborne debris are generally smooth and may contain localized depressions. Cavitation is the formation of bubbles or cavities in a liquid. In hydraulic structures, the liquid is water, and the cavities are filled with water vapor and air. The cavities form where the local pressure drops to a value that will cause the water to vaporize at the prevailing water temperature. Formation of these cavities is usually triggered by concrete surface irregularities that are subjected to high-velocity water flow. Cavitation bubbles will grow and travel with the flowing water to an area where the pressure field will cause collapse. When a bubble collapses or implodes close to or against a solid surface, an extremely high pressure is generated, which acts on an infinitesimal area of the surface for a very short time.

Resistance of concrete to abrasion and erosion is dependent on the quality of the concrete and in particular the aggregate particles used in the mix. While good quality concrete may show good 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 causes of cavitation. Additional information on the effects of erosion on concrete structures is available [49].

Thermal Exposure and Thermal Cycling

Under elevated-temperature exposure, Portland cement paste experiences physical and chemical changes that contribute to development of shrinkage, transient creep, and changes in strength. Key material features of hydrated Portland cement paste affecting the properties of concrete at elevated temperature are its moisture state (i.e., sealed or unsealed), chemical structure (i.e., loss of chemically bound water from the C-S-H in the unsealed condition, CaO/SiO_2 ratio of the hydrate in the sealed condition, amount of $Ca(OH)_2$ crystals in sealed or unsealed conditions), and physical structure (i.e., total pore volume including cracks, average pore size, and amorphous/crystalline structure of solid) [50].

When concretes are exposed to elevated temperatures [e.g., T > 100 °C (212 °F)], changes in mechanical properties and durability can occur. Nonlinearities in material properties, variation of mechanical and physical properties with temperature, tensile cracking, and creep effects affect the buildup of thermal forces, the load-carrying capacity, and the deformation capability (i.e., ductility) of the concrete structural members. The property variations result largely because of changes in the moisture condition of the concrete constituents and the progressive deterioration of the cement paste-aggregate bond, which is especially critical where thermal expansion values for the cement paste and aggregate differ significantly. The bond region is affected by the surface roughness of the aggregate and its chemical and physical interactions [50]. Chemical interaction relates to the chemical reactions between the aggregate and cement paste that can be either beneficial or detrimental. Physical interaction relates to dimensional compatibility between aggregate materials and cement paste. Behavior of concrete at high temperature depends on exposure conditions (i.e., temperature-moisture-load-time regime). Curing influences the degree of hydration, while the temperature and load history prior to exposure to elevated temperature could have a significant effect on the behavior of the Portland cement paste, and therefore the concrete. Concrete at elevated temperature is sensitive to the temperature level, heating rate, thermal cycling, and temperature duration (as long as chemical and physical transformations occur).

A good summary of the degradation reactions that occur in Portland cement concrete is available [51].

- Upon first heating, substantial water evaporation occurs from the larger pores close to the concrete surface.
- From 100 °C (212 °F) onward, the evaporation proceeds at a faster rate with water being expelled from concrete near the surface as a result of above-atmospheric vapor pressure (i.e., steam flow).
- At 120 °C (248 °F) the expulsion of water physically bound in the smaller pores, or chemically combined, initiates and continues up to about 500 °C (932 °F) where the process is essentially complete.
- From 30 °C to 300 °C (86 °F to 572 °F), in conjunction with evaporation, dehydration of the hardened cement paste occurs (first stage) with the maximum rate of dehydration occurring at about 180 °C (356 °F). (Tobermorite gel is stable up to a temperature of 150 °C (302 °F) [52].)
- In the temperature range from 450 °C to 550 °C (842 °F to 1022 °F), there is decomposition of the Portlandite [i.e., $Ca(OH)_2 \rightarrow CaO + H_2O$)].

- At 570 °C (1058 °F) a transformation occurs where the $\alpha \rightarrow \beta$ inversion of quartz takes place, with the transformation being endothermic and reversible.

- A further process of decomposition of the hardened cement paste takes place between 600 °C and 700 °C (1112 °F and 1292 °F) with the decomposition of the calcium-silicate-hydrate phases and formation of β-C_2S.

- Between 600 °C and 900 °C (1112 °F and 1652 °F) the limestone begins to undergo decarbonation (i.e., $CaCO_3 \rightarrow CaO + CO_2$). The rate of decomposition and the temperature at which it occurs are not only dependent on temperature and pressure but also on the content of SiO_2 present in the limestone.

- Above 1200 °C (2192 °F) and up to 1300 °C (2372 °F), some components of the concrete begin to melt.

- Above 1300 °C (2372 °F) to 1400 °C (2552 °F) concrete exists in the form of a melt. Liquefaction of the concrete commences with melting of the hardened cement paste followed by melting of the aggregates according to Ref [53, 54]. The melting points of aggregates vary greatly. At 1060 °C (1940 °F) basalt is at the lower limit of all types of rock, with quartzite not melting below 1700 °C (3092 °F) [55].

The response of concrete in terms of strength loss has been divided into three ranges: 20 °C to 400 °C (68 °F to 752 °F), 400 °C to 800 °C (752 °F to 1472 °F), and above 800 °C (1472 °F) [56]. In the first range, it was noted that normal strength concretes (< 50 MPa) exhibit a slight loss of strength (~15%), whereas higher strength concretes (80 to 100 MPa) maintain their strengths. In the second range, both concretes lose most of their original strength, especially above 600 °C (1112 °F). It is within this range that dehydration of the calcium-silicate-hydrate gel is most significant. Above 800 °C (1472 °F) only a small fraction of the original concrete strength remains.

As some aggregates in concrete change color at elevated temperatures (e.g., sedimentary and metamorphic) [57],[*] the color changes can be used to estimate the temperature reached:[†]

- up to 300 °C (572 °F), the concrete color will be normal, its condition unaffected, with surface crazing around 290 °C (554 °F);

- from 300 °C to 600 °C (572 °F to 1112 °F), the concrete will be pink to red and apparently sound, but its strength will be significantly reduced;

- from 600 °C to 900 °C (1112 °F to 1652 °F), the concrete will be gray to buff, and weak and friable; and

- above 900 °C (1652 °F), it will have a buff color (limestone becomes white) with little to no strength [59, 60].

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[*] It should be noted that not all aggregates (e.g., igneous) exhibit color changes as a function of temperature.

[†] Other methods for indicating the magnitude of concrete thermal exposure include differential thermal analysis, X-ray diffraction, thermoluminescence test, and thin-section petrography [58].

The extent of color change varies with type of fine and coarse aggregate. If the magnitude of thermal exposure is known, a rough estimate of the residual mechanical properties of concrete can be made. Because concrete's in situ compressive strength generally exceeds design requirements, the modest strength reductions resulting from temperature exposures up to 300 °C (572 °F) often can be tolerated.

Figure 4.3 presents the effect of temperature on the residual compressive strength* of several unsealed ordinary concretes made with various normal-weight aggregate materials and tested at room temperature after heat treatment [61]. However, applicability of information such as presented in Figure 4.3 needs to be evaluated for each concrete because a concrete's residual strength after exposure to elevated temperatures depends on a number of factors such as the temperature attained, type and porosity of aggregate, rate of heating, permeability, use of pozzolans (a supplement to Portland cement), moisture state, mix proportions, and loading and sealing conditions during heating.

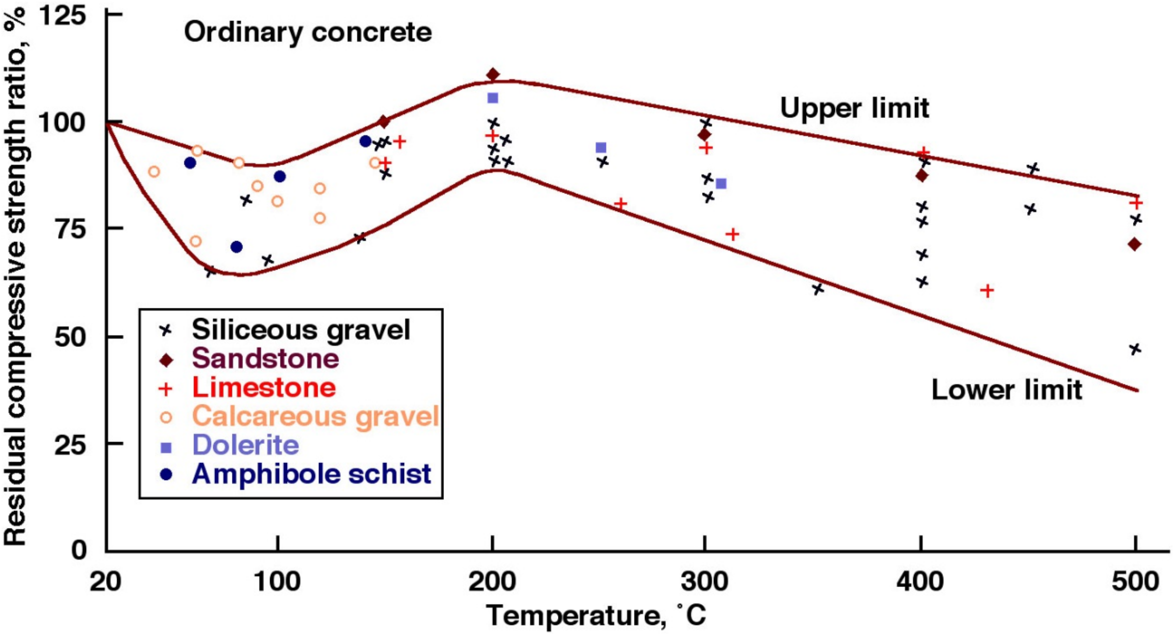

Figure 4.3. Effect of temperature on residual compressive strength: unsealed specimens [61].

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 diminish the bond between concrete and steel reinforcement [62]. Elevated temperatures also are important in that they affect the volume change and creep of concrete [63].

Thermal cycling, even at relatively low temperatures [i.e., 65 °C (149 °F)], can have deleterious effects on concrete's mechanical properties (i.e., compressive, tensile and bond strengths, and modulus of elasticity are reduced) [64]. 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 (392 to 572 °F)], 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

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* Residual ratio (%) = 100 × (value after heating)/(value before heating).

aggregate and matrix [65]. Thermal cycles also can become important if the deformation of the structure resulting from the temperature variations is constrained.

Additional information on the effects of elevated temperature on concrete materials and structures is available [66–68].

Irradiation

Irradiation in the form of 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. Changes in the properties of concrete appear to depend primarily on the behavior of the concrete aggregate, which can undergo a volume change when exposed to neutron radiation [69]. The fast neutrons are mainly responsible for the considerable growth, caused by atomic displacements, that has been measured in certain aggregate (e.g., flint). Quartz aggregates that contain crystals with covalent bonding should be more affected by radiation than calcareous aggregates that contain crystals with ionic bonding [70]. Furthermore, when nuclear radiation is attenuated or absorbed in concrete, almost all the absorbed radiation is converted into heat. 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 material. The heat generated may have detrimental effects on the physical, mechanical, and nuclear properties of the concrete. Reference [71] indicates that nuclear heating is negligible for incident energy fluxes less than 10^{10} MeV/cm^2 per second. Determination of whether any deterioration that may occur in concrete properties is due to radiation damage or thermal effects can be difficult.

Prolonged exposure of concrete to irradiation can result in decreases in tensile and compressive strengths and modulus of elasticity. Figure 4.4 presents a summary of the effects of neutron radiation on the compressive strength and modulus of elasticity of several concretes [69]. Results in the literature indicate that

- for some concretes, neutron irradiation of more than 1×10^{19} neutrons/cm^2 or 10^8 Gy (10^{10} rads) of dose for gamma radiation may cause a reduction in compressive strength;
- tensile strength of concrete is significantly reduced at neutron fluences exceeding 10^{19} n/cm^2, with the decrease of tensile strength caused by neutron irradiation more pronounced than the decrease of compressive strength;
- resistance of concrete to neutron irradiation apparently depends on the type of neutrons (slow or fast) involved, but the effect has not been clarified;
- resistance of concrete to neutron irradiation depends on mix proportions, type of cement, and type of aggregate;
- the effect of gamma irradiation on concrete’s mechanical properties requires clarification;
- the deterioration of concrete properties associated with a temperature rise resulting from irradiation is relatively minor;
- coefficients of thermal expansion and conductivity of irradiated concrete differ little from those of temperature-exposed concrete;

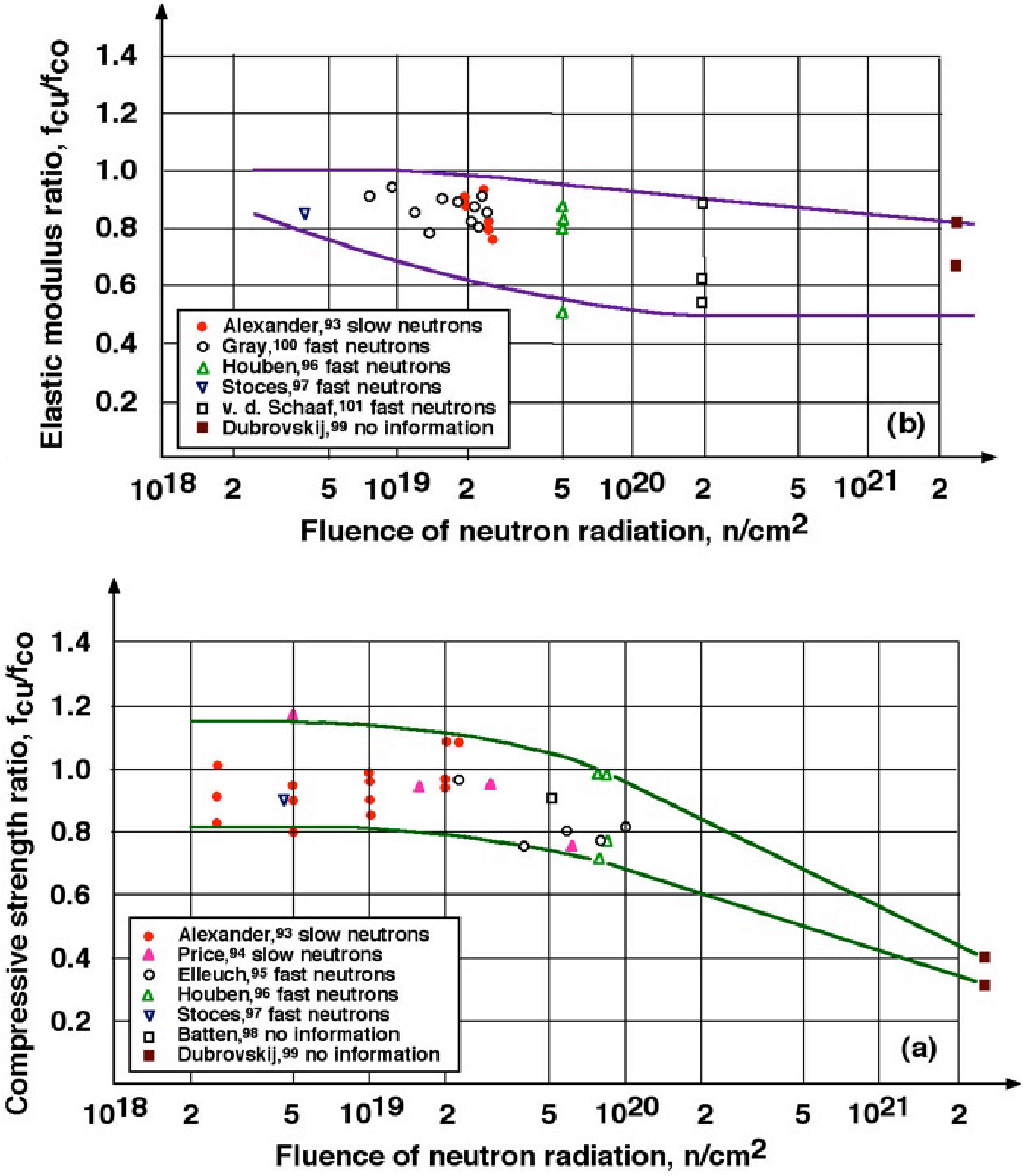

Figure 4.4. Effect of neutron radiation on concrete compressive strength and modulus of elasticity relative to unirradiated and unheated control specimen results [69]. Authorized reprint from ACI SP-55, American Concrete Institute, 1978.

- when exposed to neutron irradiation, the modulus of elasticity of concrete decreases with increasing neutron fluence;

- creep of concrete is not affected by low-level irradiation exposure, but for high levels of exposure, creep probably would increase with exposure because of the effects of irradiation on the concrete's tensile and compressive strengths;[*]

- for some concretes, neutron irradiation with a fluence of more than 1×10^{19} neutrons/cm^2 can cause a marked increase in volume;

- generally, concrete's irradiation resistance increases as the irradiation resistance of the aggregate increases; and

- irradiation has little effect on shielding properties of concrete beyond moisture loss caused by a temperature increase [69].

Furthermore, there is an indication that nuclear irradiation can significantly increase the reactivity of silica-rich aggregates to alkalis (i.e., alkali-silica reaction) [72]. Results from an investigation of the effect of γ-irradiation on the strength of a NPP concrete indicate that for a dose up to 6×10^5 Gy (6×10^7 rads), the compressive, splitting-tensile, and flexural strength of concrete decreased with dose, reaching reductions of about 10%, 5%, and 5%, respectively, at the maximum dose [73]. Also, the interaction of concrete with irradiation generated a succession of chemical reactions, starting with radiolysis of water and terminating in formation of calcite crystals, that decreased the size of pore space and the strength of the concrete [73].

Section III, Division 2 of the ASME Pressure Vessel and Piping Code gives an allowable radiation exposure level of 10×10^{20} nvt [8]. The British Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors states that the maximum permissible neutron dose is controlled by the effects of irradiation on concrete properties, and the effects are considered to be insignificant for doses up to 0.5×10^{18} neutrons/cm^2 [74]. Table 2.7 in Reference [75] provides data for estimated radiation environments at the outside surface of LWR pressure vessels for a 1000 MW(e) plant operating at a capacity factor of 80%. These results indicate that irradiation levels may approach the limits provided above in a concrete primary shield wall after 40 years of operation (32 equivalent full-power years). However, these values are upper limits and are probably higher than would be experienced because of the attenuating effects that would occur from other layers (e.g., air gaps, insulation) that could be positioned between the pressure vessel and concrete structures.

More detailed information on the interaction of radiation and concrete is available [76].

Fatigue and Vibration

Concrete structures subjected to fluctuations in loading, temperature, or moisture content (that are not large enough to cause failure in a single application) can be damaged by low-cycle fatigue. Fatigue damage initiates as microcracks in the cement paste, proximate to the large aggregate particles, reinforcing steel, or stress risers (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

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[*] 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.

deflections. Ultimate failure of a concrete structure in fatigue will occur as a result of excessive cracking, excessive deflections, or brittle fracture. As concrete ages and gains strength, for a given stress level the cycles to failure will increase. If the concrete is reinforced or prestressed, properties of the steel tend to control structural performance since the steel carries the tensile loads. Fatigue failure of concrete is unusual because of its good resistance to fatigue [77, 78], and concrete structures are designed with 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 as a result of tensioning-detensioning sequences occurring at locations where reciprocating equipment is attached, at supports for pipes that exhibit flow-induced vibrations, or at other locations subject to fluctuating loads.

Settlement

All structures have a tendency to settle during construction and early life. Settlement may be caused by errors in design of the foundations, either the result of incorrect assumptions about the properties and distribution of the soils and rocks below the structure, or errors in the structural design of elements such as pile caps [41]. In general, as a result of subgrade preparation (e.g., compacted material or use of piles), most of the settlement experienced by NPP structures has occurred within a few months after construction and became negligible after that period.

Uniform settlement will not normally cause structural distress, although if excessive it can result in damage or misalignment of connecting services or structures. Differential settlement is more of a concern as it can cause misalignment of equipment and can lead to overstress conditions in structures (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). Settlement is generally allowed for in the design of the structures and is not expected to be significant. When the structure such as an NPP is sited on soils, the potential for settlement is acknowledged and monitoring programs may be implemented to confirm that design allowables are not exceeded.

4.3.1.2 Chemical Processes

Well-designed and well-constructed concrete generally performs well when exposed to various atmospheric conditions, water, and soil. Concrete is rarely, if ever, attacked by solid, dry chemicals. However, some chemical environments can reduce the service life of even high-quality concrete. To produce significant degradation of concrete the aggressive chemicals must be in solution and sufficiently concentrated or reach a critical concentration after evaporation of the solution. Also, for maximum effect, the chemical solution needs to be circulated in contact with the concrete. In addition to the specific nature of the chemical involved, the degree to which concrete resists attack is dependent on the temperature of the aggressive solution, the water-cement ratio of the concrete, the type of cement used, the degree of consolidation of the concrete, the concrete permeability, and the degree of wetting and drying of the chemical on the concrete.

Chemical attack involves the alteration of concrete through chemical reaction with the cement paste, the coarse aggregate, or the embedded steel reinforcement.[*] Generally, the attack occurs on the exposed surface region of the concrete (cover concrete), but with the presence of

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[*] Corrosion of embedded steel reinforcement caused by carbonation of the concrete or the action of chloride ions is covered under Section 4.3.2.1 addressing mild steel reinforcement.

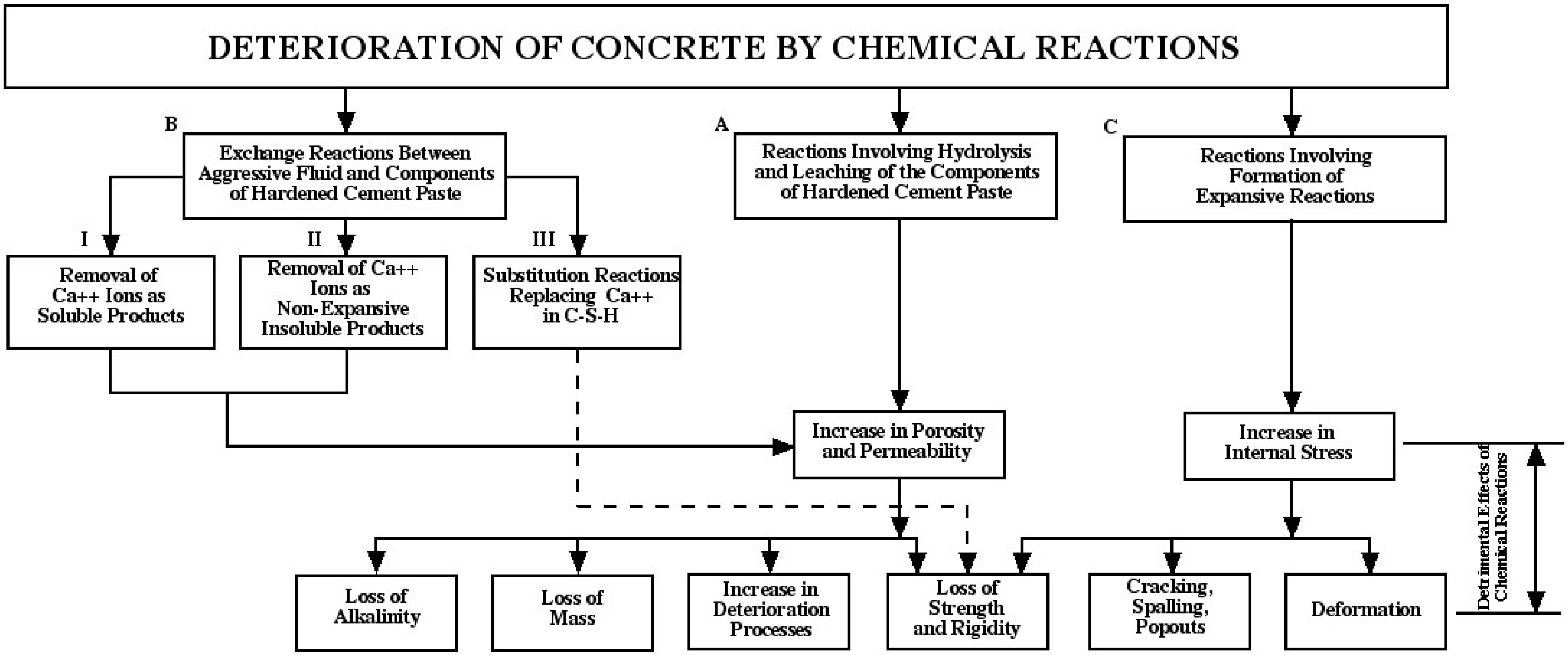
cracks or prolonged exposure, chemical attack can affect entire structural cross sections. Chemical causes of deterioration can be grouped into three categories: (1) hydrolysis of cement paste components by soft water; (2) cation exchange reactions between aggressive fluids and the cement paste; and (3) reactions leading to formation of expansion products [79].

Figure 4.5 presents a summary of the types of chemical reactions responsible for concrete deterioration and the detrimental effects that can occur [80]. Chemical attack of concrete may occur in several different forms as highlighted in the following sections.

Efflorescence and Leaching

Efflorescence is a crystalline deposit of salts, usually white, that occurs on or near 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, usually calcium, sodium, or potassium carbonate, as a result of evaporation of the fluid or interaction with carbon dioxide in the atmosphere. Occasionally, efflorescence may be a symptom of chemical reactions such as sulfate attack or it may indicate leaks in a water-retaining structure or undesired leakage of moisture through a structure. To establish that damage has occurred, it is essential to demonstrate that deleterious reactions have occurred in the interior of the concrete or at its surfaces in contact with sulfates in the surrounding soil [81]. Typically, however, efflorescence is primarily an aesthetic problem rather than affecting the concrete mechanical properties or durability. In rare cases, excessive efflorescence deposits can occur within the surface pores of the concrete, causing expansion that may disrupt the surface [82].

Leaching of cementitious materials mainly involves the transportation of ions from the interior of the material through its pore system outward into the surroundings. In the leaching process solid compounds in the concrete are dissolved by water that has penetrated the concrete followed by subsequent transport away either by diffusion based on the concentration gradients or convection through the flow of water. Pure water that contains little or no calcium ions, or acidic groundwater (acidity 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, the 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). Concrete leaching is basically of three types: (1) leaching at free surfaces of the concrete, (2) leaching from the interior of concrete, and (3) leaching at surfaces of cracks in concrete [83]. Of these types, (1) is generally of little importance, (2) is serious and can result in serious damage, and (3) is difficult to deal with and complicated to assess. Concretes produced with low water-to-cement ratios, adequate cement content, and proper compaction and curing are most resistant to leaching.



A: Softwater attack on calcium hydroxide and C-S-H present in hydrated portland cements;
B(I): acidic solution forming soluble calcium compounds such as calcium sulfate, calcium acetate, or calcium bicarbonate;
B(II): solutions of oxalic acid and its salts, forming calcium oxalate;
B(III): long-term seawater attack weakening the C-S-H by substitution of Mg^{++} for Ca^{++};
C(1): sulfate attack forming ettringite and gypsum;
C(2): alkali-aggregate attack;
C(3): corrosion of steel in concrete; and
C(4): hydration of crystalline MgO and CaO.

Figure 4.5. Types of chemical reactions responsible for concrete deterioration [80].

Sulfate Attack

All sulfates are potentially harmful to concrete. Sulfate attack of concrete is caused by exposure of concrete products or structures to an excessive amount of sulfate from internal or external sources. Internal sulfate attack results when a soluble source of sulfates is incorporated into the concrete at the time of mixing through the hydraulic cement, presence of natural gypsum or pyrite in the aggregate, and admixtures. External sulfate attack is most common and typically occurs when water containing dissolved sulfates penetrates the concrete. Magnesium, sodium, calcium, and potassium sulfates present in soils, groundwater, and seawater react with the calcium hydroxide and if enough water is present, result in expansion and irregular cracking of the concrete that can lead to progressive loss of strength and mass. The degree of sulfate attack depends on water penetration, the sulfate salt and its concentration and type, the means by which the salt develops in the concrete (e.g., is it rising and drying causing crystallization), and the chemistry of the binder present in the concrete. Sulfate attack of hardened concrete generally appears in two forms: expansive formation of ettringite and gypsum, causing cracking and exfoliation, and softening and dissolution of the hydrated cementing compounds due to direct attack on these compounds by sulfate or by their decomposition when calcium hydroxide reacts with the sulfates and is removed [37]. The end result of sulfate attack can be excessive expansion, delamination, cracking, and loss of strength. Figure 4.6 illustrates the mechanism of sulfate (sodium) attack and presents an example of cracking resulting from sulfate attack. It has been reported that concrete may suffer sulfate attack at a concentration of about 0.2% sulfate content in groundwater, magnesium sulfate can be more aggressive than sodium sulfate, and there are three key chemical reactions between sulfate ions and hardened cement pastes: (1) recrystallization of ettringite, (2) formation of calcium sulfoaluminate (ettringite), and (3) decalcification of the main cementitious phase (calcium silicate hydrate) [84]. Guidelines for assessing the potential degree of severity of expected attack have been established by organizations such as the American Concrete Institute (ACI) [4], Canadian Standards Association (CSA) [85], Deutsches Institute für Normung (DIN) [86], and British Standards Institution [87]. Table 4.3 provides current ACI Building Code requirements for concrete exposed to sulfate-containing solutions [4].

Concrete structures that may be exposed to attack by sulfates in soils and groundwater include footings, foundation walls, retaining walls, piers, culverts, piles, pipes, and surface slabs. The severest attack occurs on elements where one side is exposed to sulfate solutions and evaporation can take place at the other [88]. Structures subjected to seawater are more resistant to sulfate attack because of the presence of chlorides that form chloro-aluminates to moderate the reaction. Concretes that use cements low in tricalcium aluminate (e.g., Type V sulfate resisting) and those that are dense and of low permeability are most resistant to sulfate attack.

A rare form of sulfate attack is through formation of thaumasite as a result of the reaction between the calcium silicates in the cement, calcium carbonate from limestone aggregates or fillers, and sulfates, usually from external sources [41]. Coincident factors for thaumasite sulfate attack in susceptible concrete include source of sulfates, presence of mobile groundwater, source of calcium silicate hydrate, presence of carbonate, and a low temperature [<10 °C (<50 °F)] [89]. A thaumasite sulfate attack progresses slowly and can destroy a significant part of the calcium silicate hydrate. Eventually, a soft, white, pulpy mass forms that causes total disintegration of the concrete and exposes the steel reinforcement. However, serious damage to concrete or masonry due to thaumasite formation has not been a common occurrence.

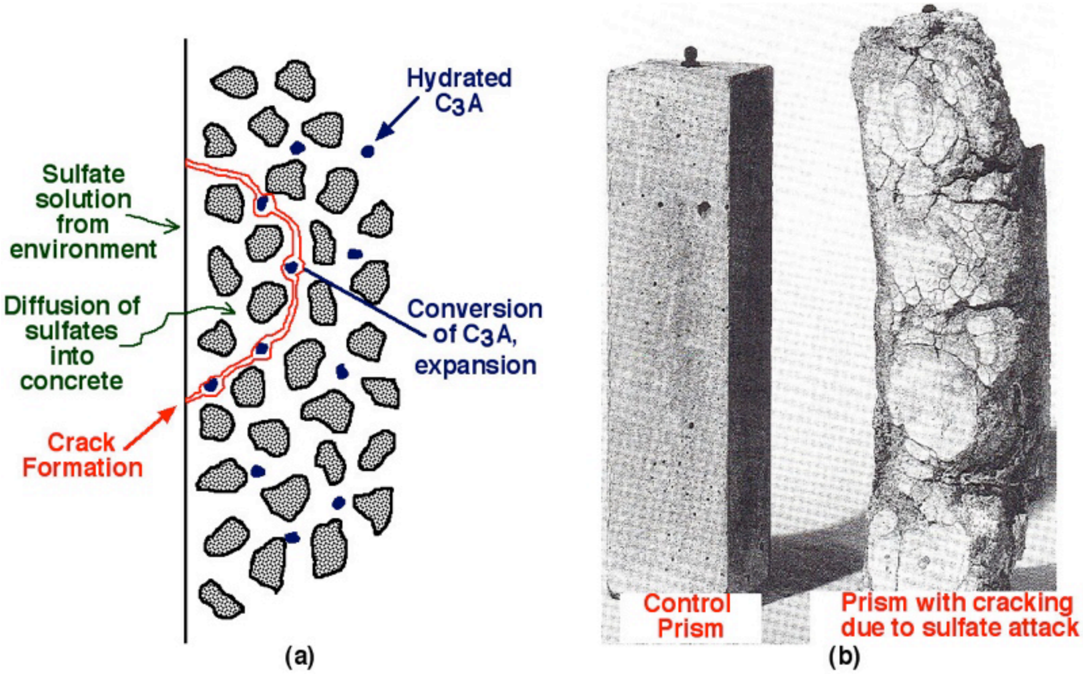


Figure 4.6. Concrete cracking due to sulfate attack: (a) mechanism, (b) example of concrete cracking due to sulfate attack [41].

Table 4.3. Building Code requirements for concrete exposed to sulfate-containing solutions [4]

Sulfate exposure	**Water soluble sulfate (SO_4) in soil (wt %)**	**Sulfate (SO_4) in water (ppm)**	**Cement type[a]**	**Maximum water-cementitious materials ratio, by weight, normal weight aggregate concrete[b]**	**Minimum f_c', normal weight aggregate concrete, psi (MPa)**
Negligible	0.00–0.10	0–150	-	-	-
Moderate[c]	0.10–0.20	150–1500	II, IP(MS), P(MS), I(PM)(MS), I(SM)(MS)	0.50	4000 (27.6)
Severe	0.20–2.00	1500–10,000	V	0.45	4500 (31.0)
Very severe	>2.00	>10,000	V plus pozzolan[d]	0.45	4500 (31.0)

[a] "Standard Specification for Portland Cement," ASTM C150-05, *ASTM International*, American Society of Testing and Materials, West Conshohocken, Pennsylvania, 2005.
[b] A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing.
[c] Seawater.
[d] Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.
Authorized reprint from ACI Standard 318-05, American Concrete Institute,2005.

Delayed Ettringite Formation

Ettringite formation by reaction of internal or external sulfate with anhydrous or hydrated calcium aluminates has an expansive character. When it occurs within several hours in a fresh concrete mixture, there is no destructive expansion. However, when ettringite forms at later ages (e.g., months or years) in a hardened concrete, delayed ettringite formation (DEF) can exhibit expansion and cracking. The distress often is attributed to improper heat treatment of concrete in which the ettringite formation is prevented or the ettringite that is normally formed during the early hydration of Portland cement decomposes [T > 70 °C (158 °F)]. Use of cements having high sulfate contents in which the sulfate has very low solubility also can lead to DEF. In this case, the sulfate concentrations in the pore liquid of the hardened concrete are high for an unusually long period of time. Eventually the sulfate will react with the calcium- and aluminum-containing phases of the cement paste, and the cement paste will expand, forming cracks around the aggregate particles. In one case where this has been reported, it was thought that the occurrence of DEF was the result of the sulfate formed in the cement clinker being present as anhydrite and as a component of the silicate phases that are slowly soluble [90]. If structures susceptible to DEF are later exposed to water, ettringite can reform in the paste as a massive development of needle-like crystals, causing expansive forces that result in cracking. The extent of development of DEF is dependent on the amount of sulfate available for late ettringite development in the particular concrete and on the presence of water during the service life. Elevated temperatures also increase the potential for damage due to DEF. Prevention or minimization of DEF can be accomplished by lowering the curing temperature, limiting clinker sulfate levels, avoiding excessive curing for potentially critical sulfate to aluminate ratios, preventing exposure to substantial water in service, and using proper air entrainment. Neither the mechanisms involved in DEF nor their potential consequences relative to concrete durability are completely understood. DEF leads to degradation in concrete mechanical properties, such as compressive strength, and can promote increased permeability. A detailed review of over 300 publications dealing with DEF is available [91]. Figure 4.7 presents an example of cracking damage to a concrete structure due to delayed ettringite formation.

Figure 4.7. Cracking damage in a concrete structure due to DEF [91].

Acids and Bases

In general, Portland cement concrete does not exhibit good resistance to acids and will not hold up for long if exposed to a solution with a pH of 3 or lower [92]. 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). Nitric, hydrochloric, and sulfuric acids are very aggressive as the calcium salts are readily soluble and removed from the acid front. Organic acids such as formic, acetic, and lactic are also corrosive to Portland cement concrete. Other acids such as phosphoric, carbonic, tannic, and humic are less harmful as the calcium salts have low solubility and inhibit the attack by blocking the pathways within the concrete. Oxalic acids have negligible effect on Portland cement concretes. Carbonic, humic, and sulfuric acids are the acids most commonly encountered by concrete since they are found in natural groundwater. Visual examination of concrete undergoing acid attack will show disintegration of the concrete in the form of loss of cement paste and aggregate from the matrix. Due to the large buffering capacity of concrete and the relatively small amount of acid contained in rain, acid rain will convert only an insignificant amount of the concrete [93]. Acid rain is even a smaller threat to NPP structures than general civil engineering concrete structures because of their massive cross sections. Surface coatings and a dense concrete with a low water-cement ratio provide improved resistance to acid environments.

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 disintegrate. 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 minimize attack by acids and bases generally involve the use of protective barrier systems. Table 4.4 presents a listing of the reactivity with concrete of various chemicals that may be found in NPPs or the surrounding environment. Additional information on the effect of chemicals on concrete is available [94].

Table 4.4. Reactivity of various materials with concrete and steel (primary source: [94])

Material	Effect on concrete	Effect on steel
Acetone	Liquid loss by penetration (may contain acetic acid and cause slow disintegration)	None
Acidic water (pH<6.5)	Disintegrates concrete slowly	May attack rebar and embedments
Boric acid	Negligible effect unless immersed	Severely corrosive to liner and reinforcing steel
Borated water (and boron)	Negligible effect unless immersed	Very corrosive at high concentration
Chlorine gas	Concrete (moist) slowly disintegrates	Highly corrosive
Demineralized water	Leaches	Slight
Deicing salt	Scaling of non-air entrained concrete	Highly corrosive
Diesel exhaust gas	May disintegrate moist concrete by action of carbonic, nitric, or sulfurous 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 10% concentration; disintegrates at concentrations >20%	Minimal
Sodium pentaborate	Disintegrates at varying rates depending on concentration	Dependent on concentration
Sulfates	Disintegrates at varying rates with concentration (concretes with low sulfate resistance such as Type I)	Harmful at certain concentrations
Sulfuric acid	Disintegrates rapidly in concentrations between 10 and 80%	Very corrosive

Authorized reprint from ACI 515.1R-79, American Concrete Institute, 1979.

Alkali-Aggregate Reactions

Expansion and cracking, leading to loss of strength, stiffness, and durability of concrete can result from chemical reactions involving alkali ions from the Portland cement, calcium and hydroxyl ions, and certain siliceous constituents in aggregates to form a calcium alkali-silicate gel. Expansion reactions also can occur as a result of alkali-carbonate reactions (i.e., dedolomitization). The type of aggregates susceptible to this type of reaction are typically dolomitic limestones that consist of a fine-grained matrix of calcite and clay in which larger crystals (20 to 80 mm or 0.07 to 0.26 ft) of euhedral dolomite rhombohedra are suspended [95]. A distinguishing feature that differentiates alkali-carbonate reaction from alkali-silica reaction is the lack of a silica gel exudation at cracks [47]. This gel takes up pore solution water due to forces of attraction between the polar water molecules and the alkali-silicate ions and expands, which can disrupt the concrete. Potentially damaging tensile stresses from 4,100 kPa to 11,000 kPa (595 to 1,595 psi) can develop within the cement paste matrix [96]. Figure 4.8 presents the mechanism of alkali-silica reaction (ASR) and the gel resulting from alkali-aggregate reaction that causes expansion and cracking. The primary factors influencing alkali-silica 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), presence of water, and temperature. Reactive aggregates have been identified in the United States as well as in the rest of the world. The potential for ASR to occur in newly constructed NPP concrete structures needs to be addressed; however, because current generation Portland cements have increased alkali contents that may result in reactivity of aggregates that were not reactive in the past, and the availability of good-quality aggregate materials is becoming limited in many areas of the United States.

The most reactive forms of aggregate are strained quartz, amorphous silica, cryptocrystalline quartz, chalcedony, and chert [97]. Table 4.5 presents a listing of some of the potentially harmful reactive minerals, rock, and synthetic materials that may cause deterioration of concrete when the reactive components are present in amounts as small as 1% [98]. In general, aggregates containing crystalline silica are stable, and those with amorphous or very fine-grained silica are reactive [98]. Although alkali-silica reactions initiate almost immediately after concrete mixing, they may not be noticeable until 5 to 10 years of construction, and evidence of deterioration may not appear 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 [45].

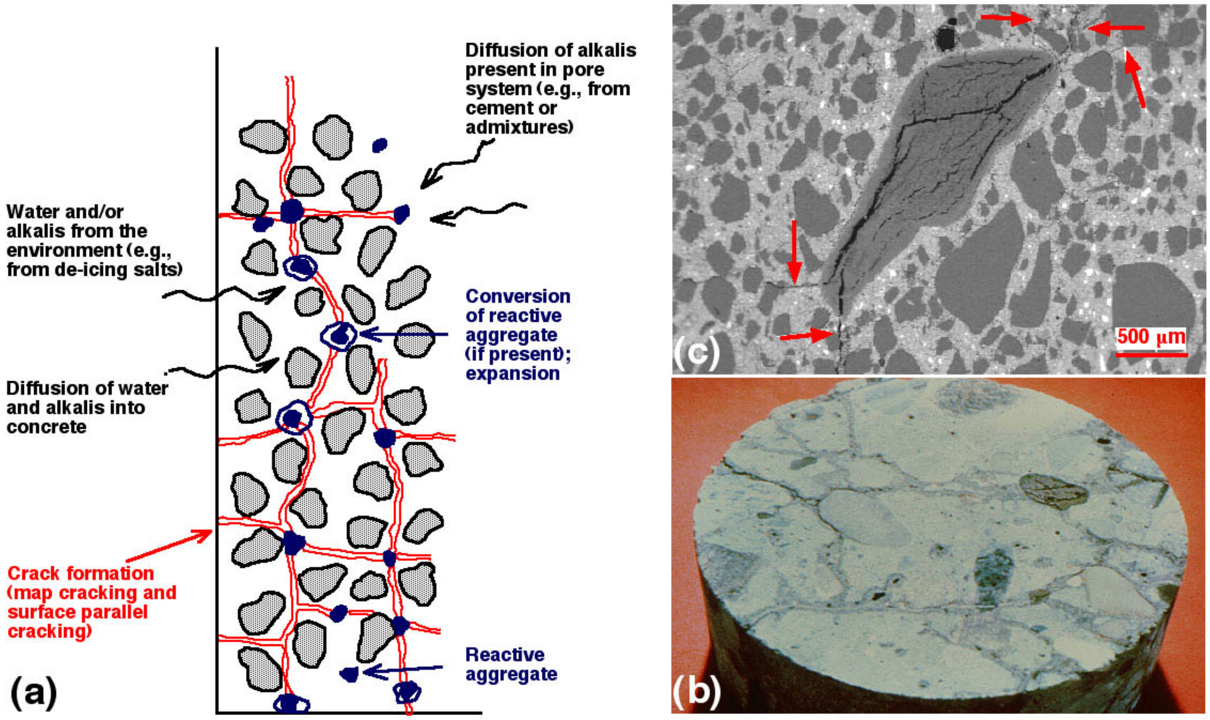


Figure 4.8. Concrete cracking due to alkali-silica reaction: (a) mechanism [41]; (b) resulting gel that causes expansion and cracking [99]; (c) polished section of concrete showing chert particle with extensive internal cracks extending from aggregate as noted by arrows [www.understanding-cement.com/alkali-silica.html, WHB Microanalysis Consultants Ltd., Suffolk, United Kingdom].

Table 4.5. Some potentially harmful reactive minerals, rock, and synthetic materials [85]

Alkali-silica reactive substances[a]		Alkali-carbonate reactive substances[b]
Andesites	Opal	Calcitic dolomites
Argillites	Opaline shales	Dolomitic limestones
Certain siliceous limestones and dolomites	Phylites	Fine-grained dolomites
Chalcedonic cherts	Quartzites	
Chalcedony	Quartzoses	
Cherts	Rhyolitic	
Cristobalite	Schists	
Dacitic	Siliceous shales	
Glassy or cryptocrystalline volcanics	Strained quartz and certain other forms of quartz	
Granite gneiss	Synthetic and natural siliceous glass	
Graywackes	Tridymite	
Metagraywackes		

[a] Several of rocks listed (e.g., granite, gneiss, and certain quartz formations) react very slowly and may not show evidence of any harmful degree of reactivity until concrete age is >20 years.

[b] Only certain sources of these materials have shown reactivity.

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In rare circumstances, ASR expansions can be as much as 2%–3% [100]. Crack widths up to 15 mm and crack depths to 300 mm (0.05 to 1 ft) have been observed in the field [101]. Since structures in service are stressed and cracked, expansive strains from ASR of 0.10%–0.20% superimposed over load-induced cracks can lead to permanent irreversible displacements. However, full-scale load tests on ASR-affected concrete structures and components indicate that visually severe ASR cracking can be deceptive and that the expansion and cracking that ASR induces may not lead to an unacceptably adverse effect on the structural performance of reinforced and prestressed concrete members [102]. No concrete structure or part of a structure has been reported to collapse due to ASR [102]. Alkali-silica reactions primarily affect the serviceability of the structure. Some of the most significant reported problems resulting from ASR are misalignment of structures, displacement of equipment, and spalling at joints. The effects of ASR on engineering properties often cannot be generalized since both the rate of expansion and the total expansion depend on the reactive aggregate, cement type, cement content, constraint, and environment. For expansive strains of 0.5%–1.5%, loss in compressive strength can vary from 40%–60%, whereas loss of tensile strength can be as high as 65%–80%, with loss of elastic modulus from 60%–80% [100]. Reference [103] provides some guidance to indicate the effects of ASR expansion on the residual compressive strength of concrete (lower bound) [i.e., for restrained expansions (mm/m) due to ASR of 0.5%, 1.0%, 2.5%, and 5.0%, the percentages of residual compressive strengths are 95%, 80%, 60%, and 60%, respectively]. In tests of lapped beams in which the effects of ASR on performance were evaluated, it was found that ASR causes a reduction in bond strength (up to 22% for those tests) and a significant reduction in the fatigue life [104]. Other investigators using lapped beams have shown reductions in bond strength in excess of 50% with losses for smooth bars greater than for ribbed bars [105]. Prestress developed by the ASR expansion can enhance the shear strength and stiffness of beams [106].

Visual detection of alkali-silica reactions is difficult in the early stages due to the fineness of the cracks and may go unrecognized for years. If the concrete member is unrestrained, visible concrete damage starts with small surface cracks exhibiting an irregular pattern (or map cracking). When the expansive forces are restrained (e.g., by reinforcement), the cracking pattern will be modified, as the expansion will develop in the direction of least constraint (i.e., in the presence of reinforcement, map-like cracks are replaced by cracks aligned with the reinforcement). Pop-outs and glassy-appearing seepage of varying composition can appear as a result of alkali-silica reactions. Furthermore, it is quite common that once cracking has developed, the cracks can allow access to the interior of the concrete to enable some other deleterious mechanisms to operate (e.g., leaching by percolating water accompanied by precipitation of calcium carbonate on surfaces, steel reinforcement corrosion, and freeze-thaw attack). It has been shown that alkali-silica reactions occurring in concretes contaminated with NaCl increases the risk of chloride-induced corrosion of steel reinforcement [107].

The best approach to prevention of alkali-aggregate reactions is to avoid using aggregates that are known to be or that are suspected of being reactive. Procedures for testing aggregates for reactivity and for minimizing the effects when reactive aggregates are used are available [37, 108]. In new concrete designs, low-alkali Portland cement (with an alkali content less than 0.6% Na_2O equivalent) has been successfully used with slightly to moderately reactive aggregates [96]. International codes or standards limit the alkali content of the concrete accounting for the cement factor and other internal sources of alkali [109]. The alkali content to prevent alkali-carbonate reaction is lower than that required for prevention of alkali-silica reaction [109]. The addition of fly ash has been shown to control ASR, but its effectiveness is highly dependent on the type of fly ash, its alkali content, chemical composition, and dosage rate [107]. Pozzolans are not effective in controlling alkali-carbonate reaction [96, 109]. Other types of finely divided minerals such as silica fume, GGBS, and natural pozzolans can also be effective in preventing ASR [109]. Also, addition of ASR-inhibiting compounds (e.g., lithium hydroxide) has been shown to be effective on highly reactive aggregates [110, 111].

Aggressive Water Attack[*]

Concrete in service may be exposed to aggressive waters, with the most common deleterious ion being sulfate [112]. In other waters, acids and chemical by-products from industrial processes may be present. Some locations have seawater or brackish water in contact with concrete. Most seawaters have a pH of 7.5 to 8.4, are fairly uniform in chemical composition, contain about 3.5% soluble salts by weight, with Na^+ and Cl^- having the highest ionic concentrations, but Mg^{2+} and SO_4^{2-} are also present [79]. Reaction of magnesium sulfate with the cement hydration products leads to formation of ettringite, calcium sulfate, and insoluble magnesium hydroxide (brucite) that reduces the rate of attack of dense concrete. The rate of attack is further reduced by the formation of aragonite (calcium carbonate), which forms more readily in the tidal zone than the surface layers of a completely immersed element [36].

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[*] Leaching of calcium hydroxide from concrete by flowing water was discussed earlier.

Concrete exposed to a marine environment may deteriorate as a result of combined effects of chemical action of seawater constituents on cement hydration products, alkali-aggregate expansion if reactive aggregates are present, crystallization pressure of salts within concrete if one face of the structure is subject to wetting and others to drying conditions, frost action in cold climates, corrosion of embedded steel reinforcement, and physical erosion due to wave action or floating objects.

Biological Attack

Growth on concrete structures may lead to mechanical deterioration caused by lichen, moss, algae, and roots of plants and trees penetrating into the concrete at cracks and weak spots, resulting in bursting forces, causing increased cracking and deterioration. Such growth can also retain water on the concrete surface, leading to a high moisture content with subsequent increased risk of deterioration due to freezing. Microgrowth may cause chemical attack by development of humic acid that can dissolve the cement paste [93].

Formation of capillaries within the concrete during the hydration process and the capillary action of water provide a means for penetration of microorganisms into concrete. The metabolism of microorganisms results in the excretion of sulfuric or nitric acid, which can contribute substantially to the degradation of cementitious materials. A number of organisms are capable of causing the dissolution of concrete through leaching of calcium and other alkaline-binding materials [113]. These organisms are known to cause serious damage to sewer lines, buildings, and monuments. In environments where sulfur compounds are present, such as sewers, the production of sulfuric acid by sulfur-oxidizing bacteria (thiobacilli) forms a corrosive layer that causes extensive cement degradation. The sulfuric acid reacts with the free lime [$Ca(OH)_2$] to form gypsum ($CaSO_4 \bullet 2H_2O$), which produces a corroding layer on the concrete surface that penetrates into the concrete. The newly formed gypsum crystals react with calcium aluminate in the cement to produce ettringite, which further contributes to the degradation of concrete by increasing the internal pressure, leading to formation of cracks, which in turn provide a larger surface area for the corrosion process [114]. In environments where reduced sulfur compounds are limiting, such as on buildings, nitric acid–producing bacteria have been found to play a role in concrete degradation [115]. The action of nitric acid on the calcareous components of concrete results in the production of calcium nitrate (soluble salt), which is either lost from the concrete, resulting in formation of corrosion pits, or remains to add salt to the pore water. The sulfate-reducing bacteria are primarily responsible for degradation of concrete aboveground and the nitrifying bacteria for degradation belowground [116]. Microbes have extremely diverse modes of metabolism, are natural inhabitants of soil, and can survive in extreme environments such as the inner wall of a geothermal cooling tower [117]. Concrete can also be corroded by gluconic, malic, and oxalic acids produced by fermentative bacteria that are natural soil inhabitants [118].

4.3.2 Mild Steel Reinforcing Systems

Although concrete has evolved to become the most widely used structural material in the world, its capacity for plastic deformation and its ability to absorb mechanically imparted energy is extremely limited. This shortcoming is generally overcome through the incorporation of mild steel reinforcement in locations where tensile stresses are anticipated. Fortunately the steel reinforcement and concrete are mutually compatible [e.g., similar coefficients of thermal expansion and the relatively high pH of the concrete pore water (~12.5–13.6) contribute to formation of oxide film that passivates the steel against corrosion]. Disruption of the passive film, however, can occur primarily due to carbonation or chloride intrusion that can lead to corrosion of the embedded steel. Durability concerns therefore need to address the performance of the embedded steel reinforcement as well as the interaction of the concrete and steel.

Mild steel reinforcing systems are provided in concrete structures to control the extent of cracking and the width of cracks at operating temperatures, resist tensile stresses and compressive stresses for elastic design, and provide structural reinforcement where required by limit condition design procedures [74]. 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 with respect to the durability of NPP concrete structures. Information on the other potential degradation factors is provided for completeness and for special situations that might occur.

4.3.2.1 Corrosion

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

Both water and oxygen must be present for corrosion to occur (i.e., there is no corrosion in dry concrete or in concrete fully immersed in water that does not contain entrained air). 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 reinforcement 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 [79]. As a result, one of 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. The cathode produces OH^- while rust forms at the anode with an associated increase in volume that can produce cracking and spalling of the concrete that affects its serviceability. Other potential causes of corrosion include the effects of stray electrical currents or galvanic action with an embedded steel of different metallurgy. Figure 4.9 illustrates the electrochemical process of steel corrosion in moist and permeable concrete. Four fundamental components are necessary for an electrochemical corrosion cell: (1) anode, (2) cathode, (3) electrolyte, and (4) electrical connection between anode and cathode.

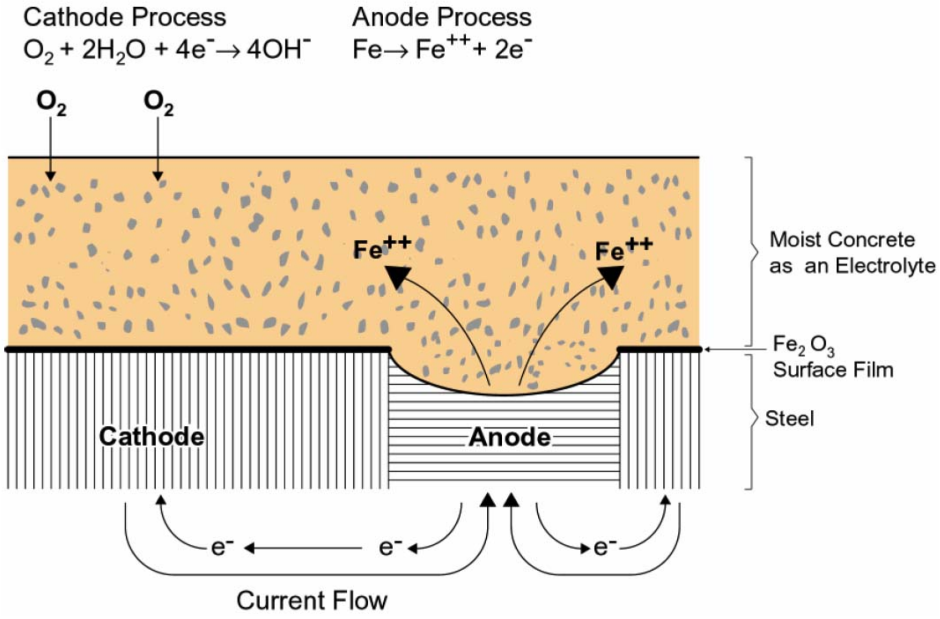

Figure 4.9. Electrochemical reaction illustrating corrosion of steel in concrete [119]. Reproduced with permission of McGraw-Hill Companies.

In good-quality, well-compacted concrete, reinforcing steel with adequate cover should not be susceptible to corrosion because the highly alkaline conditions present within the concrete (pH > 12) causes a passive iron oxide film (gamma Fe_2O_3) to form on the surface (i.e., metallic iron will not be available for anodic activity). When the passive film is relatively thick it inhibits corrosion by providing a diffusion barrier to the reaction products of the reacting species (Fe and O_2), or as is more common, the layer can be very thin. A thin passive film does not actually stop corrosion; it reduces the corrosion rate to an insignificant level [120]. Corrosion can occur if this passivating environment is altered by a reduction of the pH of the concrete, or by introduction of chlorides that destabilize the passive layer. Figure 4.10 summarizes primary and secondary factors that can depassivate steel reinforcement [121]. The discussion below will only address the primary factors.

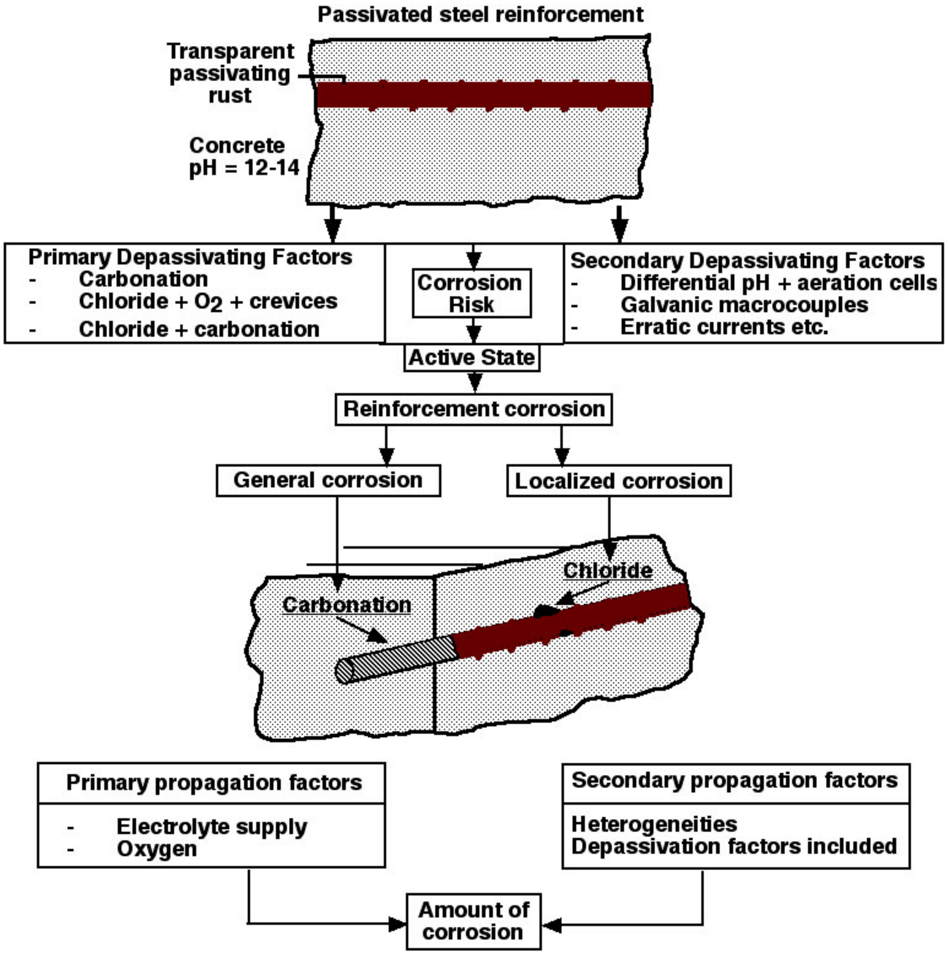

Figure 4.10. Factors leading to depassivation of steel in concrete [121]. With kind permission from Springer Science+Business Media: J. A. González, S. Feliu, S. Rodríguez, P. W. Ramírez, and C. Andrade, *Materials and Structures* 29, 40–46 (Jan-Feb 1996).

Reduction of the concrete pH can occur as a result of leaching of alkaline substances by water or carbonation [i.e., calcium hydroxide is converted to calcium carbonate (calcite)]. Carbonation causes the strength of concrete to increase, but this is generally of insignificant consequence because normally only the surface zone becomes carbonated. Although carbonation reduces the concrete permeability, it produces a greater propensity for shrinkage cracking that can negate the positive durability effects of reduced permeability [122]. It has been reported that when the concrete pH falls below about 11.5, a porous oxide layer (rust) can form on the reinforcing steel due to corrosion [123]. More recent research indicates that the corrosion threshold is considered to be reached once the pH is reduced to 9.5 and that there is a steep decrease in the electrochemical corrosion potential indicating decomposition of the passive layer at a pH about 8 [124]. Carbon dioxide is a minor component of the atmosphere (~0.03% by volume). 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, temperature, and relative humidity of the ambient medium (i.e., carbonation occurs at 50% to 75% relative humidity; however, relative humidity values outside this range are capable of preventing carbonation). The rate of carbonation at exposed surfaces is considered to be roughly proportional to the square root of time for concrete kept continuously dry at normal relative humidities [125]. Carbonation generally proceeds in concrete as a front, beyond which the concrete is unaffected and behind which the pH is reduced. Carbonation penetrates more quickly near the corners where reinforcement usually comes closest to the surface, and into concrete where it is cracked and along reinforcement where it is locally unbonded [126]. Carbonation may be accelerated because the concrete is porous (i.e., poor quality) or because microcracks are present. If significant amounts of chloride are also present in the concrete, then it is to be expected that the corrosive action on embedded steel will be further enhanced by carbonation of the concrete. It has been shown that corrosion caused by carbonation increases with increasing chloride ion concentration provided that the carbonation rate itself was not retarded by the presence of chlorides [127]. This occurs because carbonation can result in decomposition of the complex hydrated chloride salts formed by the reaction of chloride with cement components, liberating more chloride into solution [125]. In NPPs, carbonation is most likely to occur at inside concrete surfaces exposed to relatively low humidities and elevated temperatures [128]. The extent of carbonation can be determined by treating a freshly exposed concrete surface with phenolphthalein (a pH indicator) [129]. More precise methods for determining carbonation depth include petrography (microscope) and using X-ray diffraction and differential thermal analysis techniques to analyze drilled powder samples obtained from various depths [122].

The most common cause of initiation of corrosion of steel in concrete is the presence of chloride ions, which can destroy the passive iron oxide film on the steel reinforcement even at high alkalinites (pH >11.5). For example, a pH of 13.2 and a concentration of more than 8000 ppm of free chloride ions are required to induce corrosion; however, at a pH of 11.6, only a concentration of about 71 ppm is required [128]. The mechanism through which the gamma Fe_2O_3 film is destroyed is not fully understood in that either the chloride ions convert the insoluble iron oxide to soluble iron chloride or they become included in the oxide layer in a manner that makes it permeable to air [80]. Chloride penetration also introduces a source of variation in concentration along the steel, forming concentration cells. Chloride ions are attracted to anodic regions of the steel to increase the local concentration. Increased acidity in the region of the anodic sites also can lead to local dissolution of the cement paste.

Cracks resulting from such causes as direct loading of the structure, or due to chemical or physical causes, can allow the rapid penetration of carbon dioxide or chloride ions to the steel reinforcement, thereby causing local failure of the passive oxide film. In tests where cracked reinforced concrete beams were exposed to a marine environment, it was found that corrosion was somewhat accelerated in the regions of flexural cracks; however, longitudinal cracking produced by corrosion dominates corrosion occurrence with the initiation and growth of the longitudinal cracks controlled by the restraining action of transverse reinforcement [130]. Once the longitudinal cracking exceeded a critical length, the corrosion rate accelerated. This may lead to concentration of corrosion over a small area, resulting in pitting corrosion, and can be of concern as it may lead to reduction in bar cross section. The volume of corrosion products may be so small that no external signs appear. It has been indicated that the presence of the crack is more important than its width, particularly when in the tension zone of a loaded beam (i.e., the crack width influences the speed at which corrosion begins, but because this period is short, the influence is limited and the width has only an infinitesimal effect on the spread of corrosion) [131]. Diffusion of chlorides can occur in sound concrete and can proceed through the capillary pore structure of the cement paste. Thus cracks in the concrete are not a prerequisite for transporting chlorides to the reinforcing steel. The rate of diffusion is strongly dependent on a number of factors (e.g., water-cementitious material ratio, type cement, temperature, and maturity of concrete). Some of the chlorides react chemically with cement components (e.g., calcium aluminates) and are effectively removed from the pore solution. The fraction of total chlorides available in the pore solution to cause a breakdown of the passive film is a function of a number of parameters [e.g., C_3A and C_4AF content, pH, and source of chlorides (mix or environment)]. A distinction needs to be made between chlorides added during the mix and those acquired by diffusing into the concrete from the environment. Added chlorides can combine with C_3A and ferric compounds in cement to give Friedel's salt, whereas chlorides resulting from diffusion cannot. Chlorides from diffusion therefore are potentially more hazardous [132]. The threshold value of chloride concentration below which significant corrosion does not occur is also dependent on these parameters. Different organizations have proposed various values: BS 8110 [133] and European Standard ENV 206 [134] (0.4% Cl^- by mass cement) and ACI 318 (0.15% water soluble Cl^- by mass cement) [4]. Investigators have reported minimum threshold values for chloride ion contents to initiate corrosion in the range of 0.026% to 0.033% (approximately 0.6 to 0.8 kg/m^3 or 0.04 to 0.05 lb/ft^3) total chloride ion content by mass of concrete [120]. Investigators have reported threshold acid-soluble chloride contents in the range from 0.15% to 1.0% to initiate steel corrosion.

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. Oxygen availability at cathodic sites is essential for corrosion to occur. In some instances where the oxygen supply is limited at active anodes, the resulting corrosion products may be green, white, or black in color [135]. The green product probably is a chloride complex, while the black product is magnetite (Fe_3O_4). Corrosion under oxygen-deficient conditions such as this is considered to be more serious than haematite ($Fe_2O_3 \cdot 3\ H_2O$), or normal red-brown rust, since it may go on some time before any evidence is visible.

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. Corrosion of reinforced concrete structures can be visible in the form of rust spots, cracks in the concrete cover along the line of rebars, and spalling. Generally, because the corrosion is fairly uniform, cracking of the cover concrete in normally reinforced structures usually occurs prior to a particular structural cross section becoming excessively weak, thus giving visual warning of the deterioration [136]. Occasionally, however, cover spalling occurs before any visible sign of deterioration at the concrete surface is apparent. Structural strength and serviceability are only reduced and jeopardized when corrosion of reinforcement causes a significant loss of steel cross section and/or there occurs a loss of bond between the steel and concrete [137]. In addition to cracking and spalling, corrosion will result in a reduction in effective steel cross section (i.e., load capacity), a decrease in ductility, and loss of composite interaction between concrete and steel due to bond deterioration. Figure 4.11 provides the interrelationship between chloride diffusion, depassivation, corrosion, and fracture.

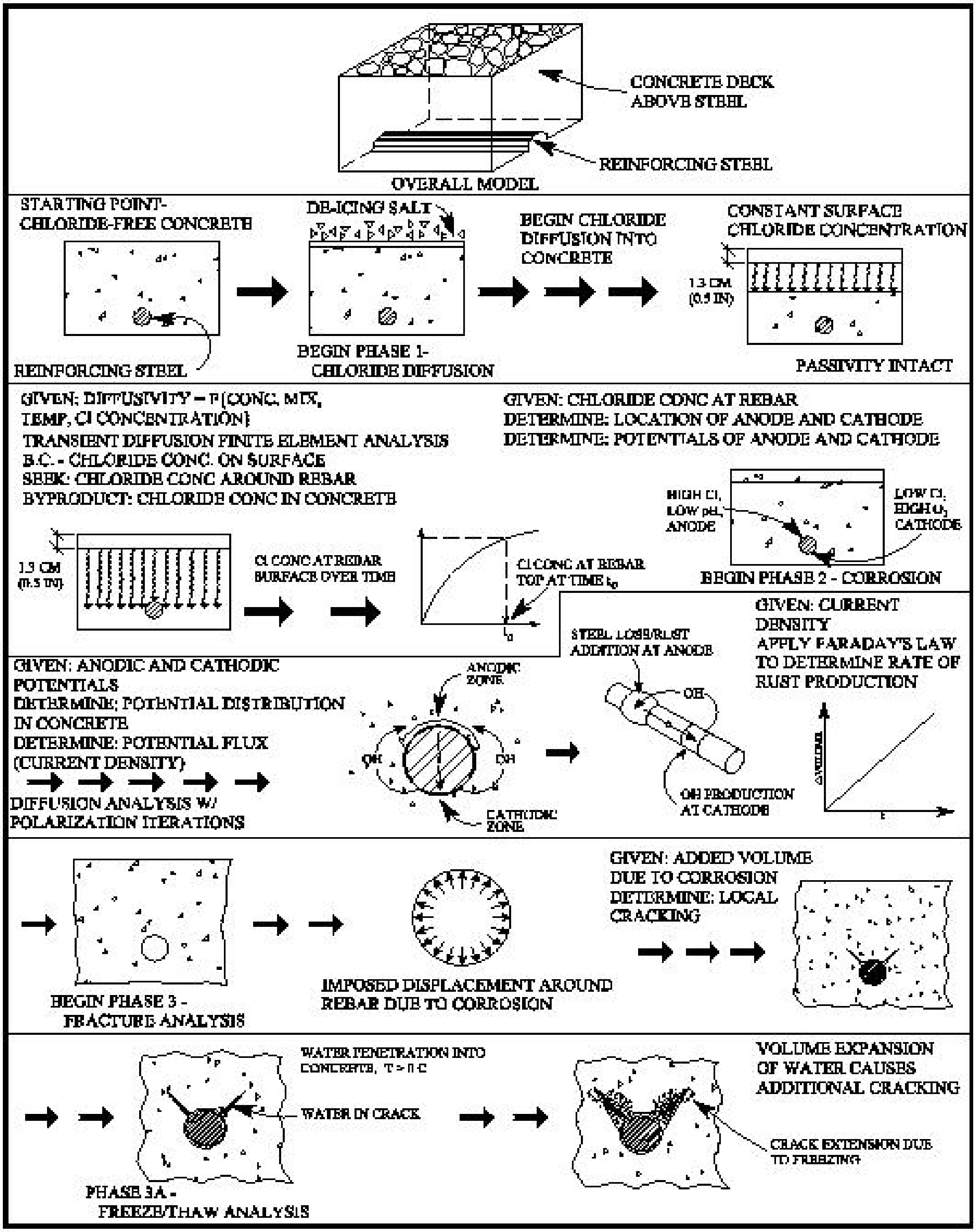

Figure 4.11. Interrelationship between chloride diffusion, depassivation, corrosion, and fracture [143].
Authorized reprint from *ACI Materials Journal* 96(2), 173–180, 1999.

4.3.2.2 Elevated Temperature

The properties of mild steel reinforcement of most importance to design are the yield stress and modulus of elasticity. Almost all of the information available on elevated-temperature effects addresses the residual strength of reinforcing bars after fire exposure and is somewhat controversial. One source reports that the mechanical properties of steels that have been heat-treated are largely unaffected by heating and normal cooling as long as the maximum temperature does not exceed 704 °C (1300 °F) [138]. Another reference indicates that temperatures up to 500 °C (932 °F) do not degrade the yield stress or ultimate strength of hot-rolled bars, but 700 °C (1292 °F) causes significant reductions in both (e.g., yield stress may be reduced by 50%) [139]. Tests in which a number of ASTM A615 Grade 60 12 mm (0.04 ft) diam reinforcing bars were heated to temperatures up to 802 °C (1476 °F), held at temperature for 1 h, and then permitted to slowly cool to room temperature indicate that the general nature of the stress-strain curve does not change in that all test results exhibited sharply defined yield points followed by a yield plateau followed by strain hardening [140]. Reductions in both the yield and ultimate stress were observed at temperatures above 500 °C (932 °F) with the largest reductions being 27% [749 °C (1380 °F)] and 17% [700 °C (1292 °F)], respectively. At 700 °C (1292 °F) elongations (203 mm or 0.66 ft gage length) increased about 40%. Data for German reinforcing steels indicate that for temperatures up to ~200 °C (392 °F) the yield strength is reduced by 10% or less, and at 500 °C (932 °F), it falls to about 50% its reference room temperature value [55]. Hot-rolled steels tend to resist the effects of temperature better than cold drawn or twisted steel. With cold-worked steel, the work-hardening effect that increases the strength of the reinforcement under normal exposure conditions suffers regression if exposed to high temperatures [e.g., >400 °C (752 °F)] [141]. With temperatures lower than 400 °C (752 °F) a residual hardening due to aging may be observed. The steel modulus of elasticity exhibits similar reductions with increasing temperature to that of the yield stress. Other data confirm the effects of temperatures above 200 °C (392 °F) on the mild steel reinforcing as well as providing a threshold temperature of about 300 °C (572 °F) for loss-of-bond properties with the concrete [142].

4.3.2.3 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} n/cm^2 has been cited for alteration of reinforcing steel mechanical properties [144]. 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 or reactor pressure vessel supports over an extended operating period [75].

4.3.2.4 Fatigue

Fatigue of the mild reinforcing system 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. For extreme conditions, the strength of the mild steel reinforcing system 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 [145]. Bond behavior (e.g., load-slip behavior) is influenced by high- and low-cycle fatigue [146].

Because of the typically low normal stress levels in reinforcing steel elements in NPP safety-related concrete structures, fatigue failure is not likely to occur.

4.3.3 Post-Tensioning Systems

The post-tensioning systems used in NPPs are designed to have (1) consistently high strength and strain at failure, (2) serviceability throughout their lifetime, (3) reliable and safe prestressing procedures, and (4) ability to be retensioned and replaced (nongrouted systems). Potential causes of degradation of the post-tensioning 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 aging management perspective.

4.3.3.1 Corrosion

Corrosion of prestressing systems can be highly localized or uniform. Most prestressing corrosion-related failures involving general civil engineering structures have been the result of localized 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 [147–149]. Failure of post-tensioning systems can also occur as a result of microbiologically induced corrosion. Due to the stress state in the post-tensioning systems, the tolerance for corrosion attack is much less than for the mild steel reinforcement.

4.3.3.2 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 (752 °F), however, may not harm the prestressing wire's mechanical properties [150]. Results of a Belgian study involving 30 types of prestressing steel indicate that thermal exposures up to ~200 °C (392 °F) do not significantly reduce (<10%) the tensile strength of prestressing wires or strands [55]. References [151] and [152] support results of the Belgian study.

Elevated-temperature exposure also affects the relaxation and creep properties of prestressing tendons. Reference [153] indicates that losses in a 15.2 mm diam strand initially stressed to 75% of its guaranteed ultimate tensile strength at 40 °C (104 °F) 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 [154]. Creep (length change under constant stress) of stress-relieved wire is negligible up to 50% its tensile strength. Also, the creep effect in prestressing steel varies with the steel's chemical composition as well as with the mechanical and thermal treatment applied during the manufacturing process. As temperature levels experienced by the prestressing tendons in LWR facilities are below 200 °C (392 °F), the possibility for thermal damage to the prestressing steels under normal operating conditions is low.

4.3.3.3 Irradiation

Irradiation of post-tensioning 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 [144]. Results obtained from studies in which 2.5 mm (0.008 ft) diam prestressing wires were stressed to 70% of their tensile strength and irradiated to a total dose of 4×10^{16} n/cm^2 (flux of 2×10^{10} n/cm^{2}s) showed that for exposures up to this level, the relaxation behavior of irradiated and unirradiated materials was similar. These flux levels are higher than the level likely to be experienced in a LWR containment vessel.

4.3.3.4 Fatigue

Repeated reversals of stress, or variations in stress, applied to concrete structural elements (beams in particular) can result in fatigue failure in any of the following modes [155]:

- failure of the concrete due to flexural compression;
- failure of the concrete due to diagonal tension or shear;
- failure of the prestressing steel due to flexural, tensile-stress variations;
- failure of pre-tensioned beams (grouted tendons) due to loss of bond stress; and
- failure of the end anchorages of post-tensioned structures.

The majority of fatigue failures that occurred while testing prestressed concrete beams have resulted from fatigue of the tendons due to stress concentrations that occur in the tendon at a location where a crack occurs. 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. Reference [8] presents high-cycle and low-cycle dynamic tensile test requirements for prestressing tendon systems used in concrete containments.

4.3.3.5 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 end slip), (3) elastic shortening, (4) tendon relaxation, and (5) concrete creep and shrinkage [156–158]. Of these factors, tendon relaxation and concrete creep and shrinkage have the largest contribution and, as time-dependent factors, are aging related.

Stress relaxation, defined as loss of stress (force) in the 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. Guidelines for developing surveillance programs acceptable to the NRC and for providing reasonable assurance (when properly implemented) that the structural integrity of the containment is being maintained are provided in a Regulatory Guides [159]. Reference [160] is a companion to Reference [159] and provides clarification with respect to determination of prestressing forces and prediction of prestressing force losses over the service life of the structure. Additional information on inspection of post-tensioning systems is available in Subsection IWL of Section XI of the ASME Boiler and Pressure Vessel Code [161].

4.3.4 Liner Plate and Structural Steel

Table 4.6 identifies examples of interior and exterior containment-related surface areas that could experience accelerated degradation or aging. Liner and structural steel members are subjected to the same general degradation mechanisms as the steel reinforcement. Of these, corrosion and fatigue are of most importance to aging management. Except for structural steel members that assist in providing support for the reactor pressure vessels in certain plants (e.g., Turkey Point), these members are generally not subjected to the effects of elevated temperature or irradiation.

Table 4.6. Examples of containment-related surface areas that could experience accelerated degradation or aging [13, 15, 162–164]

Environmental or operating conditions	Typical areas	Likely locations
Areas subject to accelerated corrosion with no or minimal corrosion allowance or Areas where the absence or repeated loss of protective coatings has resulted in substantial corrosion or pitting	Areas exposed to standing water Areas exposed to repeated wetting and drying Areas where persistent leakage has occurred Areas subject to microbiological attack Areas with geometries that permit water accumulation	Penetration sleeves and bellows Surfaces wetted during refueling Concrete-to-steel shell or liner interface Shell regions embedded in concrete including areas shielded by diaphragm floors Leak-chase channels Drain areas including sand pocket regions Sump liners Interior surfaces of BWR Mark I and Mark II suppression pools Exterior surfaces of BWR Mark I and Mark II drywells Emergency core cooling system suction intake at the bottom of BWR suppression pool Dissimilar metal welds
Areas subject to excessive wear from abrasion or erosion	Areas where mechanical wear, abrasion, or erosion cause a loss of protective coatings, deformations, or material loss Areas that experience frequent vibration	Surfaces subject to substantial traffic Sliding pads or supports (baseplates of BWR Mark I suppression chamber support columns) Pins or clevises Shear lugs Seismic restraints Surfaces exposed to water jets from testing operations Safety relief valve discharge areas BWR drywell head, vent system supports, and downcomers and bracing Personnel airlocks, equipment hatches, and control rod drive hatches

Steel component degradation can be classified as either material or physical damage. Material damage occurs when the microstructure of the metal is modified, causing changes in its mechanical properties. Degradation mechanisms that can potentially cause material damage to containment-related steels include (1) low-temperature exposure, (2) high-temperature exposure, (3) intergranular corrosion, (4) dealloying corrosion, (5) hydrogen embrittlement, and (6) neutron irradiation. Material damage to the liner and structural steel components from any of these sources is not considered likely, however. This is covered in more detail in EMDA Volume 2: Piping and Core Internals.

Physical damage occurs when the geometry of a component is altered by the formation of cracks, fissures, or voids, or when its dimensions change because of overload, buckling, corrosion, erosion, or formation of other types of surface flaws. Changes in component geometry, such as wall thinning or pitting, resulting from physical damage, can affect structural capacity by reducing the net section available to resist applied loads, or can compromise the component's leak-tight integrity if pits completely penetrate the component section.

4.3.4.1 Corrosion

Primary physical degradation mechanisms that can cause damage to steel components include

- general corrosion (atmospheric, aqueous, galvanic, stray-electrical current, and general biological);
- localized corrosion (filiform, crevice pitting, and localized biological);
- mechanically assisted degradation (erosion, fretting, cavitation, corrosion fatigue, surface flaws, arc strikes, and overload conditions);
- environmentally induced cracking (stress corrosion and hydrogen-induced); and
- fatigue.

Tables 4.7–4.10 provide descriptions of physical degradation mechanisms and associated commentary for general corrosion, localized corrosion, mechanically assisted degradation, and environmentally induced cracking, respectively. Fatigue is addressed in the next section. Material degradation caused by general corrosion or pitting corrosion represents the greatest threat to the metallic components.

Table 4.7. Types of general corrosion that can cause physical damage to metallic components [171]

Type of general corrosion	Description/commentary
Atmospheric corrosion	Atmospheric corrosion is the gradual degradation or alteration of a material by contact with substances such as oxygen, carbon dioxide, water vapor, and sulfur and chlorine compounds that are present in the atmosphere [165]. In buildings, corrosion of uncoated structural steel elements often occurs when warm, moist air from inside the building permeates through the walls and ceilings and condenses on the surfaces of colder steel beams, columns, and fasteners. In NPPs, atmospheric corrosion can be suspected whenever uncoated carbon and low-alloy steel components are exposed to air with a relative humidity that exceeds about 70%.
Aqueous corrosion	Corrosion of metals in aqueous environments occurs when two or more electrochemical reactions take place on the surface, causing the metal or alloy to change from a metallic state to a nonmetallic state [165]. A common example of this phenomenon is rusting in which metal (iron) is converted to a nonmetallic corrosion product (rust). Physical damage to carbon and low-alloy steel fasteners and pressure-retaining components of the primary system has also occurred at a number of PWR plants due to boric acid (H_3BO_3) attack [172, 173]. Similar types of physical damage caused by aqueous corrosion is possible whenever carbon and low-alloy steel components are in contact with pure, treated, contaminated, or sea water.
Galvanic corrosion	Galvanic corrosion is accelerated degradation that occurs when a metal or alloy is electrically coupled to a more noble metal in the same electrolyte [165]. The three requirements for galvanic corrosion are (1) materials possessing different surface potentials, (2) a common electrolyte, and (3) a common electrical path. Physical damage to welds and base metal in NPPs caused by galvanic corrosion can occur in locations where dissimilar metals are in contact and an electrolytic solution is present.
Stray-current corrosion	Stray-current corrosion is degradation resulting from direct current flow through paths other than the intended circuit [165]. There are many sources of stray electrical current, but cathodic protection systems, high-voltage direct current systems, and direct current welding operations are potentially the most significant for nuclear power plant structures [174]. Damage caused by alternating current is less than that caused by direct current and decreases in severity as the frequency increases.
General biological corrosion	General biological corrosion, or microbiological-induced corrosion (MIC) as it is sometimes called, is deterioration of metal as a result of the metabolic activity of microorganisms [175]. A continuous film of bacteria, algae, or slime can contribute to general biological corrosion, but the formation of a continuous film over a large area is rare. Consequently, biological corrosion is usually localized because microorganisms tend to settle in discrete colonies rather than uniformly over the surface of a material.

Note: References correspond to entries in the reference list.

Table 4.8. Types of localized corrosion that can cause physical damage to metallic components [171]

Type of localized corrosion	Description/commentary
Filiform corrosion	Filiform corrosion occurs under thin organic coatings in the form of randomly distributed threadlike filaments that appear as worm-like blemishes [165]. The source of initiation is usually a cut, nick, pore, scratch, or disruption in the coating. Because filiform corrosion produces very localized distress on metal surfaces, its effects on containment metallic components is not expected to be significant. However, coating damage may become a maintenance concern.
Crevice corrosion	Crevice corrosion is localized attack of a metal surface adjacent to an area that is shielded from full exposure to the environment because of close proximity between the metal and the surface of another material [165]. Narrow openings or spaces between metal-to-metal or nonmetal-to-metal components, cracks, seams, or other surface flaws can serve as sites for corrosion initiation. Stainless steels are more prone to crevice corrosion than carbon steels particularly in the presence of chloride solutions, but containment metallic components made from all types of steels can experience this type of localized damage when exposed to aqueous environments.
Pitting corrosion	Pitting corrosion is degradation of a metal surface confined to a point or small area that takes the form of cavities [165]. Pitting is one of the most common types of localized corrosion encountered in aqueous environments. The propagation of pits is thought to involve the dissolution of metal and the maintenance of a high degree of acidity at the bottom of the pit by the hydrolysis of the dissolved metal ions making the rate at which pitting corrosion progresses very unpredictable. Pitting corrosion of containment metallic components made from all types of steels is possible whenever they are exposed to aqueous environments.
Localized biological corrosion	Localized biological corrosion, like general biological corrosion, is deterioration of metal as a result of the metabolic activity of microorganisms [165]. Because microorganisms tend to settle in discrete colonies rather than uniformly over the surface of a material, most biologically induced corrosion tends to be localized. Localized damage to carbon, low-alloy, and even stainless steel components caused by biological corrosion is possible when environmental conditions are favorable for microorganisms.

Note: References correspond to entries in the reference list.

Table 4.9. Types of mechanically assisted corrosion that can cause physical damage to metallic components [171]

Type of mechanically assisted degradation	Description/Commentary
Erosion	Erosion is the destruction and removal of a material by the abrasive action of moving fluids. Material loss can be accelerated by the presence of solid particles or matter in suspension. Except in the torus region of BWR Mark I metal containments, moving water is not usually present inside containment vessels. Therefore, erosion of containment metallic components is not expected to be a generic concern [170].
Fretting	Fretting is wear that occurs between tight-fitting surfaces subjected to cyclic motion of extremely small amplitude. Steels used to construct containment metallic components are not routinely subjected to wear caused by fretting because these components generally remain static.
Cavitation	Cavitation is loss of material caused by a liquid that experiences rapid and intense pressure changes. This action is generally associated with pumps and piping systems and is therefore not considered to be a significant containment metallic component degradation mechanism.
Corrosion Fatigue	Corrosion fatigue is the process in which a metal fractures prematurely under conditions of simultaneous corrosion and repeated cyclic loading at lower stress levels or fewer cycles than would be required in the absence of the corrosive environment [165]. Corrosion fatigue of pressure-retaining components is not expected to be a generic concern [170].
Surface Flaws	Surface flaws such as notches, cracks, gouges, grooves, dents, and tool marks can be created during routine operations, in-service maintenance, repair activities, or equipment failures that generate missiles or pipe whips. Stress concentrations caused by these types of flaws located in critical regions of the containment can affect performance by contributing to premature structural failure at loads well below those permitted in design or loss of leak-tight integrity. Surface flaws located in aqueous environments can also serve as initiation sites for filiform and crevice corrosion
Arc Strikes	Arc strikes are another type of surface flaw that can cause loss of ductility in mild and low-alloy steels, hardening of higher carbon and alloy steels, or localized cracking in higher strength, hardenable grades of steel [176]. Arc strikes represent any localized heat-affected zone or change in the surface contour of the finished weld or base metal caused by an arc produced by the passage of electrical current from welding electrodes, magnetic particle inspection electrodes, electrical shorts, or other sources.
Overload Conditions	Equipment failures, excessive piping loads, and movements caused by thermal expansion or contraction due to extreme-temperature exposure are examples of over-load conditions that can permanently deform, bulge, stretch, bend, buckle, or neck pressure-retaining components. These changes in component geometry can be attributed to plastic deformation of the metal or brittle fracture of the component. The damage caused by overload conditions can have a detrimental effect on containment structural capacity and leak-tight integrity and adversely affect its ability to perform its intended function.

Note: References correspond to entries in the reference list.

Table 4.10. Types of environmentally induced degradation mechanisms that can cause physical damage to metallic components [171]

Type of degradation mechanism	Description/Commentary
Stress-Corrosion Cracking	Stress-corrosion cracking is a material degradation process that requires the simultaneous action of a corrodent and sustained tensile stress to initiate and propagate cracks in metals and alloys. Carbon, low-alloy, and stainless steels exposed to various aqueous environments are susceptible to stress-corrosion cracking. Aqueous solutions that may be present at NPPs and could contribute to stress-corrosion cracking include groundwater containing chlorides and sulfates, borated water in PWRs, sodium pentaborate in BWRs, and certain types of decontamination fluids [170].
Hydrogen Damage	Hydrogen damage most often results from the combined action of hydrogen and residual or applied tensile stress [170]. It manifests itself in many ways, such as cracking, blistering, hydride formation, and loss of tensile ductility. Physical damage to containment metallic components by hydrogen is not expected to be a serious problem because neither hydrogen at high pressures nor hydrogen at high temperatures is found inside the containment vessel.

Note: References correspond to entries in the reference list.

Typically, the liner plate and any installed steel are 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). Depending on the component, a corrosion allowance may also have been provided during the design stage. However, little allowance will have been provided for the relatively thin liner plate (i.e., ~6.3 mm or 0.2 ft thick).

The corrosion process that affects these components is similar to that for conventional reinforcing steel. For liner plates, the influence of local attack that can lead to loss of leak tightness of the liner plate 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. The rate of attack may be rapid, depending on the aggressiveness of the environment. Reference [165] contains corrosion data for structural steel in numerous environments. For an industrial environment, the atmospheric (general) corrosion rate was found to be 0.02 to 0.04 mm/year (6.6×10^{-5} to 1.3×10^{-4} ft/year). This same reference reported pitting rates of 0.056 mm/year (1.8×10^{-4} ft/year) for low-carbon steels placed in polluted seawater. In general, depending on the environmental parameters, surface corrosion rates were noted to range from 0.001 to 0.03 mm/year (3.3×10^{-6} to 9.8×10^{-5} ft/year).

Corrosion of structural steel piles, used in certain containment configurations 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 localized corrosion resulting in significant loss of cross-sectional area. One study examined corrosion data from 43 piling installations of varying depths (up to 41.5 m or 136 ft) with times of exposure ranging from 7 to 50 years in a wide variety of conditions [166]. 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 or salt spray may be somewhat affected [167].

4.3.4.2 Fatigue

Fatigue is the progressive, localized, and permanent structural change that occurs in a material subjected to repeated or fluctuating strains at normal stresses that have maximum values less than the tensile strength of the material [168]. Guidance for performing fatigue evaluations is provided in Section III, Division 1 of the ASME Code [169]. Although design rules for fatigue of steels used to construct containment metallic components are included in the ASME Code, fatigue resulting from such things as startup and shutdown cycles, crane loads, and leakage-rate tests is not expected to be a generic concern for containment metallic components (i.e., liner) [170]. However, the effects of conditions outside of design predictions and local stress intensification points (e.g., material flaws) may result in fatigue-related problems. With respect to the liner plate, possible fatigue sites include base metal delaminations, weld defects, arc strikes, shape changes near penetrations, structural attachments, and concrete floor interfaces. For structural steel members (liner attachments and anchorages), the locations most susceptible to fatigue include large containment penetration framing (e.g., hatches) and liner anchorages near vibrating load conditions (such as those generated in structural attachments). It is important to note that temperatures the rebar would experience, except under accident conditions, are not considered high enough to affect either rebar properties or bond to concrete and were not considered for that reason.

5. DISCUSSION OF PHENOMENA IDENTIFICATION AND RANKING TABLE (PIRT) EVALUATION RESULTS

The NPP structures and degradation modes to which the PIRT process was applied were provided previously in Figures 1.1 through 1.4. Cross-cutting issues associated with NPP containments were also provided previously in Figure 1.5. EMDA matrices for each structure were constructed as shown in Figures 5.1 through 5.5. Each matrix figure consists of two figures: the top, which is a smaller duplicate of the bottom, main figure, lists the degradation modes in numerical order. In the main figure, mean-value scores of the panel members for **Knowledge** and **Susceptibility** are plotted as the abscissa and the ordinate, respectively, in positions relative to their values (see the spreadsheets in Appendix A); mean-value scores are extracted from Appendix A and included in Table 5.1 for easy reference. The numerical value next to each symbol is the mean-value score of the panel members for **Confidence.** So far, this figure is similar to the original PMDA depiction procedure. However, to accommodate the fourth metric, namely **Structural significance** (also referenced as **Significance)**, the symbols were color-coded and assigned different sizes to reflect the severity of **Structural significance**. In this way it was possible to treat all four categories (metrics) in a two-dimensional plot.

For illustration, take Figure 5.1 as an example: **Freeze/Thaw** is degradation mode #10, from the upper image of Figure 5.1. It is plotted in the top right quadrant, with high **Susceptibility** and high **Knowledge**, shown in Table 5.1 as 2.4 and 2.5, respectively. The Confidence value is 2.3, as shown on the figure, and the symbol is green with medium size, reflecting medium Structural significance. The procedure is the same for Figures 5.2 through 5.5.

Other generic trends from these plots are worth noting. The left side of the plot with the lighter shading is indicative of low Knowledge, while the darker shading on the right side of the plot is indicative of high Knowledge. The corners of the plot indicate the high Knowledge, low Susceptibility; high Knowledge, high Susceptibility; and low Knowledge, high Susceptibility areas. Moving from upper right to lower left can be accomplished via additional research and development to understand and predict key forms of degradation. The different domains of these plots highlight key areas of concern, including:

- Low Knowledge, high Susceptibility degradation modes are indicated by the pink shading in Figure 1.1 and represent modes of degradation that could be detrimental to service with high Susceptibility scores (>2) and low Knowledge scores (<2). These scores indicate gaps in understanding for degradation modes that have been demonstrated in service. Low Knowledge and moderate Susceptibility also indicate gaps in knowledge, although with lower consequences. These scoring regions are useful in identifying potential knowledge gaps and areas requiring further research into mechanisms and underlying causes to predict occurrence.

- High Knowledge, high Susceptibility degradation modes are shown in red in Figure 5.1 and represent areas that could be detrimental to service with high Susceptibility scores (>2) and high Knowledge scores (>2). These modes of degradation are well understood and have likely been observed in service. While there may be some mechanistic understanding of the underlying causes, re-confirmation for extended service and research into mitigation or detection technologies may be warranted.

- High Knowledge, low Susceptibility degradation modes (dark green in Figure 5.1) are those that are relatively well understood and of low consequence to service with low Susceptibility scores (<1) and high Knowledge scores (>2). These modes of degradation are adequately understood and may been observed in service. Mitigation and maintenance can currently manage this form of degradation. Research on these modes of degradation is a lower priority.

Other combinations of Knowledge and Susceptibility are of course possible and fit between the cases listed above in terms of priority

It should be noted here that the EMDA applies a generic process to a wide variety of structural design, outdoor environment, and concrete mix design. The ranking resulting from this approach is intended to provide general trends but cannot pretend to cover local specificities.

The evaluation of the susceptibility index is based on the operating experience of various industries: nuclear, hydro, and transportation. The benefit of expanding the scope of the research to the mentioned sectors was to investigate the degradation modes of older concrete structures (like dams for instance) potentially subjected to a more aggressive environment (e.g., carbonation exposure in industrial and urban areas). Carefully transposing this operating experience to nuclear structures provided a helpful opportunity to extrapolate potential degradation modes. For the specific aging modes in NPPs (borated water attack and irradiation for instance) with limited background data and information, the impact could only be based on a brainstorming process. As a result, when operating experience exists, the confidence level among the panel is generally higher.

Each structural category is independently analyzed in the sequel to track the lack of knowledge, the most impactful degradations, and their susceptibilities. The general conclusions (Chapter 6) provide a general frame within which to analyze the results, for each category and combined, and thus give a ranking in terms of importance.

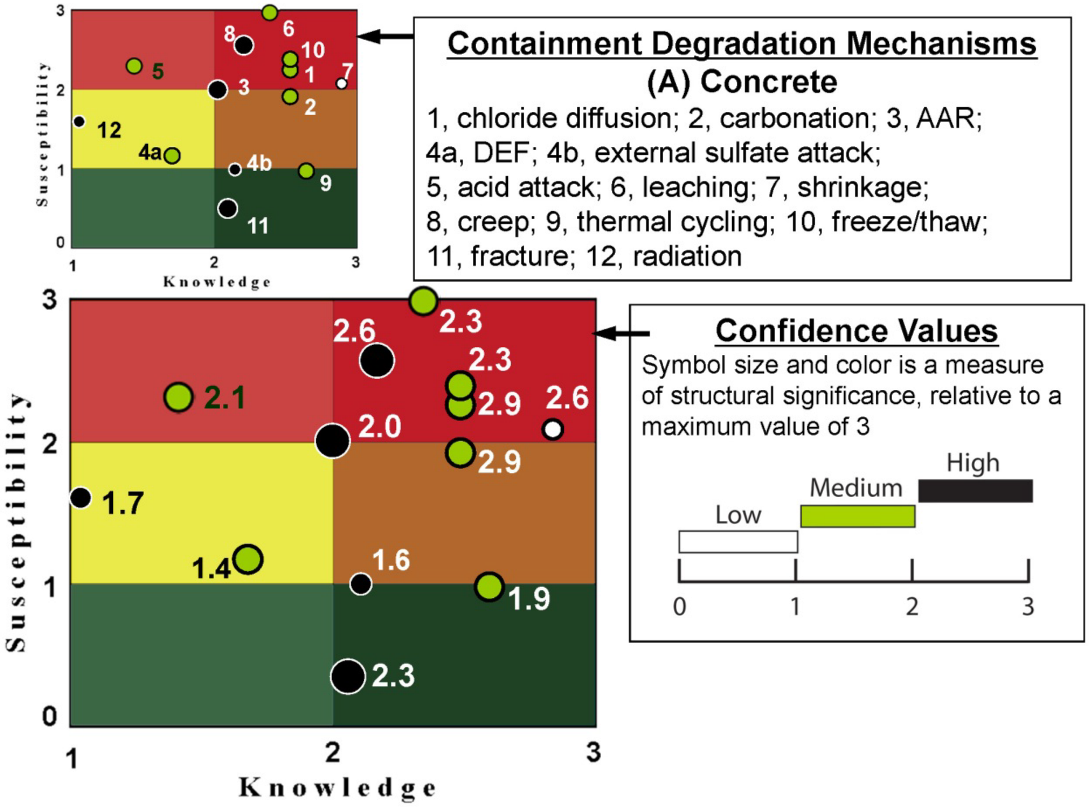

Figure 5.1. Average scores of the panel members for degradation mechanisms of concrete containments – (A) concrete.

Regarding the degradation modes relevant for the concrete containments, Figure 5.1 shows the following:

- ***Knowledge***: Irradiation damage of concrete (primarily in the reactor cavity area) and acid attack have been insufficiently studied though their effect is judged to be limited. These areas evidently need further research.

- ***Susceptibility***: Seven of the 13 identified mechanisms are ranked between 2 and 3, meaning that the knowledge is, for those mechanisms, between a strong basis for concern or known but limited plant occurrence or even demonstrated, and compelling evidence for occurrence, or multiple plant observations. The main reason is the fairly generic character of some degradation modes triggered by the outdoor environment (carbonation, freeze-thaw, chloride diffusion, etc.) and by in-service mechanical loading (creep). The moisture transport in thick concrete elements, that is, drying or wetting-drying cycles, can also be related to most degradation modes.

- ***Structural significance***: Four of the five mechanisms of high structural significance (i.e., creep, alkali-aggregate reaction, external sulfate attack, and fracture) are interestingly ranked with the same level of knowledge (around 2), showing that some reasonable basis to know dependencies qualitatively or semi-quantitatively from data or from extrapolation in similar structures exists but needs to be extended. The fifth mode ranked with a high structural significance index is irradiation damage because of its potential affect on the supporting function of the reactor vessel.

- ***Confidence***: The confidence level is higher than 2 for 9 of the 13 modes, showing a moderate to high confidence from the panel in its judgment of the degradation susceptibilities. It is noticeable that the confidence level for irradiation is rather low (1.7) though the structural significance is considered high, indicating a need for further studies.

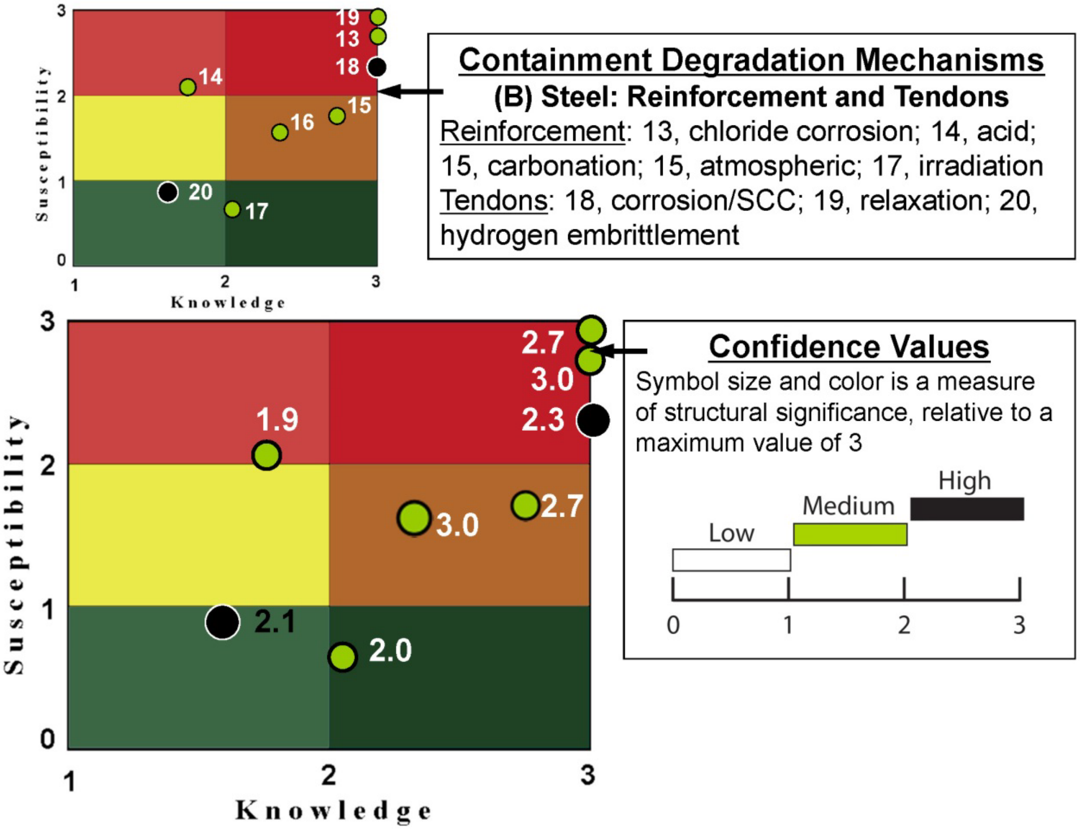

Figure 5.2. Average scores of the panel members for degradation mechanisms of concrete containments – (B) steel reinforcement and prestressing tendons.

Regarding the degradation modes relevant for the post-tensioning system and reinforcement of the containments, Figure 5.2 shows the following:

- ***Knowledge***: Whereas some modes are judged to be very well known and understood (chloride-induced corrosion of the reinforcement, corrosion/stress corrosion cracking, and relaxation of the tendons), two mechanisms appear to require further attention: effect of boric acid attack on the reinforcement and hydrogen embrittlement of multiple tendons, particularly with regard to its potential effect on the structural integrity of the concrete containment.

- ***Susceptibility***: The same modes ranked with a high level of knowledge have a high susceptibility index, indicating that limited research work is required in this direction. The acid attack of the reinforcement would benefit from additional research as it exhibits insufficient knowledge with regard to its rather high susceptibility.

- ***Structural significance***: Only two of the eight mechanisms studied show a structural significance index higher or equal to 2.0: the hydrogen embrittlement and corrosion/stress-corrosion cracking of the tendons leading to their potential failure.

- ***Confidence***: The confidence of the panel in its judgment of the susceptibility index is generally moderate to high.

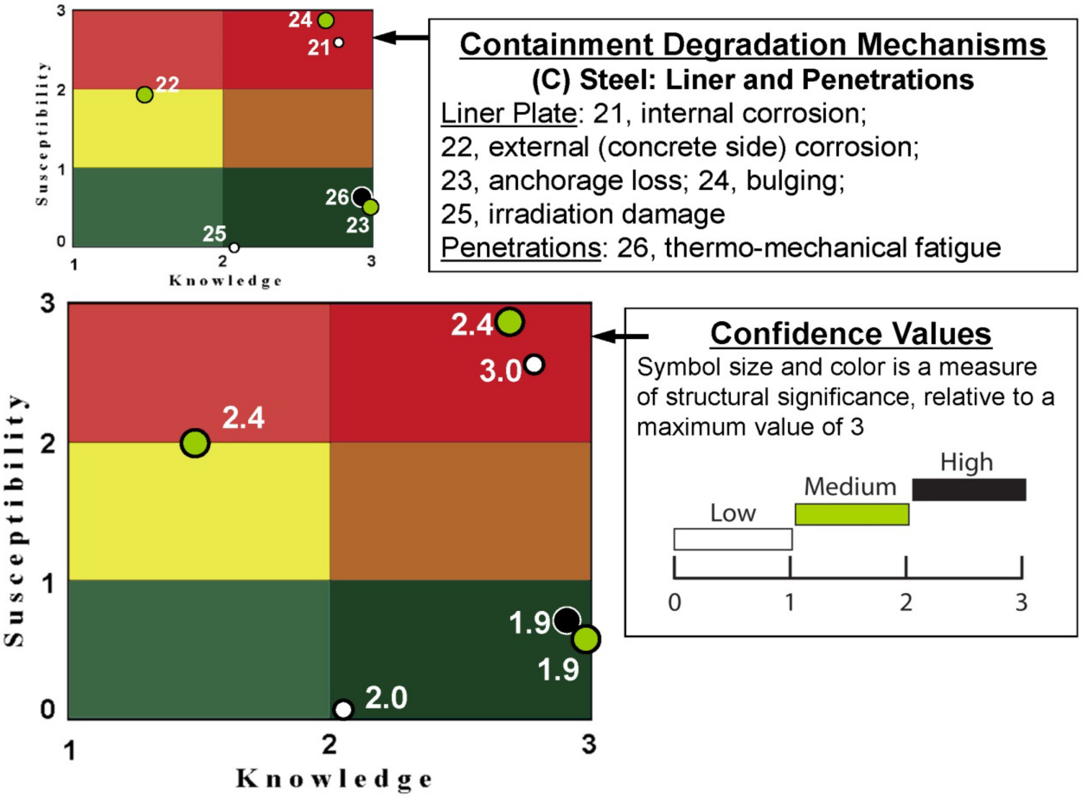

Figure 5.3. Average scores of the panel members for degradation mechanisms of concrete containments – (C) steel liner and penetrations.

Regarding the degradation modes relevant for the metallic liners and the steel penetrations of the containment, Figure 5.3 shows the following:

- ***Knowledge***: The knowledge index is generally fairly good (above 2), but the notable exception is the liner corrosion on the concrete side (i.e., the unaccessible side) observed as the result of the unlikely presence of organic/plastic materials left over during the construction. The principal gap of knowledge resulting is the lack of inspection capabilities to detect whether such objects are present or that local corrosion pitting is occurring.

- ***Susceptibility***: Three of the six mechanisms studied are judged rather unlikely (anchorage loss and irradiation damage of the liner and thermo-mechanical fatigue of the steel penetration), whereas two are considered with a high degree of susceptibility [corrosion of the liner (inside the reactor building) and bulging].

- ***Structural significance***: Only one of the six modes shows a structural significance factor qualified as high, as this type of degradation may generally only lead to local or superficial damage that would not significantly affect the liner function. The notable exception is the thermal-mechanical fatigue of the liner penetration.

- ***Confidence***: The confidence of the panel in its judgment of the susceptibility index is generally good, especially for the high-ranked susceptibility modes.

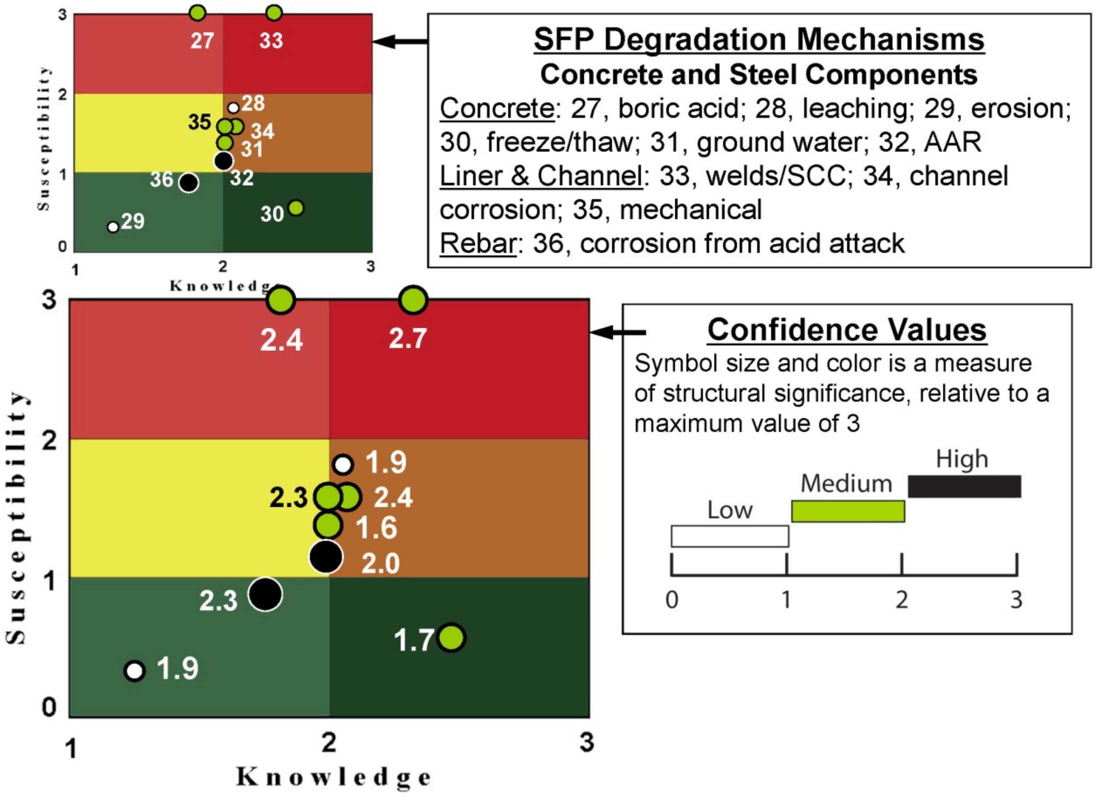

Figure 5.4. Average scores of the panel members for degradation mechanisms of spent fuel pool and transfer canal.

Regarding the degradation modes relevant for the spent fuel pools and transfer canals, Figure 5.4 shows:

- ***Knowledge***: Two modes received a grade significantly higher than 2.0 (stress-corrosion-cracking of the liner welds, freeze-thaw of the exposed concrete in PWR designs). The mechanism of erosion of the concrete resulting from a “flowing” leak is noticeably not well understood. The other five mechanisms are ranked around 2.0.

- ***Susceptibility***: The most susceptible degradation modes are related to the potential leakage of boric acid caused by weld cracking of the liner and the collection channels, and the degradation of concrete resulting from the leakage.

- ***Structural significance***: The two most significant modes are the risk of alkali-aggregate reaction of the spent fuel pool concrete walls and basemat and the corrosion of the reinforcement caused by acid attack, these mechanisms potentially affecting the structural integrity of the pool under accidental loading.

- ***Confidence***: The confidence of the panel in its judgment of the susceptibility index is generally fairly good.

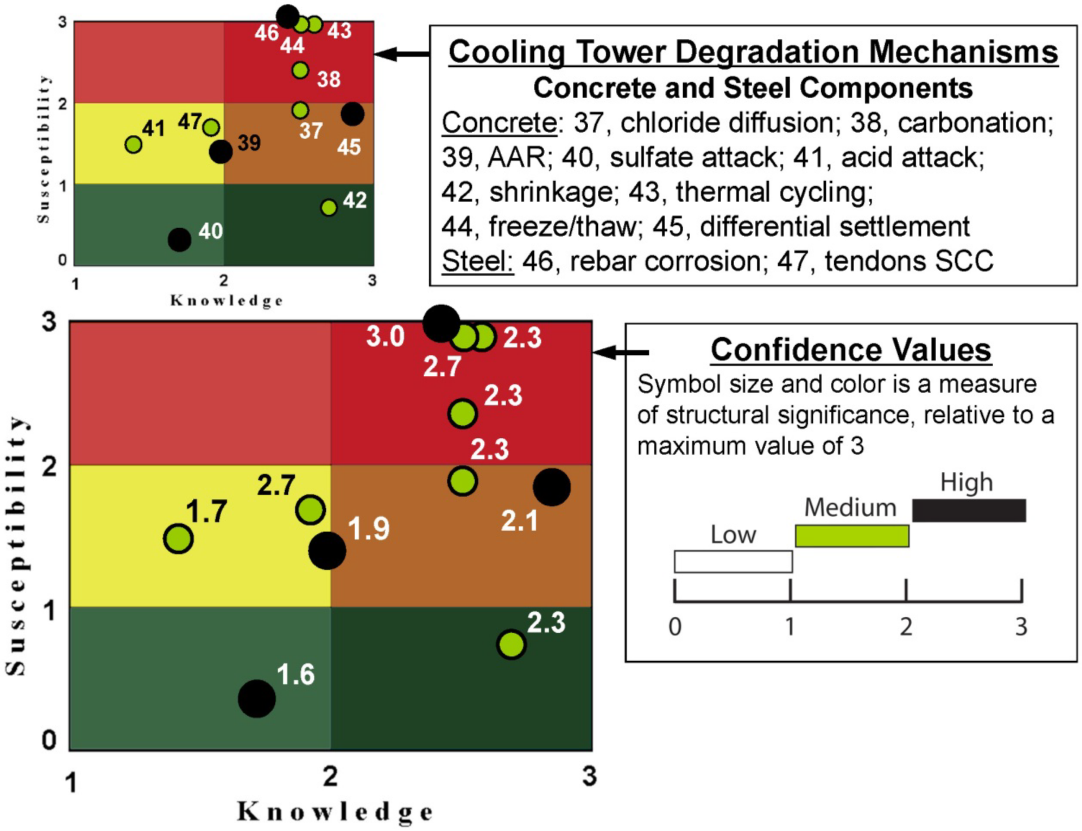

Figure 5.5. Average scores of the panel members for degradation mechanisms of cooling tower.

Regarding the degradation modes relevant for the cooling towers, Figure 5.5 shows the following:

- ***Knowledge***: Two of the 12 degradations modes studied show a knowledge index significantly lower than 2.0, indicating a limited understanding or a lack of comprehensive data: acid attack of the concrete resulting from the cooling water treatment and sulfate attack (groundwater or DEF).

- ***Susceptibility***: Several modes are judged with a rather high susceptibility: corrosion of the reinforcement most likely resulting from carbonation, but also from chloride attack (water treatment and use of deicing salts), and degradation caused by environmental cycling (freeze-thaw and thermal loading).

- ***Structural significance***: Four modes have a significant significance to the structural integrity of the towers (corrosion of the reinforcement, differential settlement, alkali-silica reaction, and internal sulfate attack). It is remarkable that they have various degrees of knowledge and susceptibility though.

- ***Confidence***: The confidence of the panel in its judgment of the susceptibility index is generally fairly good (>2), but there are three exceptions (alkali-aggregate reaction, acid attack, and sulfate attack, especially with regard to the possible delayed ettringite formation).

Table 5.1. Mean-value scores of degradation modes (extracted from spreadsheet in Appendix A)

					Indices	Mean			
Description						Succeptibility	Knowledge	Confidence	Significance
Containment Concrete	Chemical	Chloride Diffusion			31%	2.3	2.5	2.9	1.3
		Carbonation			22%	1.9	2.5	2.9	1.1
		AAR			67%	2.0	2.0	2.0	2.6
		Sulfate Attack	Delayed Ettringite Formation		35%	1.1	1.7	1.4	2.0
			External Sulfate Attack		26%	1.0	2.1	1.6	2.1
		Acid Attack, Borated Water			68%	2.3	1.4	2.1	1.6
		Leaching			52%	3.0	2.4	2.3	1.6
	Physical	Time Dependent	Shrinkage		20%	2.1	2.8	2.6	1.0
			Creep		67%	2.6	2.2	2.6	2.1
		Temperature	Thermal Cycles		16%	1.0	2.6	1.9	1.6
			Freeze/Thaw		41%	2.4	2.5	2.3	1.6
		Mechanical	Fracture	LCF/ILRT	16%	0.4	2.1	2.3	3.0
	Radiation				100%	1.6	1.1	1.7	2.6
Containment-Steel Components	Mild Steel Reinforcement	Corrosion	Aqueous	Chloride	41%	2.7	3.0	3.0	1.7
				Carbonation	23%	1.7	2.8	2.7	1.4
				Boric Acid	48%	2.1	1.7	1.9	1.4
			Atmospheric		34%	1.6	2.4	3.0	2.0
		Irradiation			9%	0.6	2.1	2.0	1.3
	Tendons	Corrosion/SCC			43%	2.3	3.0	2.3	2.1
		Relaxation			39%	2.9	3.0	2.7	1.6
		Hydrogen Embrittlement of multiple tendons			31%	0.9	1.6	2.0	2.1
	Liner	Corrosion	Internal		26%	2.7	2.8	3.0	1.0
			External concrete side		59%	2.0	1.5	2.4	1.7
		Anchorage Loss			7%	0.6	3.0	1.9	1.4
		Bulging			32%	2.9	2.7	2.4	1.1
		Irradiation			2%	0.1	2.1	2.0	1.0
	Liner Penetration	Thermal/Mechanical Fatigue			15%	0.7	2.9	1.9	2.3
SFP and Transfer Canal	Concrete	Boric Acid (PWR)			62%	3.0	1.8	2.4	1.4
		Flowing Water	Leaching (Deionized Water BWR)		23%	1.9	2.1	1.9	1.0
			Accelerated Erosion		6%	0.3	1.3	1.9	1.0
		Freeze Thaw (PWR)			8%	0.6	2.5	1.7	1.3
		Aggressive Ground Water (Below ground pool)			30%	1.4	2.0	1.6	1.6
		AAR/DEF			41%	1.1	2.0	2.0	2.7
	Steel Component	Liner & Channel	Welds; SCC		51%	3.0	2.4	2.7	1.6
			Channel Corrosion		39%	1.7	2.1	2.4	1.9
			Strain Induced by Concrete Erosion		39%	1.7	2.0	2.3	1.7
		Rebar	Corrosion from acid attack		30%	0.9	1.7	2.3	2.3
Cooling Tower	Concrete	Chloride Diffusion			33%	1.9	2.5	2.3	1.7
		Carbonation			41%	2.3	2.5	2.3	1.7
		AAR			46%	1.3	2.0	1.9	2.7
		Sulfate Attack	Delayed Ettringite Formation		11%	0.3	1.7	1.6	2.6
		Acid Attack			39%	1.4	1.4	1.7	1.4
		Time Dependent	Shrinkage		6%	0.6	2.8	2.3	1.1
		Temperature	Thermal Cycles		37%	2.9	2.6	2.3	1.3
			Freeze/Thaw		52%	2.9	2.5	2.7	1.7
		Differential Settlement			39%	1.9	2.9	2.1	2.3
	Reinforcement	Corrosion			79%	3.0	2.4	3.0	2.4
	Pretension Tendon	SCC/Corrosion			48%	1.7	1.9	2.7	2.0

Indices = (Susceptibility × Significance)/Knowledge, normalized. Higher indices provide an indication of the importance of a particular degradation mode.

6. CONCLUSIONS AND RECOMMENDATIONS

Each of the key structural categories was independently analyzed to track the level of knowledge, the significance of the most impactful degradations, and their susceptibilities. The following mechanisms emerged as most important in each category:

- Irradiation for containment concrete emerged as the most important degradation mechanism, mainly driven by insufficient data to improve the level of knowledge about the effects of irradiation on concrete mechanical properties. Alkali-aggregate reaction (AAR), acid attack and creep emerged as secondarily important mechanisms. The biggest surprise in this analysis is the result that susceptibility to fracture emerged as the least important mechanism; this should be interpreted to apply only to concrete cracking of the generally known type which is accounted for in the structural design. There are special forms of concrete damage that potentially evolve with time into discrete fracture under special circumstances involving creep-cracking interaction induced by structural modification or change in loading. These do not qualify as a general aging mechanism, and are addressed separately below.

- Concerning containment steel structures, the corrosion of the liner plate on the concrete side emerged as the mechanism with the highest level of importance, primarily because of being inaccessible. This is followed by the corrosion of reinforcement by chlorides and boric acid, and by the SCC of the pre-stressing tendons. Irradiation effects on steel components, including the liner, emerged as the least important, primarily because of the accumulated low neutron dose levels.

- Concerning the SFP and transfer canal, boric acid attack on concrete in PWRs emerged as the mechanism of highest importance. This is closely followed by SCC of welds in the liner plate and channels. Considering the available field experience, this mechanism was scored highly due to the prevalence of data (high knowledge), and should be considered as important as the boric acid attack.

- Finally, concerning the cooling tower, the corrosion of reinforcement emerged as the most important aging mechanism followed by several mechanisms, which include freeze/thaw, AAR and SCC of prestress tendons in precast elements in the Mechanical Draft cooling tower design. It is important to point out that corrosion of reinforcement, which, while not safety related, is highly important to operate the nuclear power plant economically.

Based on the rankings of important degradation mechanisms for the respective categories of concrete and civil structures, potential knowledge gaps for assessing the integrity of concrete structures for operation up to 80 years were identified as follows.

- The panel identified creep of the post-tensioned concrete containment as a potential knowledge gap. Creep is a long-term process associated with sustained loading and moisture transport that affects the internal stress state and, because it adds to tendon relaxation in causing gradual loss of prestress, which is usually restored by periodic re-tensioning thereby introducing a form of cyclic activation of primary creep, can potentially damage the concrete and lead to tertiary degradation (creep-fracture interaction) under accidental loading.

- Related to the creep mode identified above is the interaction between creep and cracking in post-tensioned containments subjected to repair involving prestress modification during the operational life of the containment. While concrete cracking is a well understood behavior characteristic of concrete structures in general, and is accounted for in the usual manner in the structural design of reinforced containments, it plays a unique role, (usually unaccounted for in design), in post-tensioned containments. Depending upon the position of the tendons relative to the surface of the containment wall, radially-oriented dilation damage, eventually leading to discrete split cracking, can form on a lamellar surface parallel to the wall surface, which evolves with time as a creep-cracking interaction mechanism. This mode of cracking can potentially occur during initial pre-stressing, during re-tensioning to repair loss of prestress due to concrete creep and tendon relaxation, or during de-tensioning and re-tensioning operations which may be undertaken as part of life extension re-construction work. This type of split cracking can be controlled by radial reinforcement, which generally is not part of the initial design, and because such cracking configuration is internal and is not visible on the surface, it can potentially evolve into an undetectable degradation mode.

- The panel also identified the irradiation of concrete as a knowledge gap. This, as mentioned above, is due to a lack of sufficient test data to support a clear evaluation of the significance of such mechanism for long-term operations. As a reminder here, the term “concrete containment” is used in a generic sense to describe any concrete part within the containment building. Irradiation mainly affects the reactor cavity and the biological shield.

- Alkali-silica reactions were also noted by the expert panel. Though this degradation is well documented by the operating experience (for bridges and dams in particular) and scientific literature, its high ranking in the EMDA analysis describes the need to assess its potential consequences on the structural integrity of the containment, considering the recent operating experience at Davis Besse and other plants.

- The next mechanism and potential gap is related to boric acid attack of concrete in the spent fuel pool. The knowledge gaps are essentially related to the kinetics and the extent of the attack (role of the concrete mix design) and their consequences on the structural integrity.

Finally, the panel identified two possible knowledge gaps when assessing the integrity of containment steel components for operation up to 80 years:

- Corrosion and SCC of the tendons, and

- Corrosion of the inaccessible side of the liner. The lack of knowledge here is associated with the absence of a current in-service inspection technique.

These degradation modes and mechanisms have been identified as having the greatest potential effect on preserving the ability of the concrete and civil structures and components to fulfill their safety related functions during long-term NPP operation. This potential effect may be mitigated by improving the overall level of knowledge about the identified degradation modes in order to better predict and mitigate possible consequences; and/or, by identifying and implementing acceptable mitigation strategies (replacement, treatments, etc.). Research will be required in either case and these topics were identified as having the highest priorities for research for concrete and civil structures and components.

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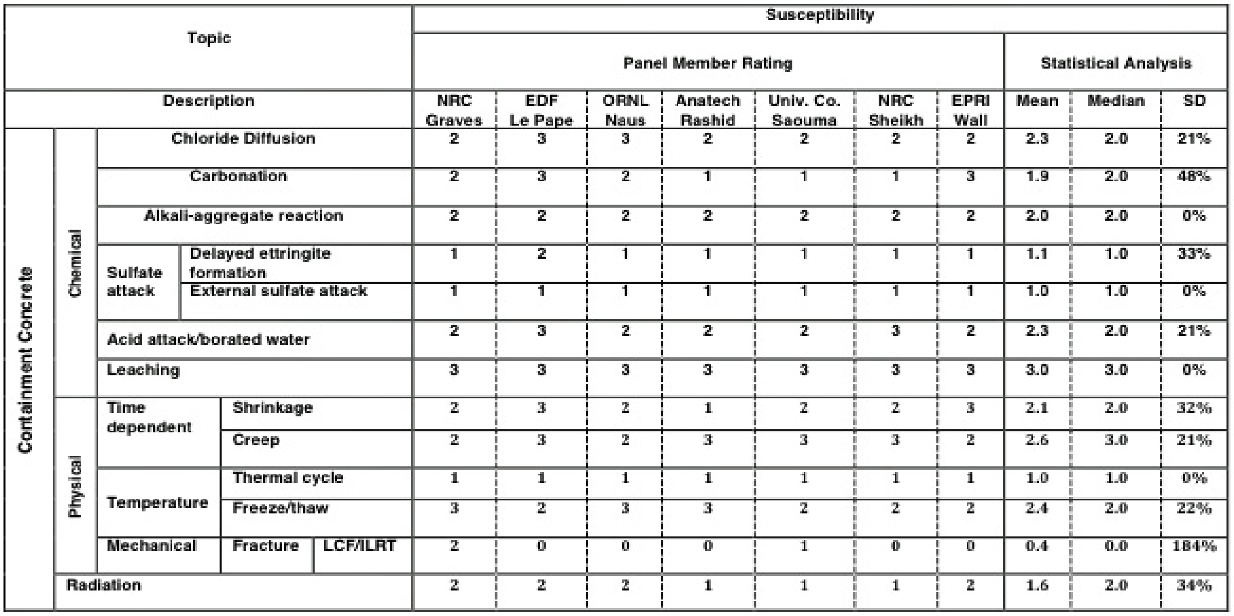
APPENDIX A

SUMMARY OF ASSESSMENT RESULTS FOR EACH COMBINATION OF COMPONENT AND DEGRADATION MODE ADDRESSED

APPENDIX A – SUMMARY OF ASSESSMENT RESULTS FOR EACH COMBINATION OF COMPONENT AND DEGRADATION MODE ADDRESSED

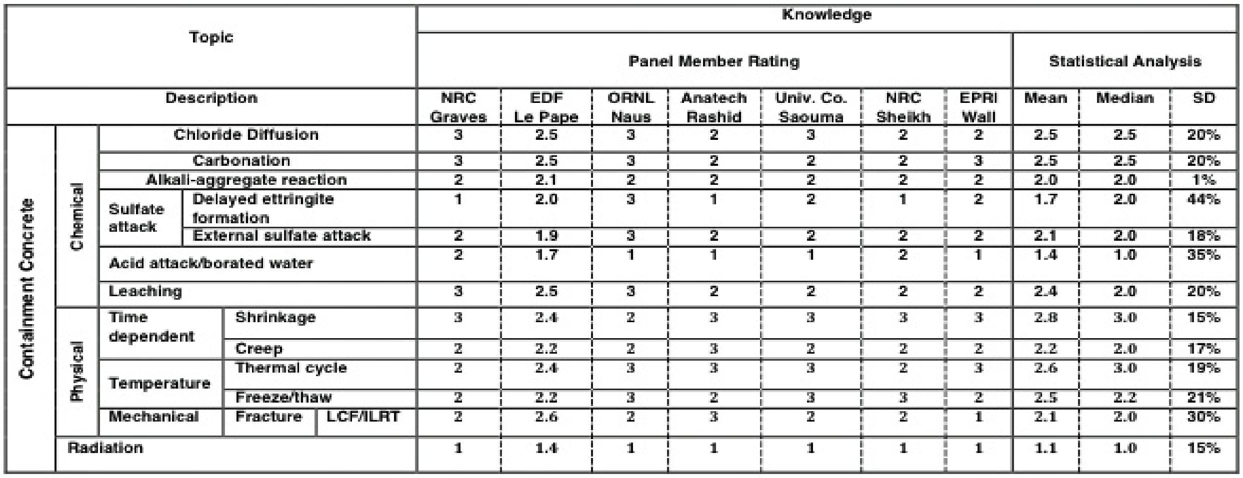
A.1 Containment Concrete

Table A.1.1. Containment Concrete – Degradation Mode Susceptibility Rankings



Topic				Susceptibility									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Containment Concrete	Chemical	Chloride Diffusion		2	3	3	2	2	2	2	2.3	2.0	21%
		Carbonation		2	3	2	1	1	1	3	1.9	2.0	48%
		Alkali-aggregate reaction		2	2	2	2	2	2	2	2.0	2.0	0%
		Sulfate attack	Delayed ettringite formation	1	2	1	1	1	1	1	1.1	1.0	33%
			External sulfate attack	1	1	1	1	1	1	1	1.0	1.0	0%
		Acid attack/borated water		2	3	2	2	2	3	2	2.3	2.0	21%
		Leaching		3	3	3	3	3	3	3	3.0	3.0	0%
	Physical	Time dependent	Shrinkage	2	3	2	1	2	2	3	2.1	2.0	32%
			Creep	2	3	2	3	3	3	2	2.6	3.0	21%
		Temperature	Thermal cycle	1	1	1	1	1	1	1	1.0	1.0	0%
			Freeze/thaw	3	2	3	3	2	2	2	2.4	2.0	22%
		Mechanical	Fracture LCF/ILRT	2	0	0	0	1	0	0	0.4	0.0	184%
	Radiation			2	2	2	1	1	1	2	1.6	2.0	34%

Table A.1.2. Containment Concrete – Degradation Mode Knowledge Rankings



Topic				Knowledge									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Containment Concrete	Chemical	Chloride Diffusion		3	2.5	3	2	3	2	2	2.5	2.5	20%
		Carbonation		3	2.5	3	2	2	2	3	2.5	2.5	20%
		Alkali-aggregate reaction		2	2.1	2	2	2	2	2	2.0	2.0	1%
		Sulfate attack	Delayed ettringite formation	1	2.0	3	1	2	1	2	1.7	2.0	44%
			External sulfate attack	2	1.9	3	2	2	2	2	2.1	2.0	18%
		Acid attack/borated water		2	1.7	1	1	1	2	1	1.4	1.0	35%
		Leaching		3	2.5	3	2	2	2	2	2.4	2.0	20%
	Physical	Time dependent	Shrinkage	3	2.4	2	3	3	3	3	2.8	3.0	15%
			Creep	2	2.2	2	3	2	2	2	2.2	2.0	17%
		Temperature	Thermal cycle	2	2.4	3	3	3	2	3	2.6	3.0	19%
			Freeze/thaw	2	2.2	3	2	3	3	2	2.5	2.2	21%
		Mechanical	Fracture LCF/ILRT	2	2.6	2	3	2	2	1	2.1	2.0	30%
	Radiation			1	1.4	1	1	1	1	1	1.1	1.0	15%

Table A.1.3. Containment Concrete – Degradation Mode Confidence Rankings

Topic					Confidence									
					Panel Member Rating							Statistical Analysis		
Description					NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Containment Concrete	Chemical	Chloride Diffusion			2	3	3	3	3	3	3	2.9	3.0	13%
		Carbonation			2	3	3	3	3	3	3	2.9	3.0	13%
		Alkali-aggregate reaction			2	2	2	2	3	1	2	2.0	2.0	29%
		Sulfate attack	Delayed ettringite formation		2	2	1	1	2	1	1	1.4	1.0	37%
			External sulfate attack		2	1	2	1	2	1	2	1.6	2.0	34%
		Acid attack/borated water			2	2	2	3	2	2	2	2.1	2.0	18%
		Leaching			2	3	2	2	1	3	3	2.3	2.0	33%
	Physical	Time dependent	Shrinkage		3	3	1	3	3	2	3	2.6	3.0	31%
			Creep		3	3	1	3	3	2	3	2.6	3.0	31%
		Temperature	Thermal cycle		3	1	2	2	2	2	1	1.9	2.0	37%
			Freeze/thaw		3	2	2	2	2	2	3	2.3	2.0	21%
		Mechanical	Fracture	LCF/ILRT	3	3	1	3	3	1	2	2.3	3.0	42%
	Radiation				3	1	2	3	1	1	1	1.7	1.0	55%

Table A.1.4. Containment Concrete – Degradation Mode Structural Significance Rankings

Topic					Structural Significance									
					Panel Member Rating							Statistical Analysis		
Description					NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Containment Concrete	Chemical	Chloride Diffusion			1	2	2	1	1	1	1	1.3	1.0	38%
		Carbonation			1	1	2	1	1	1	1	1.1	1.0	33%
		Alkali-aggregate reaction			3	2	3	2	3	3	2	2.6	3.0	21%
		Sulfate attack	Delayed ettringite formation		1	3	3	2	1	2	2	2.0	2.0	41%
			External sulfate attack		2	3	2	2	2	2	2	2.1	2.0	18%
		Acid attack/borated water			2	2	2	1	1	2	1	1.6	2.0	34%
		Leaching			2	1	2	1	1	2	2	1.6	2.0	34%
	Physical	Time dependent	Shrinkage		1	1	1	1	1	1	1	1.0	1.0	0%
			Creep		2	2	3	2	2	2	2	2.1	2.0	18%
		Temperature	Thermal cycle		2	1	2	1	1	2	2	1.6	2.0	34%
			Freeze/thaw		2	2	2	1	1	2	1	1.6	2.0	34%
		Mechanical	Fracture	LCF/ILRT	3	3	3	3	3	3	3	3.0	3.0	0%
	Radiation				2	3	3	2	3	2	3	2.6	3.0	21%

A.2 Containment Steel Components

Table A.2.1 Containment Steel Components – Degradation Mode Susceptibility Rankings

Topic					Susceptibility									
					Panel Member Rating							Statistical Analysis		
Description					NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Containment Steel Components	Mild Steel Reinforcement	Corrosion	Aqueous	Chloride	3	3	3	1	3	3	3	2.7	3.0	28%
				Carbonation	2	3	2	1	1	2	1	1.7	2.0	44%
				Boric Acid	2	2	2	2	2	3	2	2.1	2.0	18%
			Atmospheric		2	2	2	1	1	1	2	1.6	2.0	34%
		Irradiation			1	2	1	0	0	0	0	0.6	0.0	138%
	Tendons	Corrosion/SCC			2	2	3	2	2	2	3	2.3	2.0	21%
		Relaxation			3	2	3	3	3	3	3	2.9	3.0	13%
		Hydrogen embrittlement – multiple tendons			2	1	1	0	0	1	1	0.9	1.0	81%
	Liner	Corrosion	Internal		3	3	3	2	3	3	2	2.7	3.0	18%
			External – concrete side		2	2	2	2	2	2	2	2.0	2.0	0%
		Anchorage loss			1	1	1	0	0	0	1	0.6	1.0	94%
		Bulging			3	2	3	3	3	3	3	2.9	3.0	13%
		Irradiation			0	1	0	0	0	0	0	0.1	0.0	265%
	Liner penetration	Thermal/mechanical fatigue			1	1	1	0	0	1	1	0.7	1.0	68%

Table A.2.2. Containment Steel Components – Degradation Mode Knowledge Rankings

Topic					Knowledge									
					Panel Member Rating							Statistical Analysis		
Description					NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Containment Steel Components	Mild Steel Reinforcement	Corrosion	Aqueous	Chloride	3	2.9	3	3	3	3	3	3.0	3.0	2%
				Carbonation	3	2.5	3	2	3	3	3	2.8	3.0	14%
				Boric Acid	2	1.8	1	2	2	2	1	1.7	2.0	28%
			Atmospheric		2	3.0	3	2	3	2	2	2.4	2.0	22%
		Irradiation			2	2.0	3	3	2	1	2	2.1	2.0	32%
	Tendons	Corrosion/SCC			3	3.0	3	3	3	3	3	3.0	3.0	0%
		Relaxation			3	3.0	3	3	3	3	3	3.0	3.0	0%
		Hydrogen embrittlement – multiple tendons			1	2.0	3	1	1	1	2	1.6	1.0	50%
	Liner	Corrosion	Internal		3	2.3	3	2	3	3	3	2.8	3.0	15%
			External – concrete side		2	1.8	1	2	2	1	1	1.5	1.8	33%
		Anchorage loss			3	3.0	3	3	3	3	3	3.0	3.0	0%
		Bulging			3	2.0	2	3	3	3	3	2.7	3.0	18%
		Irradiation			2	2.0	3	3	2	1	2	2.1	2.0	32%
	Liner penetration	Thermal/mechanical fatigue			3	2.0	3	3	3	3	3	2.9	3.0	13%

Table A.2.3. Containment Steel Components – Degradation Mode Confidence Rankings

Topic					Confidence									
					Panel Member Rating							Statistical Analysis		
Description					NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Containment Steel Components	Mild Steel Reinforcement	Corrosion	Aqueous	Chloride	3	3	3	3	3	3	3	3.0	3.0	0%
				Carbonation	3	3	3	3	3	3	1	2.7	3.0	28%
				Boric Acid	3	1	2	2	1	2	2	1.9	2.0	37%
			Atmospheric		3	-	3	3	3	3	3	3.0	3.0	0%
		Irradiation			3	1	3	3	1	2	1	2.0	2.0	50%
	Tendons	Corrosion/SCC			3	1	2	3	3	1	3	2.3	3.0	42%
		Relaxation			3	3	2	3	3	2	3	2.7	3.0	18%
		Hydrogen embrittlement – multiple tendons			3	2	2	2	1	2	2	2.0	2.0	29%
	Liner	Corrosion	Internal		3	3	3	3	3	3	3	3.0	3.0	0%
			External – concrete side		2	2	3	3	3	1	3	2.4	3.0	32%
		Anchorage loss			2	1	2	2	2	2	2	1.9	2.0	20%
		Bulging			2	3	2	3	2	2	3	2.4	2.0	22%
		Irradiation			2	2	3	3	1	2	1	2.0	2.0	41%
	Liner penetration	Thermal/mechanical fatigue			2	2	1	2	2	2	2	1.9	2.0	20%

Table A.2.4. Containment Steel Components – Degradation Mode Structural Significance Rankings

Topic					Structural Significance									
					Panel Member Rating							Statistical Analysis		
Description					NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Containment Steel Components	Mild Steel Reinforcement	Corrosion	Aqueous	Chloride	2	2	2	2	1	2	1	1.7	2.0	28%
				Carbonation	1	1	2	2	1	2	1	1.4	1.0	37%
				Boric Acid	1	1	2	2	1	2	1	1.4	1.0	37%
			Atmospheric		2	2	3	2	1	2	2	2.0	2.0	29%
		Irradiation			1	1	1	1	1	1	3	1.3	1.0	59%
	Tendons	Corrosion/SCC			3	3	3	1	1	3	1	2.1	3.0	50%
		Relaxation			2	1	2	1	2	2	1	1.6	2.0	34%
		Hydrogen embrittlement – multiple tendons			2	2	3	2	2	2	2	2.1	2.0	18%
	Liner	Corrosion	Internal		1	1	1	1	1	1	1	1.0	1.0	0%
			External – concrete side		2	2	1	1	2	2	2	1.7	2.0	28%
		Anchorage loss			1	2	2	1	1	2	1	1.4	1.0	37%
		Bulging			1	1	2	1	1	1	1	1.1	1.0	33%
		Irradiation			1	1	1	1	1	1	1	1.0	1.0	0%
	Liner penetration	Thermal/mechanical fatigue			2	2	2	2	3	2	3	2.3	2.0	21%

A.3 Spent Fuel Pool and Transfer Canal

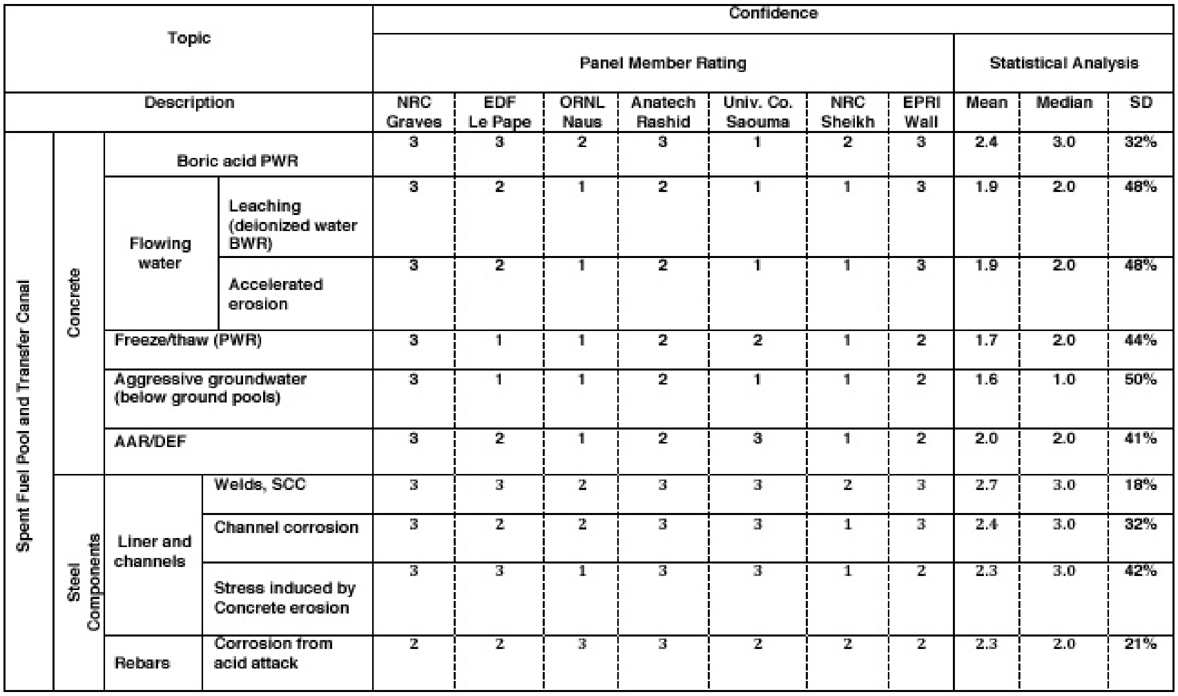
Table A.3.1. Spent Fuel Pool and Transfer Canal – Degradation Mode Susceptibility Rankings

Topic				Susceptibility									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Spent Fuel Pool and Transfer Canal	Concrete	Boric acid PWR		3	3	3	3	3	3	3	3.0	3.0	0%
		Flowing water	Leaching (deionized water BWR)	2	2	1	2	2	2	2	1.9	2.0	20%
			Accelerated erosion	0	1	0	0	0	0	1	0.3	0.0	171%
		Freeze/thaw (PWR)		1	1	0	0	0	1	1	0.6	1.0	94%
		Aggressive groundwater (below ground pools)		2	2	1	1	1	1	2	1.4	1.0	37%
		AAR/DEF		1	2	1	0	1	1	2	1.1	1.0	60%
	Steel Components	Liner and channels	Welds, SCC	3	3	3	3	3	3	3	3.0	3.0	0%
			Channel corrosion	2	1	1	2	1	3	2	1.7	2.0	44%
			Stress induced by Concrete erosion	2	2	1	2	1	2	2	1.7	2.0	28%
		Rebars	Corrosion from acid attack	1	1	1	0	0	1	2	0.9	1.0	81%

Table A.3.2. Spent Fuel Pool and Transfer Canal – Degradation Mode Knowledge Rankings

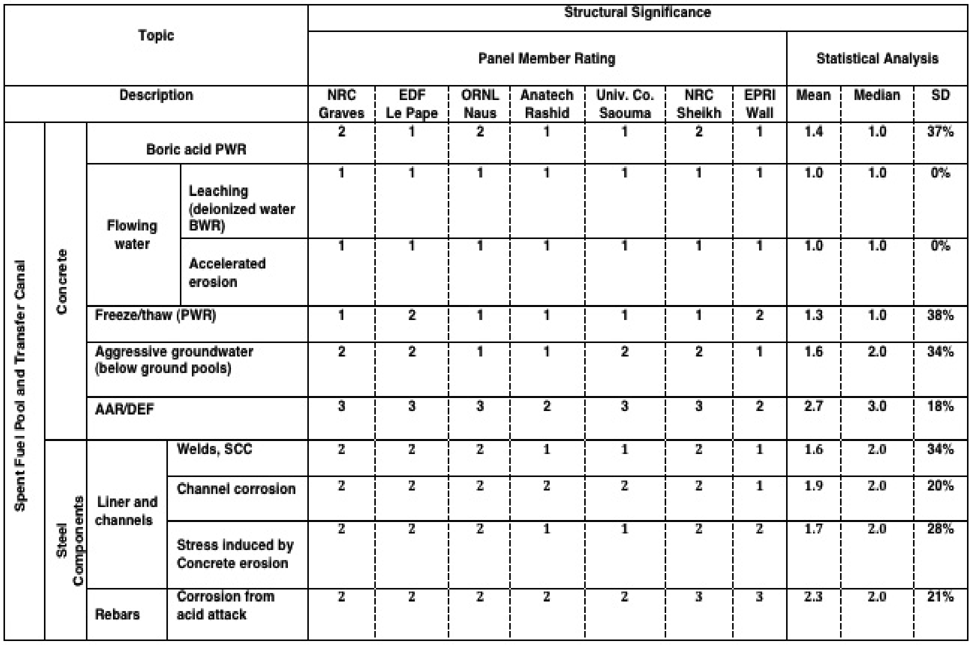
Topic				Knowledge									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Spent Fuel Pool and Transfer Canal	Concrete	Boric acid PWR		2	1.7	2	2	1	2	2	1.8	2.0	21%
		Flowing water	Leaching (deionized water BWR)	2	3.0	2	2	2	2	2	2.1	2.0	18%
			Accelerated erosion	1	1.0	2	1	1	1	2	1.3	1.0	38%
		Freeze/thaw (PWR)		2	2.2	3	2	3	3	2	2.5	2.2	21%
		Aggressive groundwater (below ground pools)		2	1.9	2	2	2	2	2	2.0	2.0	2%
		AAR/DEF		2	2.1	2	2	2	2	2	2.0	2.0	1%
	Steel Components	Liner and channels	Welds, SCC	2	2.0	3	3	3	2	2	2.4	2.0	22%
			Channel corrosion	2	2.0	3	2	2	2	2	2.1	2.0	18%
			Stress induced by Concrete erosion	2	2.0	2	2	2	2	2	2.0	2.0	0%
		Rebars	Corrosion from acid attack	2	2.0	2	1	2	2	1	1.7	2.0	28%

Table A.3.3. Spent Fuel Pool and Transfer Canal – Degradation Mode Confidence Rankings



Topic				Confidence									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Spent Fuel Pool and Transfer Canal	Concrete	Boric acid PWR		3	3	2	3	1	2	3	2.4	3.0	32%
		Flowing water	Leaching (deionized water BWR)	3	2	1	2	1	1	3	1.9	2.0	48%
			Accelerated erosion	3	2	1	2	1	1	3	1.9	2.0	48%
		Freeze/thaw (PWR)		3	1	1	2	2	1	2	1.7	2.0	44%
		Aggressive groundwater (below ground pools)		3	1	1	2	1	1	2	1.6	1.0	50%
		AAR/DEF		3	2	1	2	3	1	2	2.0	2.0	41%
	Steel Components	Liner and channels	Welds, SCC	3	3	2	3	3	2	3	2.7	3.0	18%
			Channel corrosion	3	2	2	3	3	1	3	2.4	3.0	32%
			Stress induced by Concrete erosion	3	3	1	3	3	1	2	2.3	3.0	42%
		Rebars	Corrosion from acid attack	2	2	3	3	2	2	2	2.3	2.0	21%

Table A.3.4. Spent Fuel Pool and Transfer Canal – Degradation Mode Structural Significance Rankings



Topic				Structural Significance									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Spent Fuel Pool and Transfer Canal	Concrete	Boric acid PWR		2	1	2	1	1	2	1	1.4	1.0	37%
		Flowing water	Leaching (deionized water BWR)	1	1	1	1	1	1	1	1.0	1.0	0%
			Accelerated erosion	1	1	1	1	1	1	1	1.0	1.0	0%
		Freeze/thaw (PWR)		1	2	1	1	1	1	2	1.3	1.0	38%
		Aggressive groundwater (below ground pools)		2	2	1	1	2	2	1	1.6	2.0	34%
		AAR/DEF		3	3	3	2	3	3	2	2.7	3.0	18%
	Steel Components	Liner and channels	Welds, SCC	2	2	2	1	1	2	1	1.6	2.0	34%
			Channel corrosion	2	2	2	2	2	2	1	1.9	2.0	20%
			Stress induced by Concrete erosion	2	2	2	1	1	2	2	1.7	2.0	28%
		Rebars	Corrosion from acid attack	2	2	2	2	2	3	3	2.3	2.0	21%

A.4 Cooling Towers

Table A.4.1. Cooling Towers – Degradation Mode Susceptibility Rankings

Topic				Susceptibility									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Cooling Towers	Concrete	Chloride diffusion		2	2	2	2	2	1	2	1.9	2.0	20%
		Carbonation		2	3	2	2	2	2	3	2.3	2.0	21%
		Alkali-aggregate reaction		1	2	2	1	1	1	1	1.3	1.0	38%
		Sulfate attack	Delayed ettringite formation	0	1	1	0	0	0	0	0.3	0.0	171%
		Acid attack		1	1	2	1	1	2	2	1.4	1.0	37%
		Time dependent	Shrinkage	0	3	1	0	0	0	0	0.6	0.0	198%
		Temperature	Thermal cycles	3	3	2	3	3	3	3	2.9	3.0	13%
			Freeze-thaw	3	2	3	3	3	3	3	2.9	3.0	13%
		Differential settlement		2	2	2	2	1	2	2	1.9	2.0	20%
	Steel Reinforcement	Corrosion		3	3	3	3	3	3	3	3.0	3.0	0%
	Prestressing Tendons	SCC/Corrosion		2	2	1	1	2	2	2	1.7	2.0	28%

Table A.4.2. Cooling Towers – Degradation Mode Knowledge Rankings

Topic				Knowledge									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Cooling Towers	Concrete	Chloride diffusion		3	2.5	3	2	3	2	2	2.5	2.5	20%
		Carbonation		3	2.5	3	2	2	2	3	2.5	2.5	20%
		Alkali-aggregate reaction		2	2.1	2	2	2	2	2	2.0	2.0	1%
		Sulfate attack	Delayed ettringite formation	1	2.0	3	1	2	1	2	1.7	2.0	44%
		Acid attack		2	1.7	1	1	1	2	1	1.4	1.0	35%
		Time dependent	Shrinkage	3	2.4	2	3	3	3	3	2.8	3.0	15%
		Temperature	Thermal cycles	2	2.4	3	3	3	2	3	2.6	3.0	19%
			Freeze-thaw	2	2.2	3	2	3	3	2	2.5	2.2	21%
		Differential settlement		3	3.0	3	3	3	3	2	2.9	3.0	13%
	Steel Reinforcement	Corrosion		2	3.0	3	2	3	2	2	2.4	2.0	22%
	Prestressing Tendons	SCC/Corrosion		2	2.0	3	2	2	1	1	1.9	2.0	37%

Table A.4.3. Cooling Towers – Degradation Mode Confidence Rankings

Topic				Confidence									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Cooling Towers	Concrete	Chloride diffusion		1	3	2	2	3	2	3	2.3	2.0	33%
		Carbonation		2	3	2	2	3	2	2	2.3	2.0	21%
		Alkali-aggregate reaction		2	3	1	1	3	1	2	1.9	2.0	48%
		Sulfate attack	Delayed ettringite formation	2	2	1	1	2	1	2	1.6	2.0	34%
		Acid attack		2	3	1	1	2	1	2	1.7	2.0	44%
		Time dependent	Shrinkage	2	3	1	2	3	2	3	2.3	2.0	33%
		Temperature	Thermal cycles	2	3	1	2	3	2	3	2.3	2.0	33%
			Freeze-thaw	3	3	2	3	3	2	3	2.7	3.0	18%
		Differential settlement		3	3	1	2	2	1	3	2.1	2.0	42%
	Steel Reinforcement	Corrosion		3	3	3	3	3	3	3	3.0	3.0	0%
	Prestressing Tendons	SCC/Corrosion		3	3	3	3	2	3	2	2.7	3.0	18%

Table A.4.4. Cooling Towers – Degradation Mode Structural Significance Rankings

Topic				Structural Significance									
				Panel Member Rating							Statistical Analysis		
Description				NRC Graves	EDF Le Pape	ORNL Naus	Anatech Rashid	Univ. Co. Saouma	NRC Sheikh	EPRI Wall	Mean	Median	SD
Cooling Towers	Concrete	Chloride diffusion		2	2	2	2	1	2	1	1.7	2.0	28%
		Carbonation		2	2	2	2	1	2	1	1.7	2.0	28%
		Alkali-aggregate reaction		3	2	3	2	3	3	3	2.7	3.0	18%
		Sulfate attack	Delayed ettringite formation	2	2	3	2	3	3	3	2.6	3.0	21%
		Acid attack		1	1	2	1	1	2	2	1.4	1.0	37%
		Time dependent	Shrinkage	1	2	1	1	1	1	1	1.1	1.0	33%
		Temperature	Thermal cycles	2	2	1	1	1	1	1	1.3	1.0	38%
			Freeze-thaw	2	2	2	1	1	2	2	1.7	2.0	28%
		Differential settlement		2	2	3	2	2	2	3	2.3	2.0	21%
	Steel Reinforcement	Corrosion		3	3	3	2	2	2	2	2.4	2.0	22%
	Prestressing Tendons	SCC/Corrosion		2	3	1	2	2	2	2	2.0	2.0	29%

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Most nuclear power plants in the United States are currently licensed for up to 60 years of operation. The nuclear industry is assessing the feasibility of operation for up to 80 years. The U.S. Nuclear Regulatory Commission (NRC) and U.S. Department of Energy (DOE) co-sponsored the Expanded Materials Degradation Assessment (EMDA) to identify information gaps and research priorities for aging related degradation of reactor components for up to 80 years. Expert panels were convened to examine four main component groups using the phenomena identification and ranking technique: reactor core internals and piping systems, the reactor pressure vessel, concrete and civil structures, and electrical cables. Panelists included participants from NRC, DOE national laboratories, industry, academia, and international organizations. The EMDA reports include a ranking of degradation scenarios according to the probability of occurrence and level of knowledge, along with a summary of the current state of knowledge for each component group.

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