EXPERT PANEL WORKSHOP ON CONCRETE DEGRADATION IN SPENT NUCLEAR FUEL DRY CASK STORAGE SYSTEMS—SUMMARY REPORT

Prepared for

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EXECUTIVE SUMMARY

This report summarizes the technical proceedings of an expert panel workshop organized by the U.S. Nuclear Regulatory Commission (NRC) to evaluate the degradation of concrete structures in spent nuclear fuel dry cask storage systems (DCSSs). In DCSSs, concrete is commonly used for shielding structures, such as overpacks, as well as for the support pads on which the DCSS is placed. The NRC Office of Nuclear Material Safety and Safeguards considers the potential for aging-related degradation of these structures as part of its review for the renewal of specific licenses or Certificates of Compliance (CoCs), to ensure that these structures can perform their intended safety functions during the extended operating period. To date, there have been only a small number of reported occurrences of apparent degradation in concrete structures for DCSSs, but there have also been relatively few focused inspections. Given this limited information, and seeking to enhance the technical bases for its safety reviews, NRC staff engaged a group of outside experts from industry and academia to gain insights about the progression of concrete degradation phenomena, in-service inspection and monitoring technologies, degraded concrete repair approaches, and other aspects of aging management. The expert panel activities were managed by the NRC Office of Nuclear Regulatory Research, with additional support from the Center for Nuclear Waste Regulatory Analyses. A workshop with the expert panel was held as a public meeting at NRC Headquarters in Rockville, Maryland, on February 24–25, 2015.

The panelists discussed factors affecting the progression of degradation phenomena that commonly affect concrete structures with similar characteristics to the DCSS. These included freeze-thaw, salt scaling, acid and ion attack, alkali-silica reaction (ASR), thermal desiccation, creep, delayed ettringite formation, radiation, rebar corrosion, as well as the coupling or co-progression of multiple degradation processes. Based on the system design and operating environment, the modes of degradation identified as most likely to occur were freeze-thaw cracking, acid and ion attack, ASR, and reinforcing bar corrosion. All of these require the presence of water on the surface or within the mass of the concrete. Furthermore, because many of the mechanisms are temporally correlated either through chemical reaction or diffusion kinetics, the likelihood that degradation will occur increases over time. The panelists believed that all of the mechanisms will eventually manifest on external surfaces by cracking, discoloration, or some other feature, and could be detected by periodic visual inspection. However, degradation on below-grade or otherwise inaccessible areas may require soil excavation to directly detect. The panelists did not generically analyze the structural effects of the degradation mechanisms on the DCSS, but based on their knowledge of other reinforced concrete structures, they believed that the DCSS could maintain its safety functions provided that indications of degradation are promptly analyzed and repaired when they are detected.

The panelists identified methods used to prevent or mitigate the degradation of concrete structures, some of which involve design or fabrication approaches, and others that could be applied to systems already in service. The former include modifications to the concrete mix specifications and curing processes to increase durability; the latter include the use of chemical inhibitors, sealants and coatings to prevent moisture ingress, and cathodic protection. Many of these are addressed in American Concrete Institute (ACI), ASTM International (ASTM), or NACE International (NACE) codes and standards. If proposed for use by a licensee or CoC holder, NRC will evaluate such factors as the persistence and duration of the mitigation methodology, whether the mitigation methodology itself could introduce new degradation processes, or whether the method would affect the ability to inspect the system.

The expert panel provided useful insight into inspection and monitoring techniques commonly applied to concrete structures. These could involve the direct measurement or observation of physical features in DCSS components, or indirect measurements of environmental parameters that are indicators of conditions, which could promote degradation. The most basic type of direct measurement, but also the most important, is periodic visual examination of accessible surfaces for indications of cracking, discoloration, or other signs of distress. Other nondestructive techniques, such as ultrasonic testing, could detect subsurface indications of degradation, but these have not yet been demonstrated on DCSS components. If indications of degradation are detected by visual examination, there may be a need for limited removal of concrete cores for analysis and testing, but this approach should only be used cautiously to avoid compromising the integrity of the structure. Indirect measurements that could provide useful information include the measurements of chemical species and pH in soil groundwater. Best practices for the use of various inspection methods are addressed in ACI and ASTM codes and standards.

The expert panel discussed methods to repair and remediate concrete structures, which could include grout, epoxies, or overlays to fill in, reinforce, or cover over degraded areas. The selection of the repair method would involve specific consideration of the type of degradation mode, the rate at which it is progressing, and the size of the affected area. Criteria for the use and integrity testing of concrete repairs are found in ACI and ASTM codes and standards. Further, it is expected that repairs would be performed in accordance with the licensee Corrective Action Program, subject to the Quality Assurance requirements of Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50 or Part 72, as applicable.

An example of a generic aging management program (AMP) for reinforced concrete structures is found in the draft of NUREG–1927, Revision 1, “Standard Review Plan for Renewal of Spent Fuel Dry Cask Storage System Licenses and Certificates of Compliance.” The AMP, which includes such elements as preventive actions, detection of aging effects, monitoring and trending, acceptance criteria, and corrective actions, is conceptually similar to AMPs described for reactor license renewal in NUREG–1801, “Generic Aging Lessons Learned (GALL) Report.” The AMP involves periodic visual examination of accessible and below-grade surfaces in accordance with recommendations of ACI 349.3R, “Evaluation of Existing Nuclear Safety-Related Concrete Structures,” as well as groundwater monitoring consistent with the acceptance criteria in the ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL. The panelists believed that the proposed AMP was generally adequate to detect degradation prior to the loss of intended safety functions. Some concerns were raised related to the monitoring of additional groundwater species than those specified in Subsection IWL and which may be addressed in ongoing engagements with the codes and standards organizations.

Finally, the expert panel discussed the use of time-limited aging analyses (TLAAs) for evaluating the degradation of concrete structures. TLAAs, which involve calculations based on the proposed term of service for the structures, are also similar to those described in the GALL report. A potential use for a TLAA in a DCSS could include radiation-induced degradation analyses based on a calculated fluence during the period of extended operation. The panelists agreed that this is an appropriate use for a TLAA, and that TLAAs could be applied to other degradation phenomena, such as ASR, should sound technical models be developed.

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QUALITY OF DATA, ANALYSES, AND CODE DEVELOPMENT

DATA: All CNWRA-generated original data contained in this report meet the quality assurance requirements described in the Geosciences and Engineering Division Quality Assurance Manual. Sources for other data should be consulted for determining the level of quality for those data.

ANALYSES AND CODES: No computer analyses were conducted for this report.

TABLE OF CONTENTS

FIGURES

TABLE

ABBREVIATIONS	
3D	three dimensional
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ADAMS	Agencywide Document Access and Management System
AMP	aging management program
ANL	Argonne National Laboratory
ANS	American Nuclear Society
APHA	American Public Health Association
ASR	alkali-silica reaction
ASME	American Society of Mechanical Engineers
ASTM	ASTM International
CAP	Corrective Action Program
CENG	Constellation Energy Nuclear Group
CFR	*Code of Federal Regulations*
CNWRA	Center of Nuclear Waste Regulatory Analyses
CoC	Certificate of Compliance
C-S-H	calcium silicate hydrate
DCSS	dry cask storage system
DEF	delayed ettringite formation
DOE	U.S. Department of Energy
EMDA	Expanded Materials Degradation Assessment
EPRI	Electric Power Research Institute
ESCP	Extended Storage Collaboration Program
EST	Extended Storage and Transportation
FHWA	Federal Highway Administration
GALL	Generic Aging Lessons Learned
IAEA	International Atomic Energy Agency
IBC	International Building Code
ICRI	International Concrete Repair Institute
INL	Idaho National Laboratory
IR	infrared
ISFSI	independent spent fuel storage installation
MAPS	Managing Aging Processes in Storage
NACE	NACE International
NDT	nondestructive testing
NEI	Nuclear Energy Institute
NESCC	Nuclear Energy Standards Coordination Collaborative
NIST	National Institute of Standards and Technology
NMSS	Office of Nuclear Material Safety and Safeguards
NRC	U.S. Nuclear Regulatory Commission
RES	Office of Nuclear Regulatory Research
NWTRB	U.S. Nuclear Waste Technical Review Board
RG	Regulatory Guide
RH	relative humidity
SNF	spent nuclear fuel
SSC	structure, system, and component

SS	stainless steel
TMI	Three Mile Island
TLAA	time-limited aging analysis
UDOT	Utah Department of Transportation

1 INTRODUCTION

Beginning in the 1980s, a number of operating and decommissioned reactor sites in the United States, as well as some other facilities, began to place spent nuclear fuel (SNF) in dry cask storage systems (DCSSs). The U.S. Nuclear Regulatory Commission (NRC) licenses dry storage of SNF under Title 10 of the *Code of Federal Regulations* (10 CFR) Part 72, "Licensing Requirements for the Independent Storage of Spent Nuclear Fuel, High-Level Radioactive Waste, and Reactor-Related Greater than Class C Waste." Under the provisions of 10 CFR Part 72, initial specific licenses for independent spent fuel storage installations (ISFSIs) or Certificates of Compliance (CoCs) for casks may be issued for a term of up to 40 years, though to date, have only been granted for 20 years. Thereafter, they may be renewed for additional terms of up to 40 years. When DCSSs were initially placed into service in the 1980s, it was anticipated that a permanent geological repository would be available within 20 to 40 years. To date, however, a permanent disposal facility has not been licensed and spent fuel is likely to remain in dry storage longer than was expected. As such, the industry and NRC are addressing potential technical issues related to the first renewal of specific licenses and CoCs. Further, they are identifying technical information needed to ensure that SNF can be safely stored beyond the first renewal period, if needed, and eventually be transported to a permanent disposal facility.

To evaluate the adequacy of the technical basis for the safety evaluation of specific license and CoC renewal applications, NRC is reviewing the potential for aging-related degradation to affect the ability of structures, systems, and components (SSCs) that are important to safety to perform their intended functions during the extended licensing period. The scope of this analysis includes spent fuel assemblies, the metallic casks or canisters in which the fuel assemblies are placed, concrete or metallic shielding structures that house the casks or canisters, and the concrete storage pad. The subject matter of this report is the degradation of concrete, namely as it relates to the shielding structures and storage pad. The concrete structures must be analyzed separately from metals, which are used for most of the other SSCs in the cask system, because of differences in material structure, properties, and degradation processes.

The report describes the interactions of NRC staff with a panel of external independent experts, which was assembled to share their insights concerning various aspects of degradation in concrete structures. It was not intended that the panel evaluate any particular licensing action or DCSS design, nor provide a specific critique of the NRC regulatory framework. Rather, the panel was selected to help NRC staff better understand the current state of research in the field and to benefit from lessons learned from other technological sectors with which NRC staff are not intimately familiar. The focus of the staff interaction with the expert panel was a two-day public meeting held at NRC Headquarters in Rockville, Maryland, on February 24–25, 2015. In preparation for that meeting, NRC staff also conducted teleconferences with the panelists and solicited their input for topics which would be addressed. The summary (Oberson, 2015) and transcript of the public meeting (NRC, 2015a,b) have already been made publicly available in NRC's Agencywide Document Access and Management System (ADAMS). The purpose of this report is to provide further detail on the issues discussed by reviewing background information, distilling the key points from the panel interactions, and discussing potential implications for the aging management of concrete structures in DCSSs.

The content of the report is as follows. Chapter 2 provides background information on DCSS design, prior work on concrete performance in DCSSs, and the NRC regulatory framework. Chapter 3 addresses the process and objectives for the NRC staff interaction with the panel.

Chapter 4 describes the degradation mechanisms that are thought to be most likely to affect the concrete structures in DCSSs. Chapter 5 provides information about the prevention and mitigation of concrete degradation. Chapter 6 gives an overview of concrete structures inspection and condition monitoring techniques. Chapter 7 discusses concrete repair and mitigation technologies. Chapter 8 addresses the NRC's generic guidance for aging management of structures. Chapter 9 describes time-limited aging analyses. Finally, Chapter 10 provides a brief summary and set of recommendations for issues that need further consideration.

2 TECHNICAL BACKGROUND

As discussed in Chapter 1, dry cask storage has been used for spent nuclear fuel (SNF) management in the United States since the 1980s. Dry cask storage provides a strategic option for controlling the spent fuel pool inventory and for allowing decommissioning to proceed should the plant shut down. This chapter will provide a brief overview of the key design features of dry cask storage systems (DCSSs), with emphasis on concrete structures, the regulatory framework for ensuring the safety of DCSSs, and past work to evaluate the aging-related degradation of concrete structures in DCSSs.

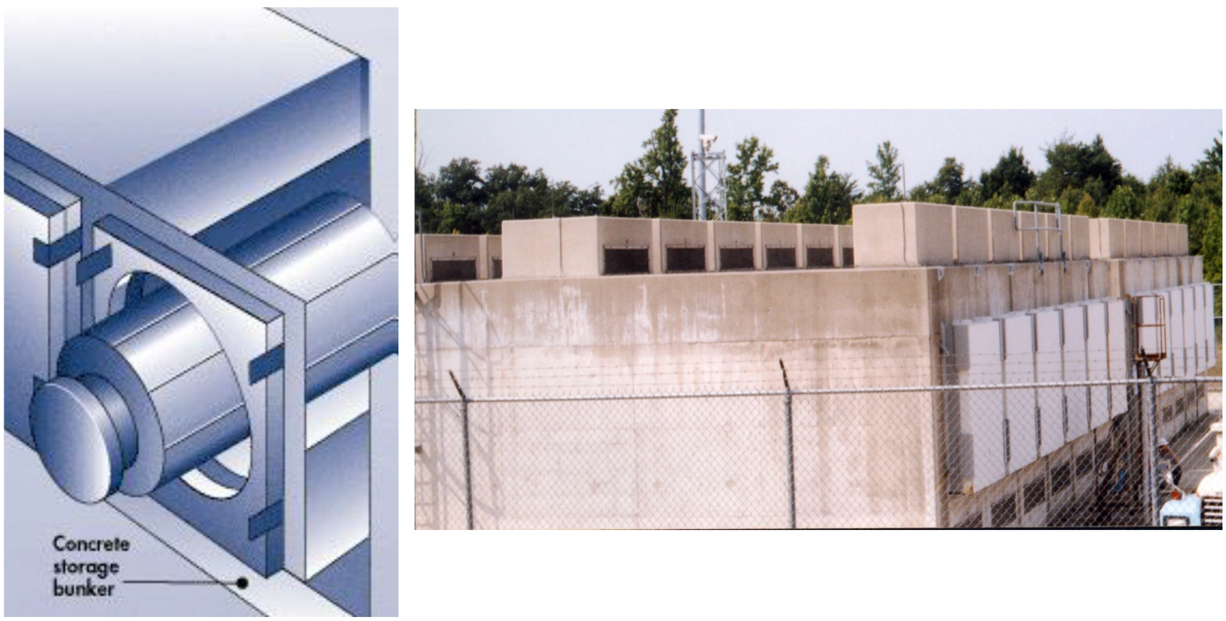
2.1 DCSS Design

A number of DCSS concepts, designed by various commercial vendors, are licensed by the U.S. Nuclear Regulatory Commission (NRC) for use at facilities in the United States. These designs are thoroughly described elsewhere and will only briefly be reviewed here. The features of a DCSS are intended to minimize the radiation dose to workers and members of the public, and to maintain the SNF in a condition that will allow its eventual retrieval and transportation to a permanent disposal facility. Generally, these safety functions include the confinement of radionuclides within multiple barriers, radiation shielding, criticality control, thermal management, and the maintenance of structural integrity to withstand design-bases events, such as seismic activity.

In the DCSS, spent fuel assemblies are typically placed within a grid-type structure in a metallic storage basket or canister. The number of assemblies in a single canister is typically between 20 and 80, depending primarily on the size of the assemblies and the fuel burnup. After loading in the spent fuel pool, the canister is dried, evacuated, backfilled with inert gas, and sealed by welding or bolting. The metallic canister may then be placed in either a horizontal or vertical orientation into a larger shielding structure referred to variously as a cask, overpack, or storage module. This shielding structure can be fabricated from metal, or metal-lined or unlined concrete. The shielding structure may have vents to the outside air to allow passive airflow for cooling. Most cask systems sit on top of thick reinforced concrete pads, though some are stored in below-grade cavities. Illustrations and photographs of vertically and horizontally oriented DCSSs are shown in Figure 2-1.

Detailed descriptions of the concrete cask, overpack, or storage module design features, including schematic drawings, are provided in a report by Electric Power Research Institute (EPRI) (EPRI, 2010). The concrete structures are of sufficient length or height to completely enclose the canister with a typical thickness in the range of 0.61 to 0.91 m [2 to 3 ft.]. The concrete may or may not have carbon steel reinforcing bars, depending on the system design and, as noted previously, some have a metallic liner. For unlined concrete structures, heat shields can be placed between the canister and concrete wall to protect the concrete against thermal damage. The concrete pad must support the weight of the cask systems, each of which may be well in excess of 100 tons, as well as the transfer equipment used to bring the canisters from the building in which they are loaded. The surface area of the pad depends on the number of systems to be stored, but may be tens of thousands of square meters [feet]. The thickness of the pad is typically about 0.91 m [3 ft.], sitting on top of soil or engineered fill. In locations with high seismic demand, concrete casks may be anchored to the pad.

(a)



(b)

Figure 2-1. (a) Illustration (NRC, 2016a) and Photograph (NRC, 2016b) of Vertically Oriented DCSS. (b) Illustration (NRC, 2016a) and Photograph (NRC, 2016c) of Horizontally-Oriented DCSS.

At present, there are approximately 2,200 loaded DCSSs in operation at over 70 licensed facilities in the United States. Of these, there are only a few incidences of known degradation for concrete structures, although the extent of focused inspections, particularly beyond exterior surfaces, is rather limited. One of these cases involves freeze-thaw cracking of horizontal storage modules used for the storage of Three Mile Island, Unit 2 fuel at Idaho National Laboratory (NRC, 2012), as shown in Figure 2-2. Shrinkage cracks were also detected

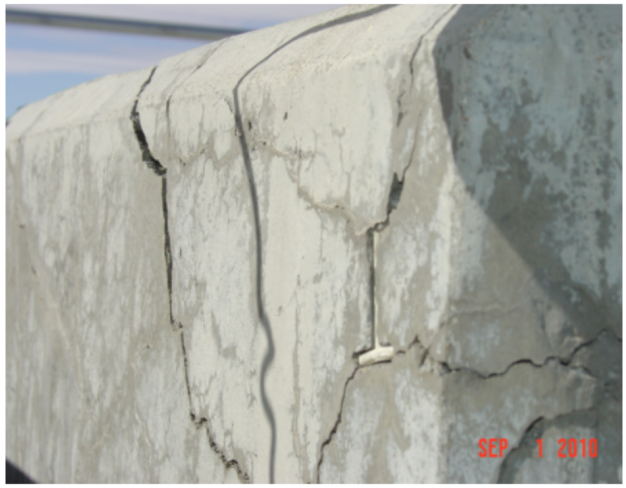


Figure 2-2. Photograph of Cracks on the Corner of a Horizontal Storage Module, Prior to Repair, at the Three Mile Island, Unit 2 Spent Fuel Storage Facility at Idaho National Laboratory (NRC, 2012)

on vertical storage casks at Arkansas Nuclear One (NRC, 2005). A few inspections were performed in which remote cameras were inserted through vents to record the condition of the interior surfaces of the DCSS. These inspections have not identified evidence of any gross degradation of concrete structures. It appears that rainwater and airborne particulate matter are able to pass through the vents and fall on the interior surfaces. Further, some small stalactites, indicative of leaching from the concrete, were seen on horizontal storage module walls at the Calvert Cliffs facility (CENG, 2012).

2.2 Regulatory Framework

As discussed in Chapter 1, the regulations addressing dry storage of SNF are set forth under Title 10 of the *Code of Federal Regulations* (10 CFR) Part 72, including the license types, license terms, required content for an initial or renewal application, and general design criteria. Many of the regulatory provisions address issues specifically related to concrete structure design, performance, and aging management. For instance, prior to the issuance of a specific license or Certificates of Compliance (CoCs), the applicant must analyze the environmental conditions and external events to which the system will be exposed, and provide a detailed description of structures, systems, and components (SSCs) important to safety, including their materials of construction, dimensions, and codes and standards applicable to their design. The applicant must demonstrate that the SSCs can withstand the effects of the environmental conditions and natural hazards for the licensing term and maintain the confinement of the fuel. There is expected to be a capability to test for and monitor the functionality of the SSCs important to safety. To renew a specific license or CoC, the applicant must demonstrate that SSCs important to safety can continue to perform their intended function during the extended licensing term. The applicant may provide time-limited aging analyses (TLAAs) or other calculations to show that design safety margins will not be exceeded with the additional service life. The applicant may also commit to aging management programs (AMPs), which typically involve periodic monitoring, inspection, and the implementation of corrective actions to detect and mitigate potential degradation.

More specific guidance on the implementation of the regulations in 10 CFR Part 72 as they pertain to concrete structures is found in NRC (2010a). Referring to reinforced nonconfinement structures important to safety, NRC (2010a) states

> "NRC accepts construction in accordance with ACI 349 or ACI 318. Selection and validation of the proper concrete mix to meet design requirements are considered a construction function. By contrast, specification of cement type, aggregates, and special requirements for durability and elevated temperatures is considered a design or material selection function and is, therefore, governed by ACI 349 (and/or ACI 359, if applicable)."

Further, regarding storage pads, NRC (2010a) states that

> "Reinforced concrete pads…should be designed and constructed as foundations under an applicable code such as, ACI 349, ACI 318, or [International Building Code] IBC. Such pads typically are not classified as important to safety; however, in some cases they may be."

NRC (2015c) provides generic guidance related to the development of TLAAs and AMPs, which will be discussed in additional detail throughout this report.

2.3 Prior Work

Over the past several years, both NRC and external stakeholders have been evaluating technical issues related to the long-term performance of concrete in DCSSs. This subsection will briefly review some of this work.

2.3.1 NRC Activities

NRC activities related to the performance of concrete in DCSSs can be understood to support the development of the technical bases for the review of the first renewal applications for specific licenses and CoCs, as well as to identify potential knowledge gaps for longer term storage, should it become necessary. With regard to the former, NRC has already issued a number of specific license renewals and one CoC renewal to allow operation up to 60 years (i.e., an initial 20-year license term and a 40-year renewal term). Staff recognized that the review of the renewal applications was lengthy and complicated because of limited guidance to licensees and staff about expectations for the technical content of the applications. To improve the efficiency of the renewal process, NRC is currently updating NUREG–1927 (NRC, 2015c) and developing new generic guidance referred to as the Managing Aging Processes in Storage (MAPS) report (NRC, 2015d). The update to NUREG–1927 (NRC, 2015c) will provide a more detailed description of the scope of the license renewal review, and the format and content of TLAAs and AMPs. The MAPS report is intended to be analogous to NUREG–1801 (NRC, 2010b) for reactor license renewal. It will include a listing of the components for commonly used DCSS designs, their materials of construction, the environments to which they are exposed, the aging-related degradation phenomena to which they could potentially be susceptible, and examples of AMPs that NRC staff would consider to be acceptable for addressing the effects of aging for up to 60 years.

Considering longer term operation, in 2010, the NRC Commissioners issued staff requirements (COMDEK-09-0001; NRC, 2010e) to develop plans for identifying data needs to ensure the ability to safely store and eventually transport SNF beyond 120 years (i.e., an initial 40-year reactor licensing term with a subsequent 20-year renewal term, then an initial 20-year storage term with a subsequent 40-year renewal term). In response to the Commission directions, staff decided on an analytical timeframe of 300 years following SNF discharge from the reactor (COMSECY-10-0007; NRC, 2010f). A primary work product of what is referred to as the Extended Storage and Transportation (EST) regulatory program review was the report entitled "Identification and Prioritization of the Technical Information Needs Affecting Potential Regulation of Extended Storage and Transportation of Spent Nuclear Fuel," or TIN report (NRC, 2014a). The following description of the TIN report (NRC, 2014a) comes from the Executive Summary:

> "For each of the major systems, structures, and components (SSCs) of a dry cask storage system, a set of potential degradation phenomena was developed based on existing assessments, new analyses, and staff experience. For each potential degradation process or issue, staff assessed the current level of knowledge, with particular emphasis on knowledge specifically related to performance of the dry storage SSC. NRC staff with experience in regulatory reviews of dry storage and transportation evaluated each of the SSC degradation mechanisms to determine how it could affect the ability of the SSC to meet the safety regulations for storage and transportation. For the final assessment, the staff prioritized the areas for further technical investigation, using the following criteria: (i) regulatory significance for safety performance and (ii) the level of knowledge about the process or issue. In general, those areas with high safety significance and low level of knowledge ranked highest for further investigation."

NRC (2014a, Appendix A–8) addressed degradation of concrete overpacks, vaults, and pads. A summary of the evaluations is provided herein as Table 2-1. These degradation mechanisms are discussed in further detail in Chapter 4 of this report.

In broad terms, the current state of knowledge concerning concrete degradation was assessed as high, but also with a high need for further research, though a second tier priority. The following rationale was provided.

> "Concrete is the primary shielding for storage and transportation in most systems. Knowledge of the various degradation mechanisms is variable, but overall has been rated high assuming that monitoring can identify early signs of degradation. If analysis of monitoring methods shows that early degradation cannot be reliably detected, then evaluation of individual degradation mechanisms will have higher priority."

NRC also maintains active research programs to address concrete degradation in reactor systems, particularly in safety-related structures such as the containment building and biological shield wall. Most degradation phenomena likely to affect concrete in DCSSs could also affect the reactor structures. Further, because the reactor structures are generally older than the DCSS, and in some respects, are subjected to a more aggressive environment, degradation of the former could be a leading indicator for degradation of the latter. Notably, NRC and the U.S. Department of Energy (DOE) recently completed a project in which a panel of subject matter experts evaluated the current state of knowledge concerning the degradation of concrete in reactor structures for timeframes up to 80 years. The results were reported in NUREG/CR–7153, "Expanded Materials Degradation Assessment (EMDA): Aging of Concrete and Civil Structures" (NRC, 2014b, Volume 4). The findings will not be discussed in detail here, but the EMDA panelists identified, for example, knowledge gaps related to the progression of alkali-silica reactions (ASRs) and the effects of irradiation on concrete, both of which are discussed in Chapter 4 of this report. NRC sponsored research to address these knowledge gaps for reactor systems, as described in a 2014 staff presentation to the Advisory Committee on Reactor Safeguards (NRC, 2014c), which can be leveraged to support the analyses of DCSSs as well.

2.3.2 External Stakeholder Activities

Industry, DOE, and other external stakeholders have also undertaken activities related to concrete performance in DCSSs that are in many ways similar to those by NRC. One set of activities can be described as gap analyses or knowledge base assessments for the long-term storage of SNF, which are generally comparable to activities that were subjects of the TIN report (NRC, 2014a). These include:

- DOE Used Fuel Disposition Campaign Report FCRD–USED–2011–000136, "Gap Analysis To Support Extended Storage of Used Nuclear Fuel" (ANL, 2012)

- U.S. Nuclear Waste Technical Review Board (NWTRB) Report, "Evaluation of the Technical Basis for Extended Dry Storage and Transportation of Used Nuclear Fuel" (NWTRB, 2010)

- EPRI Technical Report 1022914, "Extended Storage Collaboration Program (ESCP) Progress Report and Review of Gap Analyses" (EPRI, 2011)

- EPRI Technical Report 1026481, "International Perspectives on Technical Data Gaps Associated with Extended Storage and Transportation of Used Nuclear Fuel" (EPRI, 2012b)

While the respective reports take somewhat different approaches, there is broad agreement on many issues related to concrete, including the identification of potential degradation phenomena and the need for improved monitoring and inspection capabilities.

Both industry and DOE support research and development on long-term concrete performance. As is the case for NRC, these are largely programs that were conceived to address reactor structures, but would provide valuable insights for DCSSs as well. Descriptions of these programs can be found in INL/EXT-12-24562, Revision 4, "DOE-NE Light Water Reactor Sustainability Program and EPRI Long Term Operations Program–Joint Research and Development Plan" (INL, 2015a) and INL/EXT-11-23452, Revision 3, "Light Water Reactor Sustainability Program Integrated Program Plan" (INL, 2015b). Only limited work has been done specifically on concrete for DCSSs, primarily funded by DOE through the Nuclear Energy University Program; examples include work done to evaluate capabilities for detecting ASR, hazards analyses, and investigations of novel concrete designs.

Table 2-1. Level of Knowledge and Monitoring Techniques—Concrete Overpacks, Vaults, and Pads

Component	Degradation Phenomena	Level of Knowledge: Initiation Time	Level of Knowledge: Propagation Rate	Level of Knowledge: Degradation or Failure Complete	Monitoring or Inspection Capability Available
Concrete overpacks, vaults, and pads	Shrinkage cracking	H	H	H	Visual observation
	Creep	H	H	H	Visual observation
	Fatigue	H	H	H	Visual observation
	Rebar corrosion	M	M	L	Visual observation Electrochemical monitoring
	Carbonation	M	M	L	Visual observation Core sample testing
	Leaching	H	H	H	Visual observation
	Sulfate attack	H	H	M	Visual observation Petrographic examination
	Alkali-silica reaction	H	M	L	Visual observation Petrographic examination
	Radiation damage	H	M	M	Visual observation Shield testing and radiation measurements
	Freeze-thaw	H	M	M	Visual observation Petrographic examination
	Thermal dryout	M	M	L	Testing required to assess degradation
	Thermal degradation of mechanical properties	M	M	L	Testing required to assess degradation
	Coupled mechanisms	M	L	L	Visual observation Petrographic examination

H = High
M = Medium
L = Low

3 EXPERT PANEL OBJECTIVES AND PROCESS

The use of a panel of independent, outside experts as an informational resource for staff is a routine activity within U.S. Nuclear Regulatory Commission (NRC). General principles for this process are presented in NUREG–1563, “Branch Technical Position on the Use of Expert Elicitation in the High-Level Radioactive Waste Program” (NRC, 1996b). Further, staff reviewed lessons learned from other panels recently used by NRC, such as that used to analyze reactor building containment liner corrosion (Petti et al., 2011) and that used for the “Expanded Materials Degradation Assessment (EMDA)” (NRC, 2014b). With this context, the steps followed to execute the panel can be broken down as follows:

(1) Defining objectives
(2) Selecting experts
(3) Disseminating information
(4) Preparing for the workshop
(5) Conducting the workshop
(6) Documenting the workshop findings.

These will be briefly discussed in the following sections.

3.1 Definition of Objectives

The objectives for the expert panel were determined by reviewing the findings of the report entitled “Identification and Prioritization of the Technical Information Needs Affecting Potential Regulation of Extended Storage and Transportation of Spent Nuclear Fuel” or TIN report (NRC, 2014a) and the gap analysis reports of the external stakeholders (ANL, 2012; NWTRB, 2010; EPRI, 2011; EPRI, 2012b). These reviews were used to compile a listing of issues for which more information would be needed for NRC staff to assess long-term concrete performance in dry cask storage systems (DCSSs). Further, challenges were identified from the NRC staff’s first few specific license or Certificates of Compliance (CoCs) renewal reviews. From these starting points, it became clear that the objective for the panel would be to address a number of high-level topics, including

- Has the TIN report identified the degradation phenomena most likely to affect DCSS concrete structures and assessed the current state of knowledge adequately?

- What is the current state of development for methods to prevent or mitigate the degradation phenomena?

- Are there inspection methods (visual or other) to detect indications of the degradation phenomena before the functionality of the concrete structures is lost?

- Are there generally accepted criteria for determining the adequacy of concrete repairs?

- Could generic aging management programs (AMPs) or time-limited aging analyses (TLAAs) be developed to address the degradation phenomena most likely to affect the DCSS?

3.2 Selection of Experts

The staff's main criterion for selecting the panel participants was demonstrated expertise in concrete structure design, degradation processes, inspection, repair, and/or functional assessment. Further, the panelists could have no conflicts of interest, which would primarily involve the performance of similar work for NRC licensees. Finally, staff valued diversity in professional experience and desired representation from both commercial industry and academia. To identify potential panelists, NRC and the Center for Nuclear Waste Regulatory Analyses (CNWRA) staff surveyed recent technical literature, professional contacts, and individuals active on American Concrete Institute (ACI) or American Society of Mechanical Engineers (ASME) code committees. From an initial listing of several dozen names, staff selected a smaller set of individuals whose expertise would match up with each of the main topical areas (e.g., degradation, inspection, repair, functional assessment). Based on past experience, staff believed a panel of five to eight members would provide the requisite breadth of knowledge but would not be too cumbersome to manage. Staff then reviewed the current and previous work done by the potential panelists to confirm that there would be no conflicts of interest. By this process, the following individuals were selected to form the expert panel:

- NEAL BERKE—Dr. Berke has a Ph.D. in Metallurgical Engineering and an A.B. in Physics. He is an expert in concrete design, degradation, and repair and has more than 30 years of practice in research. Dr. Berke is currently the vice president of Tourney Consulting Group, LLC in Kalamazoo, Michigan.

- LAURENCE JACOBS—Dr. Jacobs has a Ph.D. in Engineering Mechanics, an M.S. in Civil Engineering (Structures), and a B.S. in Civil Engineering. He is an expert in the nondestructive examination of concrete structures and has more than 25 years of academic experience. Dr. Jacobs is currently the associate dean for academic affairs in the College of Engineering at Georgia Institute of Technology, Atlanta, Georgia.

- RANDY JAMES—Mr. James has an M.S. degree in Engineering Mechanics and a B.S. degree in Engineering Science. He is an expert in concrete design and structural integrity analysis and has more than 35 years of research experience. Mr. James is currently the senior associate and director of the Structures Division at ANATECH, a company of Structural Integrity Associates at San Jose, California.

- JOHN POPOVICS—Dr. Popovics has a Ph.D. in Engineering Science and Mechanics, an M.S. in Civil Engineering, and a B.S. in Civil Engineering. He is an expert in the nondestructive examination of concrete structures and has more than 20 years of academic experience. Dr. Popovics is currently a professor in the Department of Civil and Environmental Engineering at the University of Illinois at Urbana-Champaign, Illinois.

- YUNPING XI—Dr. Xi has a Ph.D. in Structural Engineering, an M.S. in Structural Engineering, and a B.S. in Civil Engineering. He is an expert in concrete design, degradation, and repair and has more than 30 years of combined experience as a professor, structural engineer and researcher. Mr. Xi is currently a professor at the University of Colorado at Boulder, Colorado.

3.3 Disseminating Information

Prior to the expert panel workshop, only a list of relatively few reference documents was provided to the panelists, as it was expected that they would largely draw upon their existing knowledge and experience. All of the documents were publicly available and include

- NRC TIN Report, Identification and Prioritization of the Technical Information Needs Affecting Potential Regulation of Extended Storage and Transportation of Spent Nuclear Fuel (NRC, 2014a)
- ANL Report, Managing Aging Effects on Dry Cask Storage Systems for Extended Long-Term Storage and Transportation of Used Fuel (ANL, 2014)
- ACI 349-06, Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (ACI, 2007c)
- ACI 349.3R, Evaluation of Existing Nuclear Safety-Related Concrete Structures (ACI, 2010a)
- ACI 201.1R, Guide for Conducting a Visual Inspection of Concrete in Service (ACI, 2008a)
- ACI 228.2R, Report on Nondestructive Test Methods for Evaluation of Concrete in Structures (ACI, 2013b)
- ASME Subsection IWL, Requirements for Class CC Concrete Components of Light-Water-Cooled Plants, Boiler and Pressure Vessel Code, Section XI (ASME, 2001)
- Nuclear Energy Standards Coordination Collaborative (NESCC) 11-008-13, Codes and Standards for the Repair of Nuclear Power Plant Concrete Structures: Recommendations for Future Development (NIST, 2013)

3.4 Preparing for the Workshop

Prior to the expert panel workshop, NRC staff and CNWRA conducted two teleconferences with the panelists to familiarize the panelists with the workshop objectives and process. The majority of the first teleconference constituted a presentation by NRC staff to the panelists on DCSS design concepts, operational experience, and safety regulations. After the teleconference, the panelists were asked to provide brief written answers to a series of high-level questions on long-term concrete structures' performance related to the findings of the TIN report (NRC, 2014a). At the second teleconference, NRC staff presented an overview of the generic AMP on reinforced concrete structures in NUREG–1927, Revision 1 (NRC, 2015c). After this teleconference, the panelists were again asked to provide brief written answers to a series of questions on more specific aging management-related issues. The purpose of the respective questionnaires was to allow the panelists to suggest topics to be addressed at the workshop and to help staff set the order of discussion for the workshop agenda. Copies of the questionnaires are provided as Appendices A and B of this report.

3.5 Conducting the Workshop

The expert panel workshop was conducted as a public meeting at NRC Headquarters in Rockville, Maryland, on February 24–25, 2015. Advance notification of the meeting was placed on the NRC website, following NRC policies for public meetings. The workshop was designated as a Category 3 meeting, which is described as follows in Management Directive 3.5:

> "This type of meeting would be held with representatives of non-Government organizations, private citizens or interested parties, or various businesses or industries …to fully engage them in a discussion of regulatory issues."

The format of the meeting was for staff to ask the panelists questions related to the topical areas described in Section 3.1. The panelists were to respond to the questions based on their experience and expert judgments. The responses were understood to be the individual opinions of the respective panelists, although the panelists were encouraged to discuss the basis and rationale for their judgments. The staff did not attempt to prompt the panel toward an agreed upon, consensus opinion. The staff members participating in the panel were the subject matter experts from the NRC offices of Nuclear Regulatory Research and Nuclear Material Safety and Safeguards, as well as NRC contractors from CNWRA. While the meeting was open to members of the public, their participation was limited to public comment periods at the end of each day of the workshop. Members of the public were not permitted to ask questions directly of the panelists, but rather could ask questions of the NRC staff, who could then engage the panelists to help provide a response, as needed.

3.6 Documenting the Workshop

This report is intended to stand as the most complete and final documentation of the expert panel activity. Since the workshop in February 2015, NRC staff have had no further engagement with the panel, except to provide them an opportunity to review this report prior to its publication. Staff also issued a public meeting summary (Oberson, 2015), following NRC policy, and released a full transcript of the meeting (NRC, 2015a,b).

4 CONCRETE DEGRADATION

There are several potential degradation mechanisms that could affect dry cask storage systems (DCSSs) in short- and long-term periods. In a report entitled “Identification and Prioritization of the Technical Information Needs Affecting Potential Regulation of Extended Storage and Transportation of Spent Nuclear Fuel” or TIN report (NRC, 2014a), the technical bases of many concrete degradation mechanisms applicable to DCSSs were presented and evaluated to identify relevant knowledge and practice from nuclear and nonnuclear concrete structures, and to identify information needs. The TIN report (NRC, 2014a) assessment of the level of knowledge drew on several sources (NRC, 2011a; Hanson et al., 2011; NWTRB, 2010; EPRI, 2011). This chapter presents a more detailed characterization and understanding of the existing technical basis for DCSSs and identifies additional degradation modes relevant to DCSSs not addressed in the TIN report (NRC, 2014a), including those the concrete expert panel identified (e.g., salt scaling).

4.1 Freeze and Thaw

4.1.1 Technical Background

Concrete materials that are saturated or nearly saturated with water can be damaged by repeated freezing and thawing cycles. Because water expands when freezing, fully or mostly saturated concrete will experience internal stresses from the expanding ice during a cooling event. Considering the local water transport mechanisms in a pore network with different pore sizes, less saturated pores with an initial saturation of as low as 77 percent (Yang et al., 2006) can become saturated during a cooling process. Fagerlund (1977) indicated that below the critical water saturation level of about 77 percent, concrete damage is not expected even under severe cyclic freezing and thawing. Li et al. (2012) conducted experiments on concretes with various degrees of water saturations and demonstrated that freeze and thaw does not take place below a critical water saturation of about 86 percent. As moisture enters the concrete surface, the expansion of water in concrete as the result of freezing can cause cracking, surface scaling, or joint deterioration when pressures exceed the concrete tensile strength [ACI 201.2R (ACI, 2008c); Pigeon, 1994]. Surface scaling is primarily induced by freezing in the presence of deicing salts (Marchand et al., 1994), whereas cracking is typically associated with the damage of coarse aggregates by freezing and thawing cycles (Sawan, 1987). The salt scaling degradation mechanism is described in detail in Section 4.2 of this report. ASTM C666/C666M (ASTM, 2003) provides a standard testing method to investigate the freeze and thaw damage of concrete by subjecting the concrete to 300 freeze and thaw cycles. This form of concrete damage can be detected by visual examination of the concrete surface.

Numerous damage modes of freeze and thaw cycles have been postulated. The degradation mechanisms include the expansion of concrete due to the generation of internal hydraulic pressures (Powers, 1949) and osmotic pressures (Penttala, 1998). The hydraulic pressure theory is based on the notion that when a concrete pore is critically saturated, the concrete expansion, resulting from the water to ice transformation, promotes excessive water to flow away from the freezing site. The hydraulic pressure is generated as a result of the resistance of the concrete matrix to the movement of water, which is a function of the freezing rate and the characteristics of the concrete pore network (Pigeon et al., 1985; Yamashita et al., 1997). For instance, high freezing rates and low concrete permeability could result in high hydraulic pressure. The osmotic pressure theory is based on the movement of water toward the freezing sites in the concrete pores, resulting in an increase in the ion concentration of the unfrozen pore solution, creating an osmotic potential that absorbs water from the surrounding concrete pores.

This effect leads to the gradual filling of large pores during freezing and thawing cycles. Once a pore is full, further water absorption or ice formation will cause either osmotic or expansion pressure in the pore.

Numerous studies have demonstrated that air entrainment in concrete can be used to increase the freeze-thaw resistance of Portland cement concrete exposed to freezing and thawing cycles (Powers, 1949). Microscopic entrained air bubbles, which are usually evenly distributed in the concrete paste, take on the water during freezing to relieve pressure buildup in the concrete. ACI 318-05 (ACI, 2005a) provides values of air content as a function of the aggregate size for moderate and severe freeze and thaw conditions. However, ACI 318-05 (ACI, 2005a) does not address freeze-thaw susceptibility of aggregates. Generally, air entrainment between 3.5 and 7.5 percent and air bubble spacing of less than 0.25 mm [0.01 in] provide satisfactory freeze-thaw performance. However, entrained air will not protect concretes containing coarse aggregates that undergo disruptive volume changes when frozen in a saturated condition [ACI 318-05 (ACI, 2005a)]. In addition, ASTM C260/C260M (ASTM, 2010a) provides a standard specification for air-entraining admixtures to be added to concretes in the field.

The degradation of concrete due to freeze and thaw could occur throughout the life of the structure. NRC (2014a) provides more detailed information on the mechanics of freeze and thaw, ways to enhance the freeze-thaw resistance of concrete, knowledge gap assessments, and operating experience, such as freeze-thaw damage of the concrete at the TMI-2 fuel Independent Spent Fuel Storage Installation at Idaho National Laboratory.

4.1.2 Summary of Workshop Discussion and Expert Panel Assessment

Dr. Berke indicated that the freeze-thaw process is purely a mechanical degradation; external concrete surfaces are directly exposed to outdoor environments where freeze and thaw cycles are common. Freeze and thaw is related to the amount of water saturation of the concrete pore network. Dr. Popovics indicated that freeze and thaw will likely proceed when the concrete pore network is fully saturated. In particular, freeze-thaw is more damaging for horizontal concrete surfaces (e.g., concrete roofs and pads) than vertical concrete surfaces, because external water can accumulate on horizontal surfaces for extended periods of time. Dr. Berke noted that it is very hard to saturate a vertical concrete surface, especially if there is a heat source on one side, as in the case of DCSSs. The influx of heat of a DCSS will likely remove the moisture from the concrete wall. Dr. Berke noted that freeze and thaw can also occur below ground in a water-saturated soil above the frost line.

Dr. Berke mentioned that when the concrete is not saturated with water, freeze and thaw will not happen in most cases. Dr. Xi indicated that the amount of moisture inside the concrete can dictate the initiation of freeze and thaw. Nevertheless, there is much debate about the percentage of moisture inside the concrete, which can trigger freeze and thaw. According to Dr. Xi, the typical moisture levels inside the concrete leading to freeze and thaw have to be about 80 percent or greater. This concrete moisture level limit is an estimated value based on a coupling mechanism involving 9 percent volume expansion when water turns into ice and the local water transport mechanisms in a pore network with different pore sizes. In addition, Drs. Popovics and Berke noted that freeze and thaw can occur with or without the presence of salts in solution contacting the concrete surface, as indicated later in Section 4.2 of this report. Dr. Berke indicated that porous aggregates can absorb water and, under freezing conditions, can experience freeze and thaw. Freeze and thaw is closely related to the concrete microstructure; in particular, to the degree of air entrainment. However, Dr. Popovics mentioned that there is no protection against freeze and thaw damage. In some cases, dense concretes with no air entrained are resistant to freeze and thaw damage. Drs. Berke and Popovics indicated that construction practices and the type of weather during concrete placement also play an important role in the development of freeze and thaw. For instance, concrete casting in winter weather can promote freeze and thaw due to the presence of mixing water and the initial low strength of the concrete, even with an optimal air void system. Dr. Berke noted that the outside concrete surface of a DCSS can be subject to freeze and thaw, whereas the internal temperatures of the DCSS would preclude freeze and thaw from happening on the internal concrete surfaces.

4.1.3 Considerations for Mitigation and Inspection

The panelists indicated that although there is no absolute prevention against freeze and thaw damage, provisions can be adopted to minimize its likelihood. These include proper materials selection (e.g., nonporous aggregates, low concrete water saturation, low water-to-cement ratio, and optimal air entrainment) and design to avoid water ponding on horizontal surfaces. The heat load from the emplaced fuel may contribute to drying of the internal concrete surface, though the heat load will decay over time. Finally, for existing DCSSs, mitigation of freeze and thaw could be attempted by drying and sealing the concrete surface, as discussed in further detail in Chapter 5 of this report.

In NUREG–1927, Revision 1 (NRC, 2015c), NRC staff proposed a generic program to manage the degradation of reinforced concrete structures in DCSSs, which relies on periodic visual examination of accessible surfaces, as described in ACI 349.3R (ACI, 2010a), and groundwater monitoring consistent with the acceptance criteria in the ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL (ASME, 2001). Because freeze-thaw degradation is more likely to affect exterior surfaces of the structure than the internal mass, the visual examination should be adequate to detect indications of this type of damage before it is sufficiently widespread to compromise the intended safety functions. The primary inspection challenge would relate to below-grade areas where partial removal of topsoil surrounding the concrete may be necessary to access the surface. For such cases, a licensee may need a site-specific plan for below-grade inspections and/or to use opportunistic inspections if the potential for this form of degradation cannot be excluded by engineering analysis.

If indications of freeze-thaw degradation are observed on the surface of the structure, additional examinations may be warranted to evaluate the extent of its progression within that structure, or others on the same site with similar design characteristics as the affected structure. The panelists noted that withdrawal of a core sample is likely to provide valuable information by allowing mechanical, chemical, and petrographic testing. They acknowledged, however, that caution should be exercised when coring concrete structures, as the process itself could damage the structure or place it into an unanalyzed condition. It should not be used as a purely exploratory approach. Nondestructive volumetric examination techniques—for instance, ultrasonic testing—may be able to detect subsurface voids or cracks, but have not yet been applied to DCSSs.

4.2 Salt Scaling

4.2.1 Technical Background

Similar to freeze and thaw damage, as discussed in Section 4.1, salt scaling takes place when concrete is exposed to freezing temperatures, moisture, and dissolved salts. The degradation is maximized at a moderate concentration of salt (e.g., from deicing salts), called pessimum concentration (Marchand et al., 1999). Verbeck and Klieger (1957) reported that the pessimum concentration is independent of the types of salt species and is about 3 to 4 percent of the solute by weight. The most common deicing salts are sodium chloride and calcium chloride. The other deicing chemicals include magnesium chloride, urea, potassium chloride, ammonium sulfate, and ammonium nitrate.

Salt scaling damage manifests as flaking of a small amount of concrete material from the surface. In addition, petrographic examinations following ASTM C856 (ASTM, 2014a) and air-void analyses of hardened concrete per ASTM C457/C457M (ASTM, 2012a) are the two most common laboratory procedures used to investigate concrete scaling. Other experimental methods following ASTM C672/C672M (ASTM, 2012b) are commonly used to mimic the natural environment of cyclic freeze and thaw of concrete by ponding a solution containing salts. This form of concrete damage can be detected by visual examination of the concrete surface.

Several potential salt scaling mechanisms have been proposed. For instance, in the study conducted by Lindmark (1998), the damage was attributed to salt crystallization near the concrete surface. On the other hand, Valenza and Scherer (2005) found that there is a large mismatch in the thermal expansion coefficients of ice and concrete materials, such that the frozen layer develops high tension during cooling. The unfrozen, ponded liquid creates brine pockets that weaken the ice and promote cracking. These cracks are expected to intersect the surface of the concrete and cause damage. The solute may play an additional role by contributing to weakening of the surface of the paste. As the saline solution freezes, it produces pure water ice plus brine, whose concentration increases as temperature decreases.

Jana (2004) has shown that an inadequate amount of entrained air and an air-void system in concrete decrease the salt scaling resistance of concrete. However, an adequate air entrainment and a proper air-void system alone cannot guarantee adequate salt scaling resistance of concrete, because other factors such as the concrete materials, proportioning, construction practices, and concrete maturity also affect salt scaling resistance of concrete. Preventive measures commonly applied to freeze and thaw degradation can be used to mitigate salt scaling. These preventive measures are described in Section 4.1.

Salt scaling of concrete roadways, pavements, sidewalks, driveways, decks, and other slabs is a common problem in locations exposed to cyclic freezing and thawing and deicing salts, but was not identified in NRC (2014a) as a potential degradation mechanism for concrete DCSSs.

4.2.2 Summary of Workshop Discussion and Expert Panel Assessment

Drs. Popovics and Berke indicated that salt scaling is a well-understood concrete degradation mechanism that occurs when ponded salt at a pessimum concentration is exposed to frost conditions. Dr. Popovics indicated that salt scaling will not occur at a very high or very low concentration of ponded salt. The pessimum concentration is on the order of 3 to 4 percent of the solute by weight, and it is independent of the types of salt species.

Drs. Berke and Popovics noted that this degradation mode is typically a problem for horizontal concrete surfaces, such as concrete pads, sidewalks, and bridge decks that have poor drainage. For vertical surfaces, this damage mechanism would only be a problem if the concrete is exposed to standing water in the soil with salt in it. Dr. Berke indicated that the geographical location of the concrete structure plays an important role. For instance, concrete structures placed near marine environments where temperatures can drop below the freezing point are susceptible to salt scaling.

Drs. Berke and Popovics indicated that the effect of salt scaling could become significant over time if the concrete structure continues to lose material at the surface. This degradation mode can also enhance other concrete degradation mechanisms such as the corrosion of the reinforcing steel by eroding its concrete coverage.

4.2.3 Considerations for Mitigation and Inspection

The primary approach for preventing the occurrence of salt scaling would be to limit the exposure of the DCSS to salt, as well as to avoid locations of standing water. The former could be addressed, in part, by controls on the use of deicing salts at locations near the DCSS. Other sources of salt, however, such as airborne species in near-marine environments, may still be present. Prevention of water ponding on the pad can be achieved by proper concrete mix design and grading. As is the case for freeze-thaw degradation, salt scaling will typically initiate on the surface of the affected structure. Therefore, the considerations for inspection of the DCSS described in Section 4.1.3 largely apply to salt scaling as well. In short, periodic visual examination of accessible surfaces, in accordance with ACI 349.3R (ACI, 2010a), should be adequate to detect indications of degradation in concrete structures before the loss of safety function. A site-specific plan may be needed to address below-grade structures if the occurrence of this phenomenon could not be excluded by engineering analysis. Particular attention should be given to the potential for this degradation mode if site groundwater or soil monitoring shows high concentrations of salts.

4.3 Acid/Ion Attack

4.3.1 Technical Background

The intrusion of aggressive ions or acids into the pore network of Portland cement concrete can cause various degradation phenomena. The aggressive ion attack is typically originated by external sources of sulfate, chloride, or magnesium ions in contact with the concrete. Depending on the type of aggressive ion, the degradation of concrete can manifest in the form of cracking, loss of strength, concrete spalling and scaling, and corrosion of the reinforcing steel.

External sulfate attack is a process whereby species such as K_2SO_4, Na_2SO_4, $CaSO_4$, and $MgSO_4$, which are present in groundwater, seawater, and rainwater, penetrate the concrete and chemically react with alkali and calcium ions to form a precipitate of $CaSO_4$ in addition to other forms of calcium- and sulfate-based compounds (e.g., ettringite). The manifestation of sulfate attack is cracking, increase in concrete porosity and permeability, loss of strength, and surface scaling. Surface scaling is generated by both the expansion associated with the formation of ettringite within the concrete and the internal pressure caused by the precipitation of calcium- and sulfate-based compounds inside the concrete pore network (Poe, 1998; NWTRB, 2010). Unlike the alkali sulfates, no decalcification of the calcium silicate hydrate phase occurs in the $CaSO_4$ attack. On the other hand, the $MgSO_4$ attack is significantly faster and more thorough than the attack by the other sulfate compounds because of the limited solubility of $Mg(OH)_2$ at the high concrete pH. In addition, magnesium ions present in deicing salts can react with calcium silicate hydrate, gradually converting it to magnesium silicate hydrate, which is not cementitious in nature.

According to NUREG–1557 (NRC, 1996a), the minimum threshold value of sulfates that can promote concrete degradation is 1,500 ppm with an environmental pH less than 5.5. Atkinson and Hearne (1990) developed a concrete service life model subject to sulfate attack using the external concentration of sulfate ions, concrete sorption properties, weather conditions, and concrete properties (i.e., elastic modulus, roughness factor, Poisson's ratio, and concrete porosity) as model inputs. The durability of concrete to sulfate attack can be enhanced by decreasing the water-to-cement ratio and using cements with a limited amount of tricalcium aluminates. Moreover, the use of pozzolanic admixtures and ground-granulated, blast-furnace slags can increase the life expectancy of concrete exposed to sulfates.

Another aggressive ion that can degrade concrete durability is chlorides. This degradation mechanism is well-established in the literature (Cheung et al., 2009). The presence of chlorides, as well as oxygen and moisture, can cause corrosion of steel in concrete (Tuutti, 1982). The presence of chlorides in concrete pore solution can also lower the concrete pH, disrupting the protective oxide layer of reinforcing steel and facilitating corrosion of the reinforcing steel. Corrosion of reinforcing steel in concrete due to chloride ingress can be interpreted in two stages: (i) initiation and (ii) propagation (Tuutti, 1982). The initiation stage corresponds to the time required for chloride ions to penetrate to the reinforcing steel surface and initiate corrosion once a threshold chloride concentration has been reached (Glass and Buenfeld, 1997). The length of the initiation stage depends on the concrete cover, surface chloride concentration, chloride diffusion coefficient, type of cementitious material, and the reinforcing steel material. The propagation stage relates to the ongoing corrosion of the reinforcing steel producing a decrease of the steel cross-sectional area and concrete cracking and spalling. Corrosion of the reinforcing steel in concrete is described in more detail in Section 4.9 of this report.

Regarding chloride ion concentration, it is well known that a critical concentration of internal chloride ion at the steel-concrete interface depends on the source of chloride, cement hydration products, pore water chemistry, water-to-cement ratio, cement type, concrete porosity, and curing conditions (Glass and Buenfeld, 1997; Angst et al., 2009). As a result, no agreement on a critical chloride ion concentration has been yet achieved. NUREG–1557 (NRC, 1996a) provides a threshold chloride concentration of 500 ppm with an environmental pH less than 5.5.

Unlike the sulfate attack, acids with a pH less than 3 can dissolve both hydrated and unhydrated cement compounds (e.g., calcium hydroxide, calcium silicate hydrates, and calcium aluminate hydrates) as well as calcareous aggregate in concrete without any significant expansion reaction (Gutt and Harrison, 1997; Mehta, 1986). In most cases, the chemical reaction forms water-soluble calcium compounds, which are then leached away by aqueous solutions. The dissolution of concrete commences at the surface and propagates inward as the concrete is being degraded. The signs of acidic attack are loss of alkalinity (also disturbing of electrochemical passive conditions for the embedded steel reinforcement), loss of materials, loss of strength, and rigidity. These degradation effects are gradual (Regourd, 1981). If not mitigated, strong acids may completely neutralize the alkalinity of the concrete pore water. Dorner (2002) proposed a model to predict the degradation of concrete under acid attack at pH between 4.0 and 6.5 as a function of time. Dorner's model simulates the release of calcium, iron, and aluminum ions into the concrete pore solution and the diffusion of these ions through the degraded layer into the acid boundary layer.

The extent and rate of concrete degradation due to acid attack depends on the type, concentration, and pH of the acidic solution, concrete permeability, calcium content in the cement, the level of water movement near the concrete surface, the water-cement ratio, and the type of cement and mineral admixtures (Pavlik and Uncik, 1997). For instance, sulfuric acid is particularly aggressive to concrete because the calcium sulfate formed from the acid reaction will also deteriorate concrete via sulfate attack (Pavlik, 1994). Even slightly acidic solutions that are lime deficient can attack concrete by dissolving calcium from the paste, leaving behind a deteriorated paste consisting primarily of silica gel. Acid rain containing SO_2, NO_x, and HCl can significantly compromise the durability of concrete (Webster and Kukacka, 2009). A deterioration model of acid rain was proposed by Ueda et al. (2001). In his model, the deterioration of the concrete is dependent on the amount of acid absorption into the concrete, mix proportion, and contact time or interval of rainfalls, and can effectively predict the depth of concrete damage. According to Zivica and Bajza (2002), blended Portland cements with pozzolans and slags are considered to be more resistant to acidic attack. Unlike limestone and dolomitic aggregates, siliceous aggregates are acid resistant and are sometimes specified to improve the chemical resistance of concrete (Huttl and Hillemeier, 2000; Neville, 1997). It should also be considered that the efficiency of a concrete against acid may be affected by other factors; for instance, the type of cement used for blending, the amount and fineness of pozzolan admixture, and curing conditions (Mehta, 1985).

NRC (2014a) provides more detailed information on the mechanics of sulfate attack on concrete and the knowledge gap assessments.

4.3.2 Summary of Workshop Key Findings and Expert Panel Assessment

Dr. Popovics identified magnesium ion attack as a degradation phenomenon that affects structures where magnesium chloride is used as deicing agent, such as bridge decks. Magnesium ions can also be present in marine environments, seawater, and groundwater. Dr. Berke indicated that magnesium ions could be present in dolomite aggregates and in concretes with calcium depletion, where magnesium ions can rapidly replace calcium ions in the silica hydrate compounds, weakening the concrete. In groundwater, magnesium ions are commonly found in the form of magnesium sulfate, especially in the western part of the United States. For these reasons, Drs. Berke and Popovics mentioned that the magnesium ion attack is most prevalent in certain geographical areas and more common for below-grade structures, unless magnesium-rich deicing salts are used nearby, such as at locations on the east coast of United States.

Drs. Popovics and Berke did not come to a conclusion about the threshold concentration of magnesium ion in groundwater needed to promote damage of below-grade concrete structures. Dr. Xi indicated that sulfate concentration, not magnesium concentration, is commonly measured in the field. According to Dr. Berke, the magnesium ion issue could be addressed by dewatering (lowering the water table) around the structure so that the amount of magnesium could be diminished, as discussed in Chapter 7 of this report. The magnesium ion attack can be identified by concrete coring.

Dr. Popovics recognized the presence of acids as another major contributor to concrete degradation. By nature, the concrete pore solution has a very high pH. Dr. Popovics indicated that the presence of acids can promote reactions with calcium hydroxide, decreasing the overall pH of the concrete, thereby potentially enhancing corrosion of the reinforcing steel and increasing the concrete porosity. The acids can also promote instability of the concrete hydration products. Typically, for pH <5.5 (NRC, 2010b), acid attack seems to be more significant, but this pH value depends on the type of acid. For instance, Dr. Berke noted that phosphoric acid can be highly aggressive even in a relatively high pH.

Dr. Berke indicated that the cyclic wetting and drying of the acids can cause more degradation, accelerating the diffusion of ionic species. Dr. Popovics mentioned that acids will likely come from groundwater but can also come from acidic rainwaters (with pH<4) in polluted regions with

SO_x and NO_x species. For this case, salt beds can be extremely acidic and damaging to concrete. Other ionic species that can damage the concrete are cations with divalent charges and anions, such as phosphates, ammonia, perchlorates, and any halide ions, commonly present in fertilizers. In particular, the halide ions (e.g., chlorides, bromide) can promote corrosion of the reinforcing steel. The rate of diffusion of certain halide ions, such as bromide and iodide, is substantially less than that of chlorides. Corrosion of the steel reinforcement in concrete due to chloride attack is addressed in Section 4.9 of this report.

4.3.3 Considerations for Mitigation and Inspection

Drs. Popovics and Berke indicated that the primary approach to prevent acid or aggressive ion attack is to make DCSSs resistant to the ingress of water bearing the deleterious species. A licensee may propose to accomplish this by coating or sealing of the system, as described in Chapter 5. The heat load from the emplaced fuel may contribute to drying of the concrete for a period of time, but will decay over time. Alternatively, cathodic protection uses electrochemical principles to protect the reinforcing bar from attack by modifying the galvanic characteristics of the reinforcement. The reinforcing bar is normally protected by a passive film in the high pH condition of the concrete. If the passive film is broken by the corrosive species, the anodic reaction on the rebar could cause localized corrosion damage. Cathodic protection systems, also discussed in further detail in Chapter 5, use embedded electrodes connected to external power supplies to shift the anodic reactions away from the reinforcing bar. Finally, for below-grade structures, drawing down the water table may be a suitable approach, according to Dr. Berke.

Acid attack (for instance, as caused by acidic rain) is similar to freeze-thaw and salt scaling degradation, in that it progresses from the surface of the concrete toward the interior. On the other hand, ion attack could initiate within the bulk of the concrete, such as at the rebar-concrete interface. In this case, visual examination of the type described in the aging management programs (AMPs) for reinforced concrete structures described in NUREG–1927, Revision 1 (NRC, 2015c) may not detect the degradation until it manifests at the outside surface by cracking, spalling, staining, or loss of concrete cover. A licensee would need to evaluate the extent of the condition in accordance with its corrective action program (CAP), determine whether the system can still perform its intended safety function, and determine what repairs are necessary. For the acid and ion attack, groundwater or soil monitoring is a useful approach to assess whether there are species in proximity to the DCSS that could promote this form of degradation, particularly for below-grade areas that are not readily accessible for visual inspection. The AMP in NUREG–1927, Revision 1 (NRC, 2015c) describes a program to specifically measure for pH, chloride content, and sulfate content, consistent with the acceptance criteria in the ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL (ASME, 2001). Considerations for the implementation of a groundwater or soil monitoring program are discussed in further detail in Chapter 6 of this report.

4.4 <u>Alkali-Silica Reaction</u>

4.4.1 Technical Background

There are two types of alkali-aggregate reactivity: (i) alkali-silica reaction (ASR) and (ii) alkali-carbonate reactivity. ASR is the most common and damaging: it is a chemical reaction between hydroxyl ions in the alkaline cement pore solution in the concrete and reactive forms of silica present in some aggregates (e.g., opal, chert, chalcedony, tridymite, cristobalite, strained quartz). An aggregate that presents a large surface area for reaction (i.e., poorly crystalline, amorphous, glassy) is susceptible to ASR (Poole, 1992). The resulting chemical reaction produces an alkali-silica gel that swells with the absorption of moisture, and this swelling exerts an expansive pressure within the concrete (Figg, 1987), resulting in internal damage that manifests as characteristic map cracking on the surface concrete (ACI, 1998). The internal damage results in degradation of concrete mechanical properties, and in severe cases, the expansion can result in undesirable dimensional changes and popouts. In reinforced concrete, cracks tend to align parallel to the direction of maximum restraint and rarely progress below the level of the reinforcement.

There is general agreement in the acting chemical reaction governing ASR. When poorly crystalline hydrous silica is exposed to a highly alkaline solution, there is an acid-base reaction between the hydroxyl ions and the acidic silanol (Si-OH) groups on the surface of silica (Dent Glasser and Kataoka, 1981). As additional hydroxyl ions penetrate the concrete, some of the siloxane groups (Si-O-Si) are also dissolved. The disruption of the siloxane groups ultimately weakens the structure. On the other hand, the ASR gel expansion mechanisms are subject to much debate. For instance, Hansen (1944) proposed an osmotic theory, where an increasing hydrostatic pressure is exerted on the cement, leading to cracking of the surrounding mortar. McGowan and Vivian (1952) disputed the osmotic theory, stating that cracking of the surrounding cement would relieve hydraulic pressure and prevent further expansion. Instead, McGowan and Vivian (1952) and later Tang (1981) proposed that water can be absorbed physically into the alkali-silica gel, resulting in swelling of the gel. Diamond (1989) proposed that, in the absence of calcium, silica simply dissolves in alkali-hydroxide solution and does not form alkali-silicate gel. The presence of calcium resulted in a reaction product similar in structure to ASR gel. Thomas et al. (1991) found that gels that are low in calcium and high in alkali are relatively fluid and readily dispersed into cement paste, whereas gels higher in calcium are more viscous and less able to dissipate when they swell in contact with water.

The primary factors influencing the initiation and propagation of ASR include (i) a sufficiently high alkali content of the cement (or alkali from other sources such as deicing salts, seawater, and groundwater), (ii) a reactive aggregate, and (iii) available moisture, generally accepted to be relative humidity greater than 80 percent (Pedneault, 1996; Stark, 1991). A study by the California Department of Transportation (Glauz et al., 1996) revealed that ASR increases proportionally to the cement content, alkali content greater than 0.6 percent can accelerate ASR, high calcium oxide content can promote ASR, and the use of various types of admixtures in certain doses can mitigate ASR [ACI 221.1R (ACI, 1998); ASTM C618 (ASTM, 1998)]. At higher concentrations of alkali hydroxides, even the more stable forms of silica are susceptible to ASR attack (Xu, 1987). Repeated cycles of wetting and drying can accelerate ASR [ACI 221.1R (ACI, 1998)]. As a result, it is desirable to minimize both available moisture and wet-dry cycles by providing good drainage. Moreover, concrete exposed to warm environments is more susceptible to ASR than that exposed to colder environments (Perenchio et al., 1991).

Typically, the rate of ASR deterioration is slow, so that the risk of catastrophic concrete failure is low. In general, ASR can cause serviceability issues and can also exacerbate other deterioration mechanisms. The degradation of concrete due to ASR is operative for both the short and long term as long as the primary factors for it occurring are present. NRC (2014a) provides more detailed information on the mechanics of ASR; knowledge gap assessments; and operating experience, such as the ASR degradation of the concrete in the Seabrook reactor containment structure.

4.4.2 Summary of Workshop Key Findings and Expert Panel Assessment

Three prerequisites are needed to initiate ASR according to Drs. Berke and Xi: (i) aggregate reactivity, (ii) moisture level, and (iii) concentration of alkali in the cement. Without any one of these three prerequisites, the ASR damage will likely not take place. For instance, if a given concrete structure remains dry for a certain period of time, no ASR will likely develop. Dr. Xi added that if external moisture enters the concrete, then ASR can develop if reactive aggregates and high alkali are present [ACI 221.1R (ACI, 1998)]. Dr. Berke indicated that ASR is generally a slow degradation mechanism: it takes 20 to 40 years for ASR degradation to be visible on the surface of the concrete. Dr. Berke indicated that ASR manifests as map cracking with formation of gel at the concrete cracks [ACI 221.1R (ACI, 1998)]. Drs. Jacobs and Berke noted that the ASR gel exuding from the aggregates can be detected through petrographic techniques [ASTM C295 (ASTM, 2012c); ASTM C856 (ASTM, 2011a)] on extracted samples before concrete cracking can be seen.

Drs. Jacobs and Berke noted that ASR is typically addressed through aggregate selection prior to construction. Several short-term screening tests [ASTM C1567 (ASTM, 2013a); ASTM C1260 (ASTM, 2014b); ASTM C289 (ASTM, 2007)] are available to assess the susceptibility of a particular aggregate to ASR, although these accelerated tests are sometimes not conclusive enough to completely disregard this degradation mechanism. The accelerated screening tests are less reliable when compared to long-term screening tests [ASTM C1293 (ASTM, 2008a); ASTM C227 (ASTM, 2010b)], which could give a definite answer on the aggregate reactivity and the initiation of ASR. However, these long-term tests are costly and time-consuming. Dr. Berke indicated that another approach to reduce the likelihood of developing ASR is by changing the concrete mix design to include Type F fly ash, silica fume, or slag to the concrete mix [ASTM C618 (ASTM, 1998); ASTM C441 (ASTM, 2011b)] to reduce the alkali content in the concrete [ACI 221.1R (ACI, 1998)].

In spite of all the screening tests and changes to the concrete mix design, Drs. Popovics and Berke and Mr. James were not confident that these approaches could be effectively used to completely eliminate the potential for ASR, especially for long-term exposures. For example, for large construction projects, the aggregates coming from several sources may have different reactivities. For most field applications, the ASR screening tests are not conducted on each aggregate source, but rather in selected aggregate batches. Mr. James noted that the concrete used at the Seabrook plant passed all industry standard ASR screening tests [ASTM C289 (ASTM, 2007); ASTM C295 (ASTM, 2012c)] at the time of construction, yet significant ASR has developed since that time. It was also noted that the assumed extended storage timeframe of 300 years adds uncertainty to the ability to evaluate aggregates for ASR susceptibility. A very slow reacting aggregate could potentially pass the current screening tests, but show degradation in the extended storage timeframe (NRC, 2011b). Specifically, the appendices of ASTM C289 (ASTM, 2007) and ASTM C1293 (ASTM, 2008a) stated that the tests described in ASTM C227 (ASTM, 2010b) and ASTM C289 (ASTM, 2007) may not accurately predict aggregate reactivity when dealing with late or slowly expanding aggregates containing strained or microcrystalline quartz.

4.4.3 Considerations for Mitigation and Inspection

Various methods to mitigate the potential for ASR are discussed in ACI 221.1R (ACI, 1998, Section 5), including guidelines for limiting moisture and, minimizing alkalis and selecting cement, aggregates, and chemical additives. Some of these guidelines are further detailed in Chapter 5 of this report. If a licensee were to credit any one or more of these approaches in its analysis of the potential for ASR during renewed licensing terms, it would need to provide sufficient information to confirm that the conditions under which the approach was found effective in laboratory tests or other field applications also held true for the DCSS.

The AMP for reinforced concrete structures in NUREG–1927, Revision 1 (NRC, 2015c) relies on periodic visual examination of accessible surfaces, as described in ACI 349.3R (ACI, 2010a), and groundwater monitoring consistent with the acceptance criteria in the ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL (ASME, 2001). ASR is likely to initiate by the chemical reactions within the internal mass of the structure before characteristic pattern cracking could be detected on the material surface. The structural significance of ASR is dependent on the nature of the concrete component affected by the degradation. For instance, Mr. James indicated that for mass concrete structures (different from DCSS structures), which are lightly reinforced, ASR can have very serious structural implications. On the other hand, for heavily reinforced structural members, ASR tends to be less significant. Should cracking be detected by visual inspection, a licensee would need to evaluate the extent of the condition in accordance with its CAP and determine whether the system could still perform its intended safety function and what repairs were necessary. The primary inspection challenge would be related to below-grade areas where partial removal of topsoil surrounding the concrete may be necessary to access the concrete surface. For such cases, a licensee may need a site-specific plan for below-grade inspections and/or to use opportunistic inspections.

If indications of ASR are observed on the surface of the structure, additional examinations may be warranted to evaluate the extent of damage within that structure or other structures on the same site with similar design characteristics. As discussed previously, the withdrawal of core samples should be used with caution. Nondestructive volumetric examination techniques—for instance, ultrasonic testing—may be able to detect subsurface voids or cracks, but have not yet been applied to DCSSs.

4.5 Thermal Desiccation

4.5.1 Technical Background

The concrete structure of DCSSs is exposed to sustained high temperatures for a long period of time due to the decaying heat of the spent fuel. Exposure of concrete to elevated temperature affects its mechanical and physical properties. It is well known that concretes can degrade at high temperatures due to dehydration reactions of the hydrated cement paste, thermal incompatibility between cement and aggregate, and likely physicochemical deterioration of the aggregates. As the temperature increases to about 105 °C [221 °F], all evaporable water is removed from the concrete. At temperatures above 105 °C [221 °F], the strongly absorbed and chemically combined water are gradually lost, with the dehydration essentially complete at 850 °C [1,562 °F] (Harmathy, 1970).

Phan and Carino (2000) established that the mechanical properties of concrete can be adversely affected by elevated temperature exposure. High-temperature degradation in concrete manifests as a change in compressive strength and stiffness as well as an increase in concrete shrinkage and transient creep with a consequent formation of cracks (Naus, 1981; Naus, 1988; Schneider et al., 1981) as discussed later. In the temperature range of 20 to 120 °C [68 to 248 °F], there is an insignificant concrete strength loss. The small strength loss is attributed to the thermal swelling of the physically bound water, which causes pressures. Between 120 and 200 °C [248 and 392 °F], concrete strength increases, resulting from an increase in the Van der Waals forces due to a compaction of the cement gel layers. Between

200 and 250 °C [392 and 482 °F], the compressive strength is nearly constant, followed by a rapid decrease in strength beyond 250 °C [482 °F]. An increase in temperature can also produce a significant and progressive increase in the strain corresponding to the peak stress.

The degree of concrete degradation with temperature depends on several factors, including concrete mixing, aggregate type, curing, loading condition, moisture retention and content, and exposure time (Carette and Malhotra, 1985). In general, compressive strength decreases further if moisture is not allowed to escape the concrete. In addition, concretes loaded during heating tend to lose less strength than unloaded concretes. The effect of the elevated temperature is most significant on the concrete modulus of elasticity, which can decrease up to 40 percent (Freskakis, 1979). As mentioned earlier, the effects of temperature on mix proportions and aggregate type play an important role in the concrete performance. For instance, low cement-aggregate mixes tend to lose less strength than others and limestone aggregate degrades less than siliceous aggregate.

ASME Boiler and Pressure Vessel Code, Section III, Division 2 (ASME, 2007b) and ACI 349-06 (ACI, 2007c) present a temperature limit for long-term heating exposure of concrete structures. By design, concrete structures should be kept below 65 °C [149 °F] to avoid mechanical deterioration. The French specification (France, 1970) for a prestressed concrete reactor vessel limits temperatures in active parts of the concrete to 90 °C [194 °F]. Permissible temperatures for the concrete in prestressed concrete reactor vessels for gas-cooled reactors have generally been limited to the range of 45 to 80 °C [113 to 176 °F] (Furste, 1973). However, NRC [NUREG–1536 (NRC, 2010a) and NUREG–1567 (NRC, 2000)] has endorsed ACI 349-06 (ACI, 2007c) temperature requirements for concrete structures of DCSSs, where no additional testing is required if temperatures of general and local areas do not exceed 93 °C [200 °F] during normal or off-normal conditions/occurrences. NRC (2014a) provides an additional detailed description of the mechanics of thermal degradation of concrete and the knowledge gap assessments.

4.5.2 Summary of Workshop Discussion and Expert Panel Assessment

Drs. Xi and Popovics agreed that thermal desiccation can cause shrinkage cracking or drying cracking, especially for restrained concrete structures where shrinkage stresses build up inside the concrete matrix. Dr. Xi indicated that the area of concern for DCSSs is likely to be the interior, because temperatures are greater than the outside concrete surface. The temperature differential between the inside and outside of the structure can create tensile stresses in the concrete somewhere through the wall thickness. Dr. Xi also noted that temperature-dependent concrete differential expansion has been considered as a main driving force for the formation of delamination cracks in concrete water tanks in Canada.

Mr. James indicated that from the structural point of view, the thermal desiccation effects for DCSSs should already be accounted for in the design basis. Drs. Popovics and Berke agreed that shrinkage cracking, in itself, is unlikely to affect the structural behavior of the concrete. However, Dr. Berke noted that shrinkage cracking may have implications on other degradation mechanisms; for instance, by making the structure more susceptible for ion ingress, especially in industrial and marine environments.

A moisture gradient is likely to be related to the temperature gradient through the concrete wall thickness. Dr. Xi indicated that there may be a moisture profile where the outside concrete surface will be saturated at high relative humidity's (RHs) while the interior concrete surface will remain dry. As such, moisture will move from the outside to the inside of the concrete, potentially leading to enhanced transportation of other aggressive ions inside the concrete. In addition, Dr. Popovics mentioned that the movement of physically and chemically bound water due to the sustained temperature effect can promote microstructural changes in concrete, such as changes in concrete porosity and capillary pores. However, Dr. Popovics indicated that this change in concrete microstructure does not promote a significant increase in concrete porosity.

4.5.3 Considerations for Mitigation and Inspection

Thermal desiccation of DCSS structures is most readily managed by system design to ensure that the concrete is not exposed to excessive temperatures for a duration that could cause deleterious changes in the material properties. As such, the AMP for reinforced concrete structures in NUREG–1927, Revision 1 (NRC, 2015c) states that the aging effects of thermal degradation do not need to be specifically managed if an analysis can be made to demonstrate that no part of the concrete exceeds the temperature limits specified in ACI 349-06 (ACI, 2007c) of 65 °C [149 °F] generally and 93.3 °C [200 °F] for localized areas. As discussed in Chapter 2 of this report, concrete used for overpacks or storage modules may be metal lined or protected by heat shields—in part, for thermal management. Further, the inlets and outlets for ventilated systems are typically checked daily to ensure that there are no blockages to interfere with passive cooling flow.

If thermal desiccation is not excluded by engineering analysis, the licensee could demonstrate that indications of degradation in the structure would be detected by an inspection or monitoring program prior to loss of safety function. Features such as cracking or spalling may be detected by periodic visual examination in accordance with ACI 349.3R (ACI, 2010a). Loss of concrete strength or modulus may not be directly measured without the removal of concrete cores for mechanical testing, though as discussed in prior sections, caution should be exercised when coring concrete structures.

4.6 Creep

4.6.1 Technical Background

Creep in concrete is the time-dependent deformation resulting from sustained load. Cement paste in concrete exhibits creep due to its porous structure and a large internal surface area that is sensitive to water movements (NRC, 2014a). Creep manifests as cracking extending on the concrete outer surfaces and causes redistributions of internal forces. In the case of a given concrete mix design, the age of concrete and the sustained loading period are the primary factors that determine the magnitude of the creep of concrete (Neville and Dilger, 1970). Creep is also a function of applied stress. For instance, at constant temperature and water content, creep varies linearly with stress up to 0.4 of the compressive strength. Under stresses exceeding 0.4 of the compressive strength, creep becomes progressively nonlinear with stress (Freudenthal and Roll, 1958). In addition, after unloading, creep is partly irreversible. When concrete is drying simultaneously with creep, creep is accelerated (Pickett, 1942). More importantly, creep rate grows with temperature (McDonald, 1972).

Traditionally, the mechanism of concrete creep has been separated into two superposed phenomena: (i) basic creep and (ii) drying creep. Basic creep is the time-dependent deformation under constant load and at constant humidity conditions. Basic creep is primarily influenced by the material properties (e.g., composition and fineness of cement, mineral composition of aggregates) (Wang and Salmon, 1998; Tamtsia and Beaudoin, 2000). Drying creep is defined as the deformation that exceeds the basic creep strain observed when the same material is exposed to drying while under load (Acker and Ulm, 2001). Drying creep depends on the environment and the size of the concrete structure. It has been suggested that basic creep shows two well-defined stages: (i) short-term creep kinetics, taking place during the first days after the application of a load and (ii) long-term creep kinetics. Bazant et al. (1997) proposed models based on the solidification theory for short-term aging and the microprestress-solidification theory. These models are based on the assumption that aging occurs over the short term as a result of the solidification and deposition of stress-free hydration product layers in the pore walls. Long-term creep strains are justified by the theory of relaxation of stresses at the microscopic level. Several mechanisms have been proposed to explain the drying creep. However, the mechanisms behind drying creep are still not well understood. One approach to address drying creep is by assuming that the total drying creep strain comprises (i) a structural part, corresponding to the drying-induced microcracking resulting from the applied compressive load, and (ii) an intrinsic part, resulting from the internal physicochemical mechanisms caused by the drying process (Bazant and Xi, 1994). NRC (2014a) provides some detailed information on the mechanics of concrete creep and knowledge gap assessments.

4.6.2 Summary of Workshop Discussion and Expert Panel Assessment

Dr. Xi indicated that creep depends on two main independent factors: (i) the age of the concrete and (ii) the sustained loading period. The concrete's ability to creep lessens with age. Drs. Xi and Popovics indicated that if a constant sustained load is applied on 2-year-old concrete and 140-year-old concrete, the 2-year-old concrete will exhibit significantly more creep. Therefore, there is a connection between the age of concrete and the sustained loading on it.

The load changes during the life of the concrete structures could lead to creep for both the short and the long terms. However, Dr. Berke indicated that for a DCSS, load may not change significantly after the storage casks are in place. The common agreement is that creep should not be ignored from the point of view of the component functionality, according to Mr. James. However, if creep does not manifest within the first 2 years of exposure, Drs. Berke and Popovics and Mr. James pointed out that this degradation mechanism will likely not be an issue for extended exposure times unless there is an unusual change in load, such as an accumulation of 6.1 m [20 ft] of extra snow load.

Dr. Berke indicated that during the design of concrete structures, a certain amount of concrete creep is allowed based on the design basis. Dr. Xi mentioned that in some cases creep has beneficial effects, such as the release of stress concentrations.

4.6.3 Considerations for Mitigation and Inspection

As mentioned previously, concrete creep is generally understood to be a phenomenon that would affect concrete structures early in the service life, and only then when there is sustained loading placed upon the system. It could largely be mitigated by proper design practices in accordance with ACI 318-05 (ACI, 2005a) or ACI 349-06 (ACI, 2007c). If there is no evidence of creep-related degradation within the initial licensing term for the DCSS, it is not expected that it would require aging management during the renewed licensing term. Cracking resulting from creep can be identified by periodic visual inspection, as described in ACI 349.3R (ACI, 2010a) and discussed in further detail in Chapter 6. Any detected cracks would be analyzed and repaired according to the licensee's CAP.

4.7 Delayed Ettringite Formation

4.7.1 Technical Background

At the initial stage of fresh concrete curing, ettringite, commonly referred to as naturally occurring ettringite, is formed by the reaction of tri-calcium aluminate and gypsum in the presence of water. This reaction ends as soon as the sulfate concentration in concrete reaches a minimum value (2.5–3.0 percent as SO_3) (Michaud and Suderman, 1997). The formation of naturally occurring ettringite in fresh concrete is not detrimental to the overall concrete performance. At this still early stage of concrete curing, the naturally occurring ettringite converts to monosulfoaluminate, if curing temperatures are greater than about 70 °C [about 158 °F]. The latter conversion is an expansive reaction. However, the volume expansion can be accommodated without any detrimental effects because the concrete is still in a fresh state. At this stage at a temperature above 70 °C [about 158 °F], the calcium silicate hydrate (C-S-H) gel adsorbs the free sulfate and monosulfoaluminate (Kalousek and Adams, 1951). After concrete hardens, if the temperature decreases below about 70 °C [about 158 °F], the monosulfoaluminate becomes unstable, and in the presence of sulfates released by the C-S-H, ettringite will reform [called delayed ettringite formation (DEF)], resulting in volume expansion and increasing internal pressures (Fu, 1996). Because the concrete has hardened at that stage, this volume expansion leads to cracking and spalling.

The conditions necessary for the occurrence of DEF are excessive temperatures during concrete casting, the presence of internal sulfates, and a moist environment. Limiting the internal concrete temperature to about 70 °C [about 158 °F] during casting can mitigate the formation of DEF (Day, 1992; Famy, 1999; Taylor, 1994). This can be achieved either by direct specification, or indirectly by limiting the cement content or specifying the use of low or very low heat cement. Data have indicated that use of air entrainment reduces expansions as compared to non-air-entrained mortars (Day, 1992). This observation is indirectly supported by the fact that there have been no reported field cases of DEF in adequately air-entrained concretes. Supplementary cementing materials (e.g., pozzolans and slag) have also been demonstrated to reduce the potential for deleterious expansion due to DEF.

NRC (2014a) did not identify DEF as a potential degradation mechanism for concrete in DCSSs. Instead, this degradation mechanism was proposed by the expert panel, as there is evidence of such concrete damage for other concrete applications.

4.7.2 Summary of Workshop Discussion and Expert Panel Assessment

Dr. Berke mentioned that DEF causes expansion of concrete with consequent distributed internal cracking of concrete. The expansion mechanism is associated with the conversion of ettringite, formed during the initial concrete hydration process, to monosulfate when concrete temperature exceeds 65 °C [140 °F] and the back conversion to ettringite as the temperature of the concrete drops below 65 °C [140 °F]. This reconversion of ettringite causes an increase in concrete volume.

Dr. Berke indicated that to control this degradation mechanism, the temperature of the concrete during curing has to be maintained below 65 °C [140 °F]. Dr. Berke added that this threshold temperature can be exceeded in certain areas of the DCSS, especially early on when the heat load is high, so that reformation of ettringite will not occur. As time progresses, the cask is expected to cool down to the temperature of the concrete. Dr. Berke indicated that the reformation of ettringite in concrete would take place at areas in the concrete where the temperature is below 65 °C [140 °F], promoting internal concrete cracking.

Dr. Berke pointed out that the internal distributed cracking will affect the concrete strength. The panelists indicated that the cracking morphology is similar to cracking promoted by ASR. However, Drs. Berke and Popovics were not sure whether the amount of cracking generated by DEF would be contained by the concrete and the reinforcing steel of the structure. As Dr. Berke indicated, this degradation mechanism is not expected to occur during concrete casting but rather during exposure to transient high and low internal temperatures in the DCSS.

Dr. Berke indicated that DEF degradation has been observed in concrete beams in Texas where the exposure temperatures were above the critical value. On the other hand, Drs. Popovics and Berke indicated that the DEF degradation mechanism would not have significant implications in DCSSs for either short or long terms when compared to other degradation modes.

4.7.3 Considerations for Mitigation and Inspection

DEF can largely be prevented by measures to control the curing temperature and the concrete composition. With respect to the curing temperature, ACI 318-05 (ACI, 2005a) refers to recommended curing following ACI 308R (ACI, 2008b). ACI 305.1 (ACI, 2014) and ACI 305R (ACI, 2010b) address best practices for controlling curing temperature, particularly at high ambient temperatures, and specify a temperature limit of 43 °C [95 °F] for fresh concrete at the time of discharge. Though not directly applicable to DCSSs, some states provide standard specifications or special provisions for concrete temperature limits during initial construction for roadways. In general, the maximum concrete temperature during curing is limited to 71–80 °C [160–176 °F] (Chini et al., 2003, Tables 3.2 and 3.3). Additional preventative actions include limiting the SO_3 content in concrete to 0.5 percent, establishing a fineness limit for Type III cement, and limiting calcium sulfate in hydrated Portland cement when the total SO_3 content of the cement exceeds 3 percent.

Considerations for inspection approaches to detect damage caused by DEF are largely similar to those which would apply to ASR or other mechanisms that could initiate within the internal mass of the structure prior to becoming evident on the surface. Periodic visual examination of accessible surfaces, in accordance with ACI 349.3R (ACI, 2010a), is thought to be adequate to ensure that there is no loss of safety function. If indications of DEF are observed on the surface of the structure, they should be dispositioned in accordance with the licensee's CAP. Additional examinations may be warranted to evaluate the extent of damage within that structure or within others on the same site with similar design characteristics as the affected structure.

4.8 Radiation

4.8.1 Technical Background

In general, the radiation effects on the concrete properties depend on the intensity of the radiation field, temperature, and exposure period. Investigations conducted in the late 70s are still used as the basis for concrete degradation due to radiation (Hilsdorf et al., 1978). It has been demonstrated that gamma radiation can decompose and evaporate water in concrete (Bouniol and Aspart, 1998). Because most of the water is contained in the cement paste, the effect of gamma radiation on cement paste is more significant than its effect on the aggregates. The gamma radiation can also decompose the SiO bond of calcium silicate hydrate (Kontani et al., 2010). Deterioration of concrete due to neutron radiation involves a reduction of concrete stiffness, formation of cracks due to swelling, and changes of the microstructure of aggregates with a consequent reduction in concrete strength (Kontani et al., 2010). The changes in aggregate microstructure can lead to higher reactivity of aggregates to certain aggressive chemicals.

In assessing the reduction of concrete strength and elastic modulus under irradiation, 1×10^{19} n/cm^2 for fast neutrons (neutron energy >1 MeV) and 2×10^{10} rad [2×10^8 Gy] for gamma rays have been used as the critical levels of radiation (Hilsdorf et al., 1978). In an Electric Power Research Institute (EPRI) report (EPRI, 2012a), the threshold value for gamma rays was considered as 1×10^{10} rad [1×10^8 Gy]. IAEA-TECDOC-1025 (IAEA, 1998) lists 1×10^{19} n/cm^2 for fast neutrons and 1×10^{10} rad [1×10^8 Gy] for gamma rays. Concrete structures have been regarded to be sound as long as the cumulative levels of radiation do not exceed the critical levels over the life of the structure. The decrease in compressive strength is most pronounced at neutron fluence values greater than $1–2 \times 10^{19}$ n/cm^2, where the residual compressive strength is lower bounded by about 50 percent of the initial strength at about 1×10^{20} n/cm^2 (Field et al., 2015). Soo and Milian (2001) found that loss of compressive strength could occur at gamma doses less than the threshold dose of 2×10^{10} rad [2×10^8 Gy] noted in Kontani et al. (2010). Soo and Milian (2001) postulated that the loss of strength could be connected to the radiolysis of the water of hydration in the cement as well as to pore water. A loss of hydrogen and oxygen radiolytic species during irradiation would decrease the level of cement hydration and thus the strength of the cement. In conclusion, the expected fluence levels before concrete degradation remain a matter of discussion. Similar to the concrete strength, the stiffness or modulus of elasticity of concrete is reduced under radiation. Neutron fluence of approximately 1×10^{19} n/cm^2 can lead to a decrease of the modulus of elasticity. When the fluence exceeded 5×10^{19} n/cm^2, the reduction was as high as 30 percent of the original stiffness.

The heat of radiation affects properties of concrete at two levels: (i) the structural level and (ii) the microstructural level. At the structural level, the thermal gradient due to the heat of radiation results in thermal stress, which may be high enough to create damage in concrete. At the microstructural level, the mismatch of thermal strains in cement paste and in aggregate responding to the heat of radiation may cause a large stress at the interface between the aggregate and cement paste, which in turn may cause cracking in the cement paste. Lowinska-Kluge and Piszora (2008) conducted a study on the microstructure of cement paste under various doses of gamma radiation. The microstructure of cement paste was changed significantly by the gamma radiation after 1.3×10^{10} rad [1.3×10^8 Gy]. With increasing dose, microcracks appeared on the cement paste. This study indicates that radiation affects the microstructure of cement paste. NRC reports (NRC, 2014a, 2013) provide knowledge gap assessments, information on the mechanics of concrete radiation damage, and current understanding of the effects of radiation on concrete, including:

- the effects of neutron and gamma radiation on the mechanical and physical properties of concrete
- thermal effects due to gamma heating of the concrete
- coupling effects of mechanical loading, thermal effect, moisture content, and radiation

4.8.2 Summary of Workshop Discussion and Expert Panel Assessment

Drs. Popovics and Jacobs indicated that the concrete performance cannot be assessed by the total radiation fluence limits, because those fluence limits are not well understood or well established in the literature. For instance, Hilsdorf et al. (1978) cited some radiation levels as critical levels of fluence above which concrete is degraded. On the other hand, the ACI 349.3R (ACI, 2010a) standard established the ultimate lifetime limit of a concrete structure as a total radiation fluence to be met. Dr. Popovics indicated that the literature information related to the threshold radiation fluence limits is partially conflicting, because the approach used to collect the radiation threshold values is not consistent among the published studies. To that end, Drs. Popovics, Berke, and Xi believe that the radiation limits established by ACI 349.3R (ACI, 2010a) may be overly conservative based on comparable literature data.

Dr. Xi mentioned that within a 40-year period, the fluence limits established by the ACI 349.3R (ACI, 2010a) standard are adequate. However, Dr. Xi is uncertain whether the fluence limits can be extrapolated to a 300-year timeframe. Dr. Xi indicated that several investigations conducted on the performance of concrete under radiation are not reliable, because of the difference in aggregates and cements used in those studies. The panelists indicated that the gamma radiation dose can promote concrete degradation after 100 years of exposure.

Dr. Xi also pointed out that both gamma and neutron radiations should be considered to assess the concrete performance and the total radiation fluence limits because the effects of both neutron and gamma radiation cannot be separated. Likewise, there are other factors influencing the radiation damage, such as the concrete temperature and the type of aggregates, as mentioned by Dr. Jacobs. Thus, the effect of the temperature should be accounted for when evaluating the effects of radiation. Dr. Xi suggested that radiation may affect other degradation modes of concrete, such as those from the freeze and thaw mechanism and from alkali-silica reaction. These two mechanisms have been discussed in Sections 4.1 and 4.7 of this report, respectively.

4.8.3 Considerations for Inspection and Damage Analysis

The effects of irradiation-induced damage on concrete structures are not likely to require aging management if engineering analyses can be used to demonstrate that the fluence experienced by these structures is well below that which is known to be deleterious. The AMP for reinforced concrete structures in NUREG–1927, Revision 1 (NRC, 2015c) cites the critical cumulative fluence from ACI 349.3R (ACI, 2010a) of 10^{17} neutrons/m^2 or 10^{10} rad gamma. If irradiation effects could not be excluded by engineering analysis, the licensee may need to demonstrate that indications of degradation in the structure would be detected by an inspection or monitoring program prior to loss of safety function. Features such as cracking or spalling may be detected by periodic visual examination in accordance with ACI 349.3R (ACI, 2010a). Loss of concrete strength or modulus may not be directly measured without the removal of concrete cores for mechanical testing. As discussed in prior sections, caution should be exercised when coring concrete structures, as the process itself could damage the structure or place it into an unanalyzed condition. It should not be used as a purely exploratory approach.

4.9 Rebar Corrosion

4.9.1 Technical Background

Corrosion of the reinforcing steel in concrete is mainly caused by the presence of chloride ions in the concrete pore solution and carbonation of the concrete. Corrosion of steel in concrete was briefly discussed in Section 4.3, where the main emphasis was on the effects of aggressive ions or acids on the degradation of concrete. In this section, a more thorough discussion on steel corrosion is presented. ACI 222R (ACI, 2001) provides comprehensive discussion and methods to prevent corrosion of reinforcing bar in concrete.

The highly alkaline environment provided by the concrete (normally with pore water pH>13.0) results in the formation of a metal-adherent oxide film on the reinforcement steel bar surface, which passivates the steel (Page, 1982). However, chloride ions may penetrate the concrete matrix and break down the steel passive layer, triggering a high corrosion rate and shortening the service life of a concrete structure. When the chloride concentration at the reinforcing steel surface exceeds a threshold value as discussed later, the protective passive film on the steel surface is disrupted, promoting active corrosion of the reinforcing steel. The presence of corrosion products at the steel surface can generate internal stresses within the concrete matrix, causing cracks and spalling of the concrete cover with consequent structural damage.

Chlorides may be present in concrete by several means. For instance, chlorides may already exist at low levels within the base mix constituents, or may be introduced when adding a chloride-containing admixture (e.g., calcium chloride) or when using salt water as mixing water. In many practical situations, chloride ions penetrate from the outside environment, such as when using deicing salts or when in marine service (Tang and Sandberg, 1996).

The actual threshold chloride concentration in concrete required to promote corrosion of the reinforcing steel depends on the pore solution pH of the concrete. The onset of corrosion can be enhanced by a reduction of the concrete pH at the steel surface through a mechanism called carbonation. Thus, the chloride-to-hydroxide ratio is an important parameter in evaluating the steel corrosion. Glass and Buenfeld (1997) have reviewed the chloride threshold values reported for steel embedded in concrete structures. From this investigation, it was concluded that a universal, well-defined chloride threshold value does not exist. The lowest limit of chloride threshold value in concrete ranged from 0.2 to 2.5 percent (by weight of cement). Factors such as the chemical composition of the rebar, as well as its surface roughness, can influence the chloride threshold (Szklarska-Smialowska, 1986).

Concrete carbonation results from the chemical reaction between the hydrated cement components [notably the $Ca(OH)_2$ but also the C-S-H and other compounds] and the atmospheric carbon dioxide. As mentioned previously, carbonation causes a reduction in concrete pH, which affects the passivity of the reinforcing steel, accelerating corrosion if the threshold chloride concentration at the steel is reached. Carbonation rate depends on the external CO_2 concentration, concrete type, temperature, time of wetness of the concrete surface, and degree of moisture. Typically, carbonation in concrete can progress at a rate on the order of 1–2 mm [0.039–0.078 in] per year in sheltered environments (Bertolini et al., 2004).

Tuutti (1982) established the initiation-propagation concept to predict the service life of concrete structures. In the corrosion initiation period, chloride concentration builds up around the rebar until a critical concentration is reached. The corrosion propagation period starts when chloride exceeds the critical concentration and continues until visible structural damage or a similar condition is observed and the structure is deemed to be in need of repair. Design improvements for durability have often focused on extending the length of the initiation stage (rather than that of the relatively shorter propagation period), using mixtures that make concrete more impermeable to chloride ions (Berke and Hicks, 1992).

The minimum chloride threshold value in groundwater causing concrete degradation is 500 ppm with a pH less than 5.5 (NRC, 1996a). However, the actual threshold chloride concentration in concrete required to promote corrosion of the reinforcing steel depends on the pore solution pH of the concrete, water-to-cement ratio, cement type, concrete porosity, and curing conditions (Glass and Buenfeld, 1997).

NRC (2014a) provides more detailed information on the mechanics of corrosion of embedded steel, degradation of concrete structure due to corrosion of embedded steel, and knowledge gap assessments.

4.9.2 Summary of Workshop Discussion and Expert Panel Assessment

Corrosion of the embedded steel reinforcement is a well-known degradation mode. Field experience from other applications (e.g., bridges), where corrosion of steel in concrete is often found, helps determine the parameters commonly associated with corrosion. Drs. Popovics and Berke indicated that any halide (e.g., chlorides, bromide) ions will promote corrosion. Dr. Berke mentioned that among all the halide ions or halide-type compounds, chlorides are the worst because those ions can diffuse at a faster pace through concrete. Dr. Berke added that in addition to the presence of aggressive ions, the amount of moisture in the concrete is essential for corrosion initiation and propagation. Dr. Berke also indicated that 70–80 percent RH inside the concrete is sufficient to initiate corrosion. In addition, at the onset of corrosion, a critical amount of chlorides is needed next to the embedded steel according to Dr. Berke. Once initiated, the rate of corrosion will be dependent on the available oxygen in the concrete.

Dr. Berke indicated that some corrosion of the reinforcing steel helps the structural aspects of the concrete before the appearance of concrete cracks, delamination, or spalling. However, Dr. Berke indicated that once corrosion starts, mitigation plans have to be implemented. Dr. Berke also explained the corrosion of steel in terms of the Pourbaix diagram, which relates the stability of different iron phases as a function of pH and temperature. At high pH and high temperature, iron is converted to iron oxide on the surface of the steel, which has no passivating properties, leading to high corrosion rates.

Dr. Berke indicated that the expansive corrosion products (of composition Fe_3O_4 or FeOOH) formed on the embedded steel will promote cracking of the concrete, with potential delamination, spalling, and swelling if no corrective actions are implemented. In a concrete with low oxygen content, the corrosion products [typically of the form $Fe(OH)_2$] formed are not expansive in nature so that no cracking or swelling of concrete can be noted, according to Dr. Berke. However, corrosion of the embedded steel can progress to reduce the steel cross section to the point that the steel may compromise the load capacity of the structure. Dr. Berke believes that this condition of low oxygen content will likely not occur in DCSSs because oxygen availability is high. However, the high temperature exposures under the presence of moisture may create conditions of accelerated corrosion.

Drs. Berke and Popovics also pointed out that carbonation of concrete can promote corrosion of steel in concrete by dropping the concrete pH and depassivating the steel. However, carbonation is a slower process than the chloride-induced corrosion. Dr. Berke indicated that carbonation is more uniform in nature, thus, the concrete takes longer to develop cracks. Dr. Berke further mentioned that enhanced carbonation has been recorded when the RH inside the concrete is between 40 and 70 percent.

Dr. Berke attempted to predict the effects of corrosion of steel in concrete. For reasonably good concrete cover exposed to marine-type chlorides, the expected durability could reach 60 years. On the other hand, Dr. Berke is not certain whether a concrete can attain 300 years of service life, because carbonation may play a role in the corrosion development. Drs. Berke and Popovics also added that for DCSSs, carbonation may be an issue because the temperature and moisture content inside the concrete could lead to a cyclic, partial dryout of the concrete, in turn leading to enhanced carbonation.

4.9.3 Considerations for Mitigation and Inspection

Methods to prevent corrosion of reinforcing bars in concrete are discussed in ACI 222R (ACI, 2001) and include control of chloride content in the concrete mix; the use of corrosion-resistant steels; the application of coatings, sealants, or overlays to prevent the ingress of moisture and ionic species; the use of chemical inhibitors; and cathodic protection. Most of these are also discussed in further detail in Chapter 5 of this report. If a licensee were to credit these approaches in its analysis of the potential for corrosion during the renewed license term, it would need to provide sufficient information to confirm that the conditions under which they were determined to be effective in laboratory tests or other field applications also hold true for the DCSS.

The AMP for reinforced concrete structures in NUREG–1927, Revision 1 (NRC, 2015c) relies on periodic visual examination of accessible surfaces, as described in ACI 349.3R (ACI, 2010a), and groundwater monitoring consistent with the acceptance criteria in the ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL (ASME, 2001). ACI 349.3R (ACI, 2010a) recommends that visual inspection complemented by nondestructive testing [e.g., ASTM C876 (ASTM, 2009)] could help assess whether reinforcing bars are undergoing active corrosion. Although the reinforcing bar is largely embedded, surface staining, loss of cover, or spalling could indicate a degraded condition. The presence of a high concentration of chlorides in groundwater or soil could also warrant a site-specific plan for managing the potential degradation of below-grade structures. Should degradation of reinforcing bars be detected, a licensee would need to evaluate the extent in accordance with its CAP and determine whether the system can still perform its intended safety function and what repairs are necessary.

4.10 Coupled Mechanisms

4.10.1 Technical Background

Coupled degradation mechanisms in concrete refer to chemical, physical and mechanical degradation modes that can interact, affecting the initiation and progress of each other. Examples of coupled degradation mechanisms include ASR, chloride penetration, sulfate attack, carbonation, freeze-thaw cycles, and shrinkage. Sagüés et al. (1994) demonstrated that the presence of cracks in concrete can exacerbate the diffusion of aggressive species inside the concrete, further enhancing the initiation of corrosion of the reinforcing steel. Leaching of calcium hydroxide and carbonation can decrease the concrete pH, affecting the stability of the passive film of the reinforcing steel and increasing the corrosion of the reinforcing steel (Sagüés et al., 1997; Broomfield, 1997). Carbonation and leaching of concrete can affect chloride diffusivity by releasing bound chlorides in concrete and can promote a decrease in concrete pH (Fagerlund, 2003). In addition, leaching of concrete can lead to an increase in concrete porosity, and thus, an increase in water absorption with the potential of ASR development. Interaction between ASR and freeze and thaw has been studied by Wei et al. (2005). In this study, concrete cracks produced by ASR can exacerbate freeze and thaw due to the enhanced pathways of moisture ingress. The additional influence of heat and radiation damage can compound environmental damage. Research into the significance of radiation effects for concrete is ongoing. There is evidence that radiation may play a significant role in promoting ASR degradation (Ichikawa and Koizumi, 2002; Ichikawa and Kimura, 2007; Mirhosseini et al., 2013). This phenomenon appears to be tied to the amorphization of crystalline silica when subject to radiation. Because amorphous silica is more soluble than crystalline silica in the high pH pore fluid of concrete, the ASR susceptibility is increased. There is no direct operating experience in commercial reactors to support this degradation phenomenon. However, there is potential for this coupled mechanism to occur, as has been pointed out in NUREG/CR–7171 (NRC, 2013). The potential for radiation to accelerate development of ASR has been identified at the Seabrook Nuclear Power Plant (NRC, 2013).

Jacobsen et al. (2006) proposed a mathematical method to quantify the overall concrete degradation based on coupled degradation mechanisms using superposition of the individual damage modes and phenomenological coupling coefficients. To verify this approach and to obtain the phenomenological coefficients, laboratory and/or field data are required. To quantify the degree of interaction of coupled degradation modes (e.g., temperature, RH inside the concrete, chloride concentration, carbonation) and the effect on the rate of deterioration of concrete, coupled, nonlinear, partial differential equations solved by a nonlinear finite difference code can be implemented (Puatatsananon, 2005).

In-depth studies of the effects of concrete damage subjected to all the potential coupled degradation mechanisms are lacking. NRC (2014a) provides information on the coupled concrete degradation mechanism, including knowledge gap assessments applicable to DCSSs.

4.10.2 Summary of Workshop Discussion and Expert Panel Assessment

Dr. Berke indicated that the size of concrete cracks is important in assessing the corrosion of the reinforcing steel. Dr. Berke noted that research on coupled degradation mechanisms of concrete was performed in Florida to address the effects of combined shrinkage cracks and corrosion of the reinforcing steel (Lau and Sagüés, 2009). Dr. Berke also indicated that even small cracks have a greater effect on the transport processes for high performance concrete than for normal concretes. This is because a small crack in high performance concrete is actually a much bigger path than the pore network in the concrete around the crack.

Another example of the coupled degradation modes is the effect of cations on the diffusion coefficient of chloride ions. Drs. Xi and Berke noted the study of the coupled diffusion of anions and cations in concrete. They indicated that the companion cations (e.g., Mg^{+2}, Ca^{+2}, and Na^{+}) of chloride salts play a significant role in the overall diffusion of chlorides into concrete. For instance, Drs. Xi and Berke indicated that Mg^{+2} and Ca^{+2} ions are particularly aggressive to concrete.

Dr. Xi provided additional examples of coupled mechanisms such as freeze and thaw and corrosion of the reinforcing steel and the combined effect of concrete temperature and moisture profile inside the concrete matrix. The freeze-thaw cycles cause cracking that increases flow of aggressive ions or acids and accelerates the corrosion of the reinforcing steel. The concrete temperature accelerates the distribution of moisture that activates other mechanisms to degrade concrete and steel.

4.10.3 Considerations for Mitigation and Inspection

Considerations for mitigation and inspection of individual concrete degradation mechanisms are discussed in Sections 4.1 to 4.9 of this report. As mentioned earlier, interaction between two or more degradation modes may result in overall accelerated degradation of concrete. If indications of a particular degradation mode are detected, part of the licensee's corrective action may be to analyze whether the consequences of such degradation mode could make the structure more susceptible to any other mode of degradation. In such case, a site-specific aging management approach may be needed.

4.11 Summary

The degradation modes addressed by the expert panel included freeze-thaw cracking, salt scaling, acid and ion attack, thermal desiccation, creep, ASR, DEF, radiation damage, reinforcing bar corrosion, as well as coupling of multiple modes. These mechanisms were identified in the TIN report or by suggestion of the panelists as potentially being able to affect concrete in DCSSs. The discussion for each mechanism included contributing factors, structural significance, detectability, and prevention or mitigation methods. Based on the system design and operating environment, the modes of degradation thought to be most likely to occur are freeze-thaw cracking, acid and ion attack, ASR, and reinforcing bar corrosion. The panelists believed that all of the mechanisms will eventually manifest on external surfaces by cracking, discoloration, or some other feature, and could be detected by periodic visual inspection; for example, performed in accordance with ACI 349.3R (ACI, 2010a). However, degradation on below-grade or otherwise inaccessible areas may require soil excavation to directly detect. The panelists could not generically analyze the structural effects of the degradation mechanisms on the DCSS, but based on their knowledge of other reinforced concrete structures, believed that the DCSS could maintain its safety functions provided that indications of degradation are promptly analyzed and repaired when they are detected.

5 DEGRADATION PREVENTION AND MITIGATION

5.1 Technical Background

Depending on the type of degradation mode, a number of prevention and mitigation methods have been developed for concrete structures. These methods are normally used to extend the structure design life. There are five distinctive approaches for mitigating and preventing concrete degradation that are applicable to new or existing concrete structures or both: (i) materials selection, (ii) use of chemical compounds, (iii) use of concrete sealants and coatings, (iv) application of a cathodic protection system, and (v) concrete curing. Approaches (i) and (v) are only applicable to new concrete structures, whereas the remaining approaches can be used for both new and existing constructions.

5.1.1 Materials Selection

Proper materials selection of the concrete and reinforcing bar could prevent or mitigate some of the degradation processes discussed in Chapter 4. One of the most common ways of mitigating and preventing sulfate attack is to reduce the alumina content by limiting the amount of tri-calcium aluminate in the Portland cement. The use of slag, silica fume, and fly ash as a cement replacement is also effective in reducing the potential for corrosion of the reinforcing steel and sulfate attack by decreasing concrete permeability to aggressive ions and reducing the amount of tri-calcium aluminate in the concrete [ACI 201.2R (ACI, 2008c)]. Minimizing and preventing alkali-silica reaction (ASR) degradation in concrete can be accomplished by using nonreactive aggregates, limiting the alkali content of concrete to less than 0.6 percent by weight Na_2O equivalent. Concrete mixtures with a high water-to-cement ratio (i.e., >0.45), tend to have a relatively high porosity and can exhibit substantial drying shrinkage and reduced protection of the reinforcing steel from chlorides. ACI 222.3R (ACI, 2011a) provides a guide to design and protect in-service concrete structures exposed to corrosive conditions.

Corrosion-resistant steels can prevent and mitigate corrosion of the steel reinforcement (rebar) in concrete. Various steel alloys [e.g., 316, 2205, 2304 stainless steels (SSs)] containing different amounts of chromium and nickel have been successfully used to prevent and/or mitigate corrosion of the steel reinforcement in concrete even in concretes of low quality (McDonald et al., 1998; Clemena and Virmani, 2002). Although many studies reported superior corrosion performance of SSs in concrete, the main impediment to their use in reinforced concrete structures is the high initial costs. Rebar produced with a composite structure incorporating a thick SS outer layer with a carbon steel interior has been developed to reduce the use of costly corrosion-resistant metals but still provide enhanced corrosion resistance. SS clad rebar with 316L cladding was investigated because it may become a cost-effective means of controlling corrosion in steel under very aggressive environments. However, SS clad rebar is vulnerable to corrosion at cladding breaks resulting from local mechanical damage. Upon chloride contamination of the surrounding concrete, an intense galvanic couple may develop between the exposed carbon steel and the passive SS clad (Zayed and Sagüés,1990; Cui and Sagüés, 2006).

5.1.2 Concrete Curing

Concrete curing plays an important role in strength development and durability of concrete, including the resistance to freezing and thawing, resistance to deicing chemicals, and minimization of creep (Siddiqui et al., 2013). ACI 301 (ACI, 2010c) recommends a minimum curing period corresponding to concrete attaining 70 percent of the specified compressive strength. Researchers have indicated that extended moist curing can increase the modulus of elasticity and reduce the creep, making the concrete more prone to cracking (Burrows, 1998). In addition, preventing internal concrete temperatures from exceeding 70 °C [158 °F] at the early stages of concrete curing is effective in preventing later development of delayed ettringite formation (DEF) (Famy et al., 2002).

5.1.3 Use of Chemical Compounds

The use of chemical compounds as part of the concrete mix or applied to the surface of an already built structure could prevent or delay degradation of concrete or reinforcing bar. The application of corrosion inhibitors (e.g., water soluble silicates, calcium-nitrite-based and amine-based organic inhibitors) at the concrete surface or applied as concrete admixture can mitigate the corrosion rate of the steel reinforcement by hindering the anodic and/or cathodic reactions (Siegwart et al., 2002). The effectiveness of the inhibitor applied at the concrete surface depends on the penetration rate and the concentration of the inhibitor at the steel surface. A critical concentration ratio between inhibitor and chloride of about 1.0 or greater is required in order to prevent the onset of corrosion. This implies that quite high inhibitor concentrations have to be present in the pore water of concrete in order to act against chlorides penetrating from the concrete surface. On another hand, the use of lithium compounds can effectively mitigate ASR in concrete. The data indicated that a molar ratio of lithium to alkali of 0.6 for LiF and 0.92 for Li_2CO_3 or above was sufficient to suppress expansion efficiently (Stark, 1992; Stark et al., 1993). A lithium nitrate-based admixture, typically used for protection against ASR, was also found to be effective in suppressing DEF-induced expansion in heat-cured mortars.

5.1.4 Use of Concrete Sealants and Coatings

As mentioned previously, coatings and sealants can be applied to the external concrete surface or to the surfaces of reinforcing bars to mitigate and prevent structural deterioration. For instance, coatings, membranes, and sealers that limit the penetration of moisture and aggressive ions in concrete may be applied to the concrete surface and can effectively prevent and/or mitigate the concrete degradation modes that require the presence of moisture (e.g., ASR, freeze and thaw, corrosion of the reinforcing steel). Concrete sealers (e.g., polyurethanes, methyl methacrylates, certain epoxy formulations, relatively low molecular weight siloxane oligomers, and silanes) are liquids applied to the surface of hardened concrete to either prevent or decrease the penetration of water, carbon dioxide, and chlorides, among other aggressive species (Naus et al., 1996). Coatings and membranes (e.g., epoxy resins, polyester resins, acrylics, urethanes, neoprenes, vinyls, rubberized asphalts) are usually thicker than sealers and generally do not penetrate the concrete. In addition, polymer, latex, and silica-fume-containing concrete overlays have been developed to significantly reduce chloride ion penetration. Epoxy-coated steel reinforcement has gained acceptance since the early 1980s as a means to extend the useful life of highway structures and is an accepted practice per ACI 318-05 (ACI, 2005a) and ACI 349-06 (ACI, 2007c). The epoxy coating prevents moisture and chlorides from reaching the surface of the reinforcing steel by acting as a barrier to prevent corrosion of the reinforcing steel. However, results from recent research activities cast doubt on the ability of epoxy coatings to provide long-term corrosion protection to steel in concrete exposed to chlorides. The issue has been attributed to the number and size of breaks or defects in the coating and reduction in adhesion between the epoxy coating and steel substrate (Sagüés, 1994; Weyers et al., 1997).

5.1.5 Cathodic Protection

For reinforced concrete structures, impressed current and galvanic cathodic protection are commonly employed to prevent and/or mitigate corrosion of the steel reinforcement in a variety of structures, including bridges, pavements, and pipelines (Broomfield, 1994). Under most conditions, a polarization of about −200 mV to −300 mV with respect to the open circuit potential or a cathodic current density of about 0.2 and 2.0 mA/m^2 with low chloride content is sufficient to achieve optimal cathodic protection. Excessive cathodic polarization should be avoided to prevent onset of the hydrogen evolution, to reduce the possibility of hydrogen embrittlement of high strength reinforcing steel, and to prevent the development of ASR (Ali, 1993). For existing salt-contaminated concrete structures, operating current densities can range between 2 and 20 mA/m^2 [ACI 222R (ACI, 2001)]. A necessary requirement for the application of a cathodic protection system is that all the electrically connected reinforcing steels are connected to a rectifier (in the case of the impressed current system) or to a sacrificial anode (in the case of the galvanic cathodic protection). The key to success in cathodic protection is the ability to achieve uniform distribution of current over the entire steel surface. However, this uniform current distribution is difficult to attain in field exposures due to potential inhomogeneous concretes and the effects of the environment.

5.2 Summary of Workshop Discussion and Expert Panel Assessment

At the present time, there are more than 2,200 loaded dry cask storage systems (DCSSs) in place at various dry storage sites. Thus, the prevention and mitigation of concrete degradation is of vital importance for existing DCSSs, especially for the expected long-term exposures. Dr. Berke indicated that there are a variety of approaches with good experience in mitigating concrete degradation for a great variety of applications. The expert panel's discussion regarding the various concrete degradation mitigation approaches is summarized next.

Drs. Xi, Berke, and Popovics agreed that the initiation and propagation of almost all the degradation mechanisms rely on the presence of moisture. Thus, if moisture can be mitigated (e.g., by proper drainage, coatings), the service lives of the DCSSs can be potentially extended. To that end, the panelists agreed that a variety of prevention strategies employed in new or existing concrete structures could be used to deter or minimize some of the concrete degradation mechanisms presented in this report. Dr. Berke indicated that some of the applications of the prevention and mitigation approaches presented here can be used for both the foundation pad and the walls and roofs of the DCSS. In addition, Dr. Berke suggested that among all the mitigation methods discussed here, the most cost-effective would be a combination of silane or some silane-type treatments and inhibitors applied to the outside of the concrete surface. The approaches for mitigation and prevention measures discussed by the panelists are presented next.

5.2.1 Materials Selection

Drs. Berke, Xi, and Popovics discussed the implementation of an optimal concrete mix design (e.g., water-to-cement ratio, entrained air) so that there is a balance between concrete strength and durability with the purpose of extending the lifetime of the DCSS. Dr. Popovics mentioned that for any concrete structure, there are different sets of design criteria for durability as opposed to the strength and loads. For instance, Dr. Berke indicated that changes in structural designs of DCSSs may be needed to take advantage of potentially greater concrete strengths (e.g., >3,000 psi) to improve concrete durability while maintaining the concrete's intended safety functions. To that end, Dr. Xi indicated that the highway industry has started to use high performance concretes where there is a balance between strength, shrinkage, cracking resistance, and chloride diffusivity. The panelists agreed that this approach was conducive to the implementation of additional concrete testing to identify the optimal concrete mix design. Dr. Berke agreed that a similar approach could be undertaken for the next generations of DCSSs.

Dr. Berke mentioned that concretes should be designed based on a performance-based standard, taking into consideration many of the effects of the different degradation mechanisms. For the concrete design basis, for instance, the ASR degradation mode is highly linked to the reactivity of aggregates, as described in Section 4.7 of this report. Drs. Berke and Popovics and Mr. James indicated that the current screening tests for ASR are sometimes not useful in assessing the degree of reactivity of the aggregates, and thus, the later implications on concrete ASR. However, extending the duration of screening tests for ASR could be useful for assessing the degree of reactivity of the aggregates and the prevention of short- and long-term ASR degradation in concrete. Dr. Berke mentioned that changes in concrete design basis could also involve the use of higher corrosion-resistant steels. This approach has been undertaken by the highway industry to extend the service lives of concrete bridges.

5.2.2 Concrete Curing

Drs. Popovics and Berke discussed the simple practice to prevent concrete damage by extending the curing period. This approach could potentially reduce shrinkage stresses by providing external moisture for continued hydration during the early stages of concrete hardening. Dr. Berke indicated that the outside surface of the concrete benefits the most from enhanced curing. In addition to the enhanced concrete curing, the mix design has to be able to generate a concrete with low ionic transport and the ability to sustain a certain level of internal stress to minimize the formation of cracks.

5.2.3 Use of Chemical Compounds

For the past decades, inhibitors have been commonly used to treat concretes for corrosion mitigation. If the concrete is dry enough, silane-based inhibitors of low viscosity and low surface tension can be effectively used for corrosion mitigation. Dr. Berke stated that even with a lack of moisture inside the concrete, these inhibitors can only penetrate a few metric [inches] into the concrete. Dr. Berke indicated that new products (organo-functional silanes) with added inhibitors are being developed to enhance both the penetration distance into the concrete and the drying of the concrete. Some of these new technologies can both allow the transfer of internal moisture out of the concrete and prevent external moisture from entering the concrete matrix. To that end, for highway structures penetration sealers are being used after drying the concrete surface. Dr. Berke also stated that one approach to make a hydrophobic concrete is by spreading butyl oleate or calcium stearate.

Dr. Berke indicated that the lithium nitrate method is another approach to protect concrete structures. The lithium nitrate is being successfully used as an admixture for new constructions. For existing structures, however, the lithium nitrate exhibits similar issues as the silane-based inhibitors in that the penetration rate is rather slow, reaching only a few metric [inches] of concrete cover. There are approaches that can be implemented in the field to accelerate the diffusional process, but those require drilling the concrete to pour lithium nitrate. Dr. Berke mentioned that another issue with the lithium nitrate is that nitrate in conjunction with chloride ions can exacerbate other degradation modes, such as corrosion of the steel reinforcement.

Drs. Berke and Popovics agreed that electrochemical approaches have been extensively used in concrete for chloride extraction. A ponded solution with target cations (e.g., lithium ions) is placed on the concrete surface. A current (or potential) is driven between the reinforcing steel and an auxiliary electrode placed outside the concrete in contact with the solution. This applied potential can accelerate the migration of lithium ions inside the concrete while removing chlorides away from the steel reinforcement surface. This approach can significantly reduce corrosion of the steel. However, this methodology is costly to implement.

5.2.4 Use of Concrete Sealants and Coatings

Drs. Berke and Xi mentioned the preventative methodology of the use of concrete polymer-based coatings or siloxane-based sealers applied on the concrete exterior surface for both new and existing concrete structures. Dr. Popovics indicated that the application of concrete coatings and sealers is an approach used to prevent the ingress of external moisture into the concrete. As mentioned earlier, ingress of external moisture in concrete can enhance various degradation modes; for instance, ASR, freeze and thaw, and corrosion, to cite a few. Drs. Xi and Berke agreed that there are several types of concrete coatings and sealers in the market. These coatings and sealers are commonly classified into two main groups: (i) breathable and (ii) unbreathable. The breathable products allow the escape of internal moisture in the concrete, whereas the unbreathable materials do not. Dr. Xi indicated that the application of unbreathable products could be detrimental to the concrete performance in that the coating or sealer can completely block the transfer of internal moisture out of the concrete, potentially triggering other degradation modes, such as corrosion of the reinforcing steel. To prevent this, the concrete surface is initially dried so that there is no moisture trapped inside the concrete. Drs. Xi and Berke agreed that breathable products are better suited for most of the outdoor applications, and they are being successfully used to protect bridge decks and parking garages. However, polymer-based coatings tend to embrittle by thermal degradation and neutron irradiation.

5.2.5 Cathodic Protection

Another methodology commonly used to protect the concrete reinforcing steel from corrosion is the use of cathodic protection. Drs. Berke and Popovics agreed that this approach is expensive and has some limitations. Dr. Berke indicated that cathodic protection normally increases the pH as well as drives the chloride ions away in the vicinity of the steel reinforcement, which is beneficial for mitigating steel corrosion. However, Dr. Popovics mentioned that hydrogen gas development could take place at the steel surface at low temperatures. In addition, Dr. Berke mentioned that this effect can be conducive to ASR degradation.

5.3 <u>Considerations for Prevention and Mitigation</u>

As discussed previously, there are numerous technologies to prevent and mitigate degradation of concrete. The U.S. Nuclear Regulatory Commission (NRC) will evaluate such factors as the persistence and duration of the mitigation methodology, whether the mitigation methodology itself could introduce new degradation processes, or whether the method would affect the ability to inspect the system. As appropriate, NRC may participate in the development of, and eventually endorse by reference in regulatory guidance, consensus codes and standards that would govern the use of the mitigation methods, whether they be applied at the time of system design or in service. Some specific considerations for the main mitigation approaches are described as follows.

5.3.1 Materials Selection

Based on the panelist's inputs, for new DCSSs, the primary approach for the prevention of most of the concrete degradation modes is to achieve an optimum balance of the concrete mix design between concrete strength and durability to improve concrete lifetime and prevent the development of concrete damage while maintaining the concrete's intended safety functions. Implementation of an optimal concrete mix design, including relatively low cement-to-water ratio, the use of pozzolanic materials, and higher corrosion resistance steels, could potentially lead to extended durability for the next generations of DCSSs. In recent years, the concrete industry developed high performance concretes designed to provide enhance durability and long-term performance as well as enhanced mechanical properties that otherwise cannot be achieved using conventional ingredients, mixing, and curing practices. Typically, such concretes have low water-to-cement ratios of 0.30 or below and contain pozzolanic admixtures and additives to reduce concrete permeability while increasing workability that is necessary for ease of placement. A reduction in concrete permeability, as well as the use of pozzolanic materials, promotes a greater resistance to corrosion of the reinforcing steel and ASR, and surface attack, to cite a few degradation modes. However, the resulting high concrete strength of the high performance concretes may affect the behavior of other DCSS components. For instance, the concrete pad is designed to have certain strength as well as flexibility to protect the fuel inside the cask during a nonmechanistic tip-over event. Otherwise, internal cask damage can occur.

The current NRC guidance provided in NUREG–1536 (NRC, 2010a) accepts construction in accordance with ACI 349-06 (ACI, 2007c) or ACI 318-05 (ACI, 2005a). Further, the guidance in NUREG–1536 (NRC, 2010a) considers aggregates to be acceptable if they conform to ASTM C33 (ASTM, 2002). Criteria for reinforcing bars are related to the code to which the structure was designed. Examples include ASTM A615 (ASTM, 2015a) and ASTM A706 (ASTM, 2015b). The use of corrosion-resistant SS reinforcing bar is addressed in ASTM A955 (ASTM, 2015c). The likely approach for any modifications or enhancements to materials selection practices for improved performance is advancement though the governing standards body, subject to endorsement by NRC. Licensees may also propose site-specific approaches with the appropriate technical basis to demonstrate the design attributes.

5.3.2 Concrete Curing

ACI 318-05 (ACI, 2005a) refers to detailed recommendations for concrete curing practices in ACI 308R (ACI, 2008b), with more prescriptive guidance in ACI 308.1 (ACI, 2011b). Special considerations for cold or hot weather curing are given in ACI 306.1 (ACI, 2002) and ACI 305.1 (ACI, 2014). Modifications or enhancements to curing practices for improved performance could be advanced though ACI or another governing standards body, subject to endorsement by NRC.

5.3.3 Use of Chemical Compounds

The use of chemical inhibitors has been a common practice to treat concretes for corrosion mitigation of new and existing concrete structures. Silane-based inhibitors can enter the concrete cover but for just a few inches, limiting their use. The panelists indicated that new inhibitors are being developed to enhance both the penetration distance into the concrete and the diffusion rate. Some of these inhibitors can transfer the internal moisture out of the concrete while preventing external moisture from entering the concrete matrix. Calcium nitrite is the most extensively tested inhibitor used in parking, marine, and highway structures (Berke and Weil, 1994). That investigation revealed a critical concentration ratio between inhibitor and chloride of about 0.6 (with some variation from 0.5 to 1.0) in order to prevent the onset of corrosion. This implies that quite high inhibitor concentrations have to be present in the pore water of concrete in order to prevent chlorides from penetrating the concrete surface. To avoid chloride ingress and the use of excessively high inhibitor concentrations, inhibitors admixed in high quality concretes are recommended (Berke and Weil, 1994). The panelists also indicated that the lithium nitrate, used as an admixture or as a surface-applied inhibitor in new and existing structures, can mitigate the progression of ASR. This observation has been corroborated in an investigation by Thomas and Stokes (1999). However, the lithium nitrate exhibits similar issues as the silane-based inhibitors in that the penetration rate is rather slow, reaching only a few metric [inches] of concrete cover, and can exacerbate other degradation modes, such as corrosion of the steel reinforcement. Additionally, corrosion inhibitors might affect plastic and hardened concrete properties when used as an admixture during concrete casting. Thus, appropriate steps should be taken to overcome or minimize detrimental interactions in concrete. Another concern is that because corrosion-inhibiting admixtures are water soluble, inhibitor leaching from the concrete can occur, effectively reducing the concentration of the inhibitor.

ACI 318-05 (ACI, 2005a) and ACI 349-06 (ACI, 2007c) permit the use of concrete admixtures provided that they conform to relevant ASTM International (ASTM) standards. These include ASTM C494 (ASTM, 2015d) and ASTM C1582 (ASTM, 2011c). The ASTM standards address provisions to ensure the composition of the admixtures and tests to demonstrate their effects on the concrete properties. The application of chemical treatments, such as lithium-bearing compounds, to the surface of as-built structures, is not directly addressed in ACI 318-05 (ACI, 2005a) and ACI 349-06 (ACI, 2007c), but could be reasonably understood to be analogous to the application of coatings, to the extent that they may require specifications for material selection, surface preparation, and application procedures. Representative standards include NACE International (NACE) SP0892 (NACE, 2007b), ASTM D4258 (ASTM, 2012d), and ACI 515.2R (ACI, 2013a).

5.3.4 Use of Concrete Sealants and Coatings

External concrete coatings and sealers are commonly used to prevent and mitigate concrete degradation. It is worth noting that this approach can be employed for both new and existing DCSSs. The panelists noted that there is variability among commercially available preventative products and that consideration should be given to the expected performance of the specific product used. As a preventative measure, the panelists indicated that the use of concrete polymer-based coatings or siloxane-based sealers applied on the concrete exterior surface could prevent external moisture ingress in concrete. This approach could be more relevant later in the life of the DCSS when the fuel heat load decay over time may limit the internal drying of the concrete (or the resistance to moisture ingress) for a period of time. Among the commercially available coatings and sealers, the panelists suggested that the application of the unbreathable sealers could be detrimental to the concrete performance in that the coating or sealer can completely block the transfer of internal moisture out of the concrete, potentially triggering other concrete degradation modes, such as corrosion of the reinforcing steel in the DCSS concrete structure. As such, the panelists concluded that breathable products are better suited for most of the outdoor applications. The use of concrete coatings has other added benefits to prevent and mitigate concrete deterioration, such as sulfate and chloride attack. The degree of concrete protection and coating performance depends on the quality of surface preparation of the concrete, coating application, environmental conditions during application, coating film thickness, and environmental exposure. Field studies have shown that the majority of coating failures are caused by improper material selection and concrete surface preparation (ICRI, 1997b). To that end, most concrete sealers and coatings are not effective for sealing cracks; concrete cracks would need to be sealed prior to the application of the concrete sealer. The literature review performed on concrete coatings and sealers indicates that there is a wide difference in the performance of these surface treatments for protecting or minimizing concrete deterioration.

The use of coatings is addressed in NUREG–1536 (NRC, 2010a), particularly for the corrosion protection of carbon steel, but with guidance that would be generally applicable to their use on any important to safety structure. NUREG–1536 (NRC, 2010a) states that coatings should not be credited for protecting the substrate material unless a periodic coating inspection and maintenance program is required. Further guidance is given for the NRC staff review of the coating material selection, surface preparation, application method, repair, and qualification testing. NUREG–1801 (NRC, 2010b) provides a generic aging management program (AMP) for coating monitoring and maintenance of reactor structures, referencing Regulatory Guide (RG) 1.54 (NRC, 2010c). NUREG–1801 (NRC, 2010b) and RG 1.54 (NRC, 2010c) provide further details on a number of the relevant ASTM standards, notably ASTM D5144-08 (ASTM, 2008b).

5.3.5 Cathodic Protection

A widely used corrosion preventive and mitigation method that can be used for new and existing DCSSs is the cathodic protection system. This approach has been extensively used for corrosion protection of reinforced concretes for bridges and pavements, to cite a few examples. As mentioned previously, the application of a cathodic protection system can also restore the concrete alkalinity, as well as decrease the chloride concentration in the vicinity of the steel reinforcement. This effect can be beneficial from the corrosion protection standpoint. Before designing a cathodic protection system for a reinforced concrete structure, various parameters need to be established, mainly by nondestructive testing. The electrical continuity of the reinforcement must be established to ensure that all rebars will be protected. The electrical resistivity of the concrete is measured in order to determine operating voltages, and the applied current densities are calculated on the basis of steel surface area. A maximum anodic current density of 110 mA/m^2 is commonly used. It is also necessary to ensure that no steel is polarized to a more negative potential than −1,150 mV versus a copper/copper sulfate reference electrode to avoid the possibility of hydrogen embrittlement of the steel surface. The panelists agreed that this effect can be conducive to ASR degradation, limiting the use of cathodic protection. However, the likely high resistivity of concrete for DCSS applications may pose challenges to the use of cathodic protection due to the difficulty in obtaining a uniform current spread to all the embedded steel components. As such, development of special materials and procedures may be required for the field implementation of a cathodic protection system.

Cathodic protection is already widely applied to safety-related nuclear components; namely, buried piping and tank systems for operating reactors. NUREG–1801 (NRC, 2010b) states that cathodic protection for these components should be performed in accordance with NACE International (NACE)-recommended practices. Should cathodic protection be proposed for DCSS components, consideration should be given to the applicability of NACE SP0290 (NACE, 2007a). NACE SP0290 (NACE, 2007a) addresses cathodic protection system design, installation, operation, and acceptance criteria, among other considerations.

6 CONCRETE MONITORING AND INSPECTION

6.1 Technical Background

Rules for renewal of specific licenses or Certificates of Compliance (CoCs) under the provisions of Title 10 of the *Code of Federal Regulations* (CFR) Part 72 require that licensees be able to demonstrate that structures, systems, and components (SSCs) important to safety can perform their intended function during the period of extended operation. This demonstration could involve engineering analyses to justify that certain degradation phenomena will not adversely affect the dry cask storage system (DCSS), or the description of a monitoring and inspection program to ensure that the phenomena are detected prior to the loss of functionality. Monitoring and inspection, which is the subject of this chapter, could involve the direct measurement or observation of physical features in DCSS components, or indirect measurements of environmental parameters that are indicators of conditions that could promote degradation.

While the terms *monitoring* and *inspection* are sometimes used interchangeably, for the purpose of this discussion, monitoring can be described as the continuous or semicontinuous acquisition of data by use of sensors or similar devices as a means to actively determine the condition of the structure. Inspection, on the other hand, can refer to examinations of the structure at periodic intervals to determine its condition at a particular point in time. Both of these approaches are addressed in this chapter, although inspection in more detail than monitoring. NRC (2014c) covers monitoring of DCSSs at greater length.

6.1.1 Direct Measurement

6.1.1.1 Visual Inspection

For accessible concrete surfaces, visual inspection is typically carried out prior to the execution of any other inspection approaches. It provides a rapid and effective method for identifying and defining areas of concrete distress. Crack comparators and remote portable cameras are practical to establish the extent of concrete surface damage, even in concrete areas difficult to access. This simple technique can provide both quantitative and qualitative information, such as surface cracking, spalling, volume change, cement/aggregate interaction, coating performance, and the presence of degradation deposits, among others. Periodic execution of visual examinations can provide a history of the degradation mode and identify whether the degradation is active or dormant. ACI 201.1R (ACI, 2008a) provides guidelines for conducting visual inspection of in-service concrete structures. ACI 201.1R (ACI, 2008a) also indicates the importance of having an experienced inspector document any observations related to environmental and loading conditions of the structure, including the age, type, and location. In addition to written descriptions, sketches of relevant features, photographs, and videos are valuable tools for proper visual inspections.

6.1.1.2 Other Nondestructive Testing Techniques

Subsurface cracking, internal delaminations, voids, and corrosion of the steel reinforcement cannot be detected by visual inspection, but would require nondestructive testing (NDT) techniques, such as ultrasonic, acoustic emission, chain drag, radiography and radar, and thermography. ACI 349.3R (ACI, 2010a), ASTM D4580 (ASTM, 2012e), and ACI 228.2R (ACI, 2013b) list the NDT methods applicable to monitor concrete degradation. Descriptions of each of these methods can be found elsewhere (Refai and Lim, 1992; Malhotra and Carino, 1991; Schickert et al., 2003; Friese and Wiggenhauser, 2008). Corrosion of reinforcing steel is one of the main threats to durability of reinforced concrete structures. Assessment of steel corrosion can be carried out by half-cell and surface potential measurements [Broomfield, 1997; ASTM C876 (ASTM, 2009)], electrical resistivity (Sagüés et al., 2001), linear polarization resistance (James Instruments, 2010), and galvanostatic pulse techniques.

6.1.1.3 Invasive Techniques

Invasive testing involves the removal of concrete (i.e., concrete coring) or steel reinforcement sections from the structure to assess the physical, chemical, microstructural, and mechanical properties of the concrete. In general, invasive techniques are typically conducted on concrete structures displaying degradation using a limited number of samples. Concrete coring is commonly used to assess concrete petrographic information, compressive strength, and chloride and sulfate profiles [ASTM C823 (ASTM, 2012f); ASTM C856 (ASTM, 2011a)]. ASTM C42 (ASTM, 2013b) provides guidelines for extracting cores from concrete structures. Typical concrete core dimensions are such that the core height is about twice as large as its diameter. ASTM C42 (ASTM, 2013b) also provides correction factors for tests on cores with length-to-diameter ratios between 1 and 2. The general rule adopted for core size, besides the height-to-diameter ratio, is the nominal size of aggregate: the core diameter should be not less than three times the maximum size of the aggregate. Cores are normally taken perpendicular to concrete surface near the middle of a concrete placement. The diameter of the cores must be at least 93.9 mm [3.70 in]. For concrete strength determination, a minimum of three concrete cores may be required. Limiting concrete coring to a minimum is beneficial because coring may create or enhance other degradation modes to the concrete. At present, there are no standard practices that relate the number of cores to the structural damage caused by coring.

Embeddable corrosion sensors have also been developed for tracking corrosion in concrete (Reis and Gallaher, 2006; Yang and Dunn, 2002). These sensors are designed to be embedded in concrete to monitor parameters (e.g., chloride ion, concrete resistivity, linear polarization resistance, corrosion potential, and concrete temperature) related to corrosion of the reinforcement in concrete. More recently, small, coupled, multielectrode array sensors were developed for *in-situ* monitoring of various forms of corrosion applicable to carbon steels, stainless steels (SSs), and nickel-based alloys as probe elements in a wide range of aqueous solutions (Yang et al., 2001; Yang and Dunn, 2002). Past studies (Yang and Sridhar, 2003) have demonstrated that the sensor can measure, in real time, the corrosion rate of reinforcing steel in concrete.

6.1.2 Indirect Measurements

Techniques that measure the environmental conditions to which concrete structures are exposed can be used to assess the potential for degradation when the structure itself is inaccessible for inspection; for instance, those below grade. Typically this would involve measurements of the pH or concentrations of chemically aggressive species in groundwater or soil. NUREG–1927, Revision 1 (NRC, 2015c) refers to criteria from the ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL (ASME, 2001), which designate an environment as aggressive if pH <5.5, chlorides >500 ppm, and sulfates >1,500 ppm.

6.2 <u>Summary of Workshop Discussion and Expert Panel Assessment</u>

In the next subsections, a panel discussion of the monitoring and inspection methods is presented. The discussion focused on the application of the most relevant and promising methods to assess deterioration pertaining to the DCSS concrete components.

6.2.1 Direct Measurement

6.2.1.1 Visual Inspection

Drs. Jacobs and Popovics agreed that visual inspection is critical and should always be performed to qualitatively assess concrete degradation before other inspection techniques are considered. However, one potential problem with visual inspection is the level of cleanliness of the concrete surface; the identification of cracks may be cumbersome. Drs. Berke and Popovics agreed that for other applications such as concrete bridges, the concrete surfaces are not cleaned prior to inspection, because it is best to look at the concrete surface for any presence of degradation product residues that might suggest that a certain mode of degradation is taking place. Cleaning the concrete surface can be done at a later stage if more details of the concrete surface are needed (e.g., if crack measurements are required). Dr. Xi indicated that the concrete surface needs to be cleaned prior to inspection and that has to be done every time concrete inspection is performed. The latter concept was supported by Mr. James, who indicated that concrete cleaning is required for accurate crack measurement. If cracks are present, a crack comparator is needed to more accurately map the concrete. Dr. Berke mentioned that an optical crack comparator was recently developed where cracks of 1 mm [0.04 in] wide can be measured at distances of about 6.1–30.5 m [20–100 ft]. Drs. Berke, Popovics, and Xi added that another broadly used technique usually associated with visual inspection is the hammer or chain drag monitoring test. In that test, the operator listens to the sound coming from the concrete to qualitatively determine whether internal degradation is ongoing, as indicated by Dr. Xi. Due to its low cost and simplicity, this approach is typically used for bridge decks and it is used in conjunction with visual examination. Regarding below-grade monitoring and inspection, Dr. Berke indicated that visual inspection of a concrete surface can be executed by removing the top metric [inches] of soil surrounding the concrete; in particular, near concrete corners.

Another method commonly listed under enhanced visual examination is the three-dimensional (3D) vision mapping. Dr. Popovics mentioned that this powerful methodology uses high resolution images or videos that are reconstructed using computer software, giving a 3D rendering, including texture, cracking, and other features. Dr. Jacobs added that the benefits of using this approach are that it is relatively inexpensive, is easy to implement, and provides storable documentation and comparison of the concrete surface at the selected inspection times. The shortcomings of this technique are that it is not mature and only the outer concrete surface can be inspected without assessing the concrete matrix so that there is no inference to the depth of cracking, if present.

6.2.1.2 Other Nondestructive Testing Techniques

As mentioned earlier, any visual inspection methodologies can provide an effective and qualitative assessment of the concrete outer surface. However, if a more in-depth analysis of the extent of a given degradation mode is needed, other more sophisticated inspection tools (e.g., impact echo used for bridge decks) should be used to characterize the behavior of the entire concrete thickness. Drs. Popovics and Jacobs agreed that the impact echo is a method that works very well for certain concrete defects, such as horizontal or vertical delamination, but has limited application scope due to the localized type measurement and the lack of detection of concrete voids. Dr. Popovics indicated that the limits of detection depend on the aspect ratio (i.e., how the delamination compares to its lateral extent). For instance, if the aspect ratio is less than three or four, detection would be highly compromised. Dr. Popovics added that there are other methods that are more susceptible to the detection of local concrete voids and local stiffness measurements such as the impulse response. This technique is similar to impact echo, but it uses a transducer to capture the signal. The gathered signal is then digitized and analyzed. One drawback of the impulse response technique is that it cannot detect voids that are away from the concrete surface and this approach is a point measurement.

Dr. Jacobs indicated that another noninvasive technique commonly used for concrete is radar waves. Radar is a mature tool, because a significant amount of data (e.g., concrete moisture) can be collected without touching the concrete surface. An air-coupled antenna can be used, which has the ability to quickly scan structures for metallic components inside the concrete. The presence of steel reinforcement tends to shield other defects from radar. Thus, this inspection tool is limited to detecting metals and not for concrete defects. Drs. Popovics and Jacobs mentioned that another more promising candidate for inspection of DCSS concretes is infrared (IR) thermography. The IR thermographic technique requires the application of a heat flux, and that can be provided by the differential temperature between the internal and external concrete surfaces in DCSSs. Drs. Popovics and Jacobs agreed that IR thermography can be applied to the outside concrete so that its implementation for DCSSs can be practical. Drs. Popovics and Jacobs indicated that although this technique may hold promise, the expected constant heat flux of the dry cask may pose challenges in the application of this method.

Another technique mentioned by Drs. Jacobs and Popovics was passive acoustic technique. However, the expected low amplitude and frequency of the signal will likely attenuate significantly throughout the concrete thickness, limiting the signal detection from the outside concrete surface. Dr. Jacobs indicated that a variation of the passive acoustic technique is the acoustic emission where multiple sensors are attached to the surface of the concrete.
Dr. Popovics mentioned that acoustic emission has gained popularity in detecting fractures and large brittle events in prestress concrete pipes. The energy produced by concrete degradation propagates through the concrete and is captured by the sensors to determine the location of the flaw and the magnitude of the defect. Dr. Jacobs indicated that the acoustic signal strength can be undetected for low energy magnitudes and low frequencies typical of alkali-silica reactions (ASRs) or delayed ettringite formation (DEF) concrete degradation events. As a result, it is believed that the acoustic emission technique would not be suitable for detection of flaws in DCSSs due to physical constraints. Another issue with the acoustic emission denoted by Dr. Xi is that the acoustic sensors would not likely be able to detect the location of distributed cracking events.

Dr. Popovics indicated that an NDT that has gained much popularity lately is the ultrasonic acoustic imaging array. The array contains a set of about 40-point sensors that are placed on the concrete surface. The sensor data provide an image of the cross-sectional structure. This technique can detect both local cracks and voids within the concrete as well as delamination. On the other hand, Dr. Jacobs mentioned that this inspection approach requires a dedicated technician in charge of holding the array sensors for about 5 seconds at different locations in the concrete surface, making this inspection technique is time consuming; additionally, the measurement is restricted to the location of the sensors. Another NDT technique mentioned by Dr. Jacobs was nonlinear ultrasound, which is not wavelength dependent. This technique measures the material nonlinearity *in situ*. In addition, this method can find distributed small defects. Dr. Jacobs indicated that both the ultrasonic acoustic imaging array and nonlinear ultrasound techniques offer high information quality, but the technological aspects are not mature yet. Dr. Popovics mentioned several other NDT methods (e.g., seismic echo or cross-hole sonic logging) to monitor the mechanical properties of concrete foundations. Some of these techniques are somewhat invasive in that a bore hole has to be drilled in the soil to embed a sensor tube next to the slab.

Dr. Popovics indicated that, from a monitoring or inspection perspective, techniques that are complementary can provide helpful information to assess the internal condition of the concrete. For instance, ultrasonic arrays with IR imaging can be useful to detect concrete voids with greater confidence. Drs. Popovics and Jacobs indicated that proposing a probability of detection or the minimum size of a concrete defect may not be useful. Instead, it is more important to determine the worst location and the maximum size of a structural defect that is tolerable based on the design basis, and then see whether an inspection technique that can detect that size of a defect or defect density. Dr. Berke indicated that concrete degradation has to be evaluated over time to assess the level of cumulative damage. For instance, corrosion of the reinforcing steel can be determined via electrochemical techniques. Performing such techniques yields actual corrosion rates at present time. However, no implications of cumulative damage can be obtained unless periodic measurements are taken at selected timeframes.

Once the concrete damage has been detected, quantification of the damage needs to be conducted. Quantification of concrete damage, however, is highly challenging. The panelists agreed that the concrete design basis (e.g., strength, stiffness, seismic) has to be reevaluated after cracks or voids are formed. The metrics by which concrete damage are quantified are variable. As an example, Dr. Berke mentioned that steel corrosion by itself may not be problematic from the point of view of the concrete strength loss. However, corrosion of steel can have other adverse effects on the concrete, such as cracking and spalling, which can accelerate other degradation mechanisms.

6.2.1.3 Invasive Techniques

The panelists agreed that among the destructive methods commonly used for concrete inspection is concrete coring. Small cores are normally extracted from concrete structures to evaluate the internal condition of the concrete. Dr. Berke indicated that chloride concentration and pH tests are typically carried out on concrete cores to determine concentration profiles. In addition, Drs. Berke and Jacobs mentioned that petrographic examination can be performed on concrete cores to detect internal degradation modes. Dr. Berke added that the opening left in the structure after coring can be used to connect a wire to the exposed reinforcing steel inside the concrete hole and the wire be used to map the steel corrosion potential using an external reference electrode placed at several locations outside the concrete surface. This corrosion potential would indicate the likelihood of steel corrosion in the vicinity of the reference electrode. On the other hand, Mr. James indicated that concrete coring in existing DCSSs is not desirable in situations where no concrete degradation signs are apparent. To that end, Dr. Berke added that extracting cores from concrete is worthy for cases where advanced stages of concrete damage (e.g., ASR, DEF) or functionality problems have been noted.

Once the concrete core is removed from an existing structure, Drs. Xi and Berke suggested embedding small sensors at various depths inside a tube that is placed at the concrete opening to monitor moisture gradient or concrete resistivity across the concrete thickness . Dr. Berke noted that for new concrete structures, these sensors can be placed during concrete casting. The same concept applied to the moisture and resistivity monitoring of the soil surrounding the concrete structure. The main issue with this sensing approach is that the measurements are highly localized and the long-term sensor performance and reliability remain unknown. Thus, for large structures, Dr. Popovics indicated that a significant number of these sensors may be required. Dr. Popovics indicated that the sensor may not be reliable many years from now, which diminishes their utility for long-term monitoring.

Drs. Popovics and Jacobs believe that the theoretical understanding of how these concrete degradation mechanisms progress can help develop a strategy about the way concrete can be monitored and inspected, keeping in mind that these concrete structures can sustain extensive degradation and still perform functionally and structurally. However, at some point, corrective actions may be needed. The data collected from inspections can then be used to model the performance of the structure, and thus, determine when structural problems will arise and when corrective actions will be required. This is true for well-developed predictive models, such as for corrosion of the reinforcing steel. However, for other concrete degradation mechanisms, such as ASR and freeze and thaw, no predictive models that can accurately determine the extent of damage were developed. For those cases, particular attention has to be paid to the concrete properties and ingredients, such as pH and aggregate types, to mitigate the occurrence of such mechanisms.

6.2.2 Indirect Monitoring and Inspection

As mentioned in Section 4.3, Drs. Popovics, Xi, and Berke agreed that the aggressive species in soil or groundwater are sulfate, chloride, phosphate, and magnesium ions. These panelists specified that indirect measurements, such as those performed in groundwater and soil, are often valuable in assessing the likelihood of developing degradation of below-grade concretes (i.e., foundation slab in the DCSS). Drs. Berke and Popovics added that, in practice, the parameters obtained from the groundwater and soil samplings can serve as input for modeling tools developed for estimating concrete durability. The panelists agreed that soil sampling tends to be more accurate than groundwater sampling due to the close proximity of the soil to the concrete structure. However, Dr. Berke noted that groundwater sampling is normally conducted for simplicity. Dr. Berke indicated that the measurements of soils and groundwater parameters serve as a guide, but they may not be reliable; additional information could be needed to fully assess the condition of the concrete. For instance, groundwater monitoring will be able to detect whether there is leaching of calcium in concrete, but it will fail to predict ASR and freeze and thaw initiation and propagation, according to Dr. Berke. In addition to groundwater monitoring, Dr. Berke indicated that the top slab area could give important information about the condition of the below-grade portion of the slab in that if concrete degradation (e.g., freeze and thaw, ASR) takes place at the above-grade slab, it will probably occur at the below-grade portion.

Mr. James indicated that to complement the groundwater monitoring, the behavior of the below-grade concrete can be indirectly assessed by installing monuments on top of the foundation slab to monitor the degradation of the below-grade slab, such as slab swelling and settlement. Dr. Popovics added that an alternative approach to monitor underground concrete is by conducting soil resistivity, which is directly related to the soil aggressiveness. Dr. Popovics suggested that rather than focusing on threshold values for the groundwater and soil measurements, it may be more appropriate to establish, for instance, a safe, a questionable, or a dangerous range of the parameters in terms of concrete degradation. The panelists indicated that these ranges can be a function of the type of cement, mix design, and other factors. Dr. Popovics indicated that some approaches can be applied to complement groundwater monitoring. For instance, soil resistivity can provide information about soil aggressiveness. Another approach is to employ methods such as seismic echo or cross-hole sonic logging, from which mechanical properties of concrete embedded in soil can be inferred. Some of these methods, however, are somewhat invasive in that a sensor tube is embedded in soil next to the concrete.

6.3 Considerations for Implementing Inspection and Monitoring Programs

NUREG–1927, Revision 1 (NRC, 2015c) provides generic guidance for the information that should be included in a license or CoC renewal application to adequately describe an inspection and monitoring program. This includes sufficient detail to demonstrate that the inspection method is capable of detecting the degradation phenomena for which it is credited and includes inspection frequencies, sample sizes, data collection methods, and timing of inspections. Specific considerations for the implementation of the various inspection and monitoring approaches are described as follows.

6.3.1 Direct Measurement

6.3.1.1 Visual Inspection

NUREG/CR–1927, Revision 1 (NRC, 2015c) identifies visual examination as an acceptable inspection methodology for accessible portions of the DCSS. Visual examination is generally a well-established methodology, as discussed in further detail in ACI 201.1R (ACI, 2008a). Inspection frequencies are given in accordance with those in ACI 349.3R (ACI, 2010a). The primary challenge related to visual inspection would relate to below-grade or inaccessible surfaces, where a licensee-specific approach for periodic soil excavation or opportunistic inspection may be warranted.

6.3.1.2 Other NDT Techniques

To detect subsurface degradation or rebar corrosion not revealed by visual inspection, NDT techniques, such as ultrasonic, acoustic emission, radiography and radar, and thermography, could be applied. They are not explicitly addressed in NUREG–1927, Revision 1 (NRC, 2015c), but could be implemented as part of a corrective action program should visual inspection identify a condition for further analysis. ACI 228.2R (ACI, 2013b) and ACI 349.3R (ACI, 2010a) provide information on the use of a number of the NDT techniques. Because these have not been applied to DCSSs to date, however, a licensee would need to provide sufficient information to NRC to demonstrate their effectiveness for this application. There is limited experience with the use of chain drag and sounding hammer methods to probe for subsurface delaminations on pads. Accepted approaches are given in ASTM D4580 (ASTM, 2012e).

6.3.1.3 Invasive Techniques

Coring of concrete should only be applied with great caution to DCSSs. Prior to carrying out such testing, a licensee would be required to perform analyses to demonstrate that such a modification to the system would not compromise the intended safety functions. Should coring be applied, relevant guidance for core extraction and analysis could be found in ASTM C42 (ASTM, 2013b), ASTM C823 (ASTM, 2012f), and ASTM C856 (ASTM, 2011a). Similar considerations would apply to drilling into the DCSS for placing embedded sensors. Technical challenges related to the use of embedded sensors are discussed in further detail in He et al. (2014).

6.3.2 Indirect Monitoring and Inspection

NUREG–1927, Revision 1 (NRC, 2015c) states that a groundwater chemistry or soil monitoring program can be used to determine whether there is an aggressive below-grade environment.

No specific guidance is given on groundwater or soil testing methods. These are, however, addressed in Regulatory Guide (RG) 1.138 (NRC, 2003). The primary focus is soil testing, but the RG refers to standard methods of testing water for physical, chemical, radioactive, and microbiological properties, as described by the American Public Health Association (APHA) in "Standard Methods for the Examination of Water and Wastewater" (APHA, 2012). This manual primarily addresses analytical techniques that could be used to determine the concentrations of chemical species in the water. Electric Power Research Institute (EPRI) Report 1016099 (EPRI, 2008) describes steps for locating, installing, testing, and maintaining monitoring wells. Referenced standards in include ASTM D5092 (ASTM, 2012g), ASTM D5978 (ASTM, 2011d), ASTM D4448 (ASTM, 2013c), and ASTM D5903 (ASTM, 2012h). With respect to soil testing, RG 1.138 (NRC, 2003) presents a comprehensive listing of the applicable ASTM standards. Tests for measuring soil resistivity are described in ASTM G57 (ASTM, 2012i) and ASTM G187 (ASTM, 2012j).

7 CONCRETE REPAIR AND REMEDIATION

7.1 Technical Background

If degradation of concrete structures compromises their ability to perform their intended safety functions, they are required to be repaired in order to meet the dry cask storage system (DCSS) design bases. For Title 10 of the *Code of Federal Regulations* (CFR) Part 50 or Part 72 license holders, repairs are typically undertaken within the scope of the corrective actions program, subject to the quality assurance requirements of the governing regulation. Concrete structure repair techniques are well developed and in fact have already been applied to a small number of DCSSs, notably at the Three Mile Island (TMI) Unit 2 Independent Spent Fuel Storage Installation in Idaho. Cracks on the outer surface of the storage module were repaired using the resin injection method and an sealer applied to the outer concrete surface. In other instances, polyurethane foam was used to prevent water intrusion at anchor bolts.

Typically, concrete repairs involve the following steps:

1. Determination of the cause of damage

2. Evaluation of the extent of damage on the functional and performance requirements of the structure

3. Evaluation of the need to repair

4. Selection of the repair method

5. Preparation of the concrete for repair

6. Application of the repair method

7. Curing of the repair

8. Acceptance testing of the repaired area

Degraded concrete may need to be removed as part of the repair process (Axon, 1986). Approaches for removing concrete from the DCSS may be different than those used for nonnuclear applications such as roads, bridges, or civil structures because of access limitations in high radiation dose areas, and the need to maintain safety functions even while the repair is ongoing. There are likely to be more options for repairing pads than for shielding structures that encompass loaded fuel casks, and certain methods may not be appropriate for such systems. High pressure hydroblasting at water pressures ranging from 544 to 1,021 atm [8,000 to 15,000 psi] is considered to be a useful technique for removing unsound concrete without causing microfracture of the sound concrete (Perkins, 1986). Impact concrete removal techniques (e.g., jackhammering or bush-hammering) have been widely used, but these repair methods can produce concrete microfractures. In regard to shallow surface degradation {typically less than 1.27 cm [0.5 in] deep}, concrete damage is usually removed by shot or dry-wet blasting or by scrabblers. For repairs with exposed steel reinforcements, all scales and rust poorly bonded to the steel should be removed by wire brushing, high pressure water, or sand blasting (Delange, 1980). However, if the cross section of the reinforcing steel has corroded more than 75 percent of its original diameter, the affected steel should be removed and replaced in agreement with ACI 318-05 (ACI, 2005a).

Concrete repair is achieved through a variety of methods, depending upon the type and extent of damage needing repair [(NIST, 2013); ACI 562, (ACI, 2013c); ACI 506R, (ACI, 2005b)]. The selection of the proper repair method is of vital importance to eliminate future concrete damage. To avoid substandard repairs, there are general requirements for workmanship, procedures, and materials selection (Schutz, 1981). It is important that the damaged concrete is repaired in a timely manner, which depends on the rate of deterioration and how the damage affects the serviceability of the structure. For instance, structural cracks due to foundation settlement and freeze and thaw degradation may require repair as soon as the damage is detected (Smoak, 2002). For perceived durability rating, repairs of small dormant concrete cracking are typically conducted using penetrating sealers, epoxy treatment, or the application of an overlay or membrane on the concrete surface. For dormant large cracks, epoxy injection is considered to be the preferred repair method. For active cracks, installation of expansion joints and penetrating sealers are two common repair methods. For shallow concrete spalling {less than 20 mm [0.79 in]}, polymer or Portland cement-based grouts are used, whereas Portland cement concretes and polymer overlays are used for deep spalling. Corrosion-inhibiting admixtures can also be included in the concrete patch material. If the steel reinforcement is corroded, the corrosion products should be removed and the steel coated with a barrier material (e.g., epoxy resin). The physical properties of a particular product must be carefully reviewed to ensure that the necessary features are included.

Application of the basic remedial measures strategy includes the repair of damaged concrete and mitigation of the cause of deterioration. Concrete remedial measures typically address the corrosion damage of the steel reinforcement in concrete, which involve the repassivation of the steel. Steel repassivation can be achieved by using alkaline patch materials (e.g., mortar or concrete based), concrete electrochemical realkalization, and electrochemical chloride removal. More detailed information on each of these principles is provided elsewhere [Technical Committee 124-SRC (Technical Committee, 1994); ACI 222R-01 (ACI, 2001)].

7.2 Summary of Workshop Discussion and Expert Panel Assessment

Concrete repair and remediation is normally conducted on concrete structures by removing the portion of damaged concrete and restoring its structural integrity to extend the service life of the structure. Drs. Berke and Xi and Mr. James provided examples of concrete repair procedures used for other applications. For instance, highway and dam structures are commonly repaired and their repair methods are well understood (UDOT, 2012; AASHTO, 2010; AASHTO, 2008). Drs. Xi and Berke and Mr. James mentioned that the most important aspect in the repair process is the selection of the correct repair system. Drs. Jacobs, Xi, and Berke indicated that the repair methods presented here can be applicable to DCSSs. However, the effects of radiation present in the DCSS on the application of a repair have to be considered.

Mr. James indicated that structure repairs typically consider the performance-based analyses. For instance, when a concrete patch or thin overlay of concrete is applied, there is a pressing need to determine whether the patch was well bonded to the concrete while maintaining low shrinkage and strength when compared to the base concrete. Dr. Berke mentioned that the International Concrete Repair Institute (ICRI) presents the guidelines for concrete repairs (ICRI, 2009; 2008; 2006a,b; 2004; 1997a,b; 1996). Dr. Berke added that prior to performing a repair, the root cause of the concrete damage needs to be identified, as well as how the repair potentially affects other degradation modes. For instance, if a damaged concrete area was associated with alkali-silica reactions (ASRs) and the concentration of chlorides around that area is low, then the intended repair will not likely affect the corrosion behavior of the concrete. Dr. Berke also mentioned that after the repair is concluded, visual observation [ACI 201.1R, (ACI, 2008a)] of the repaired area is needed. Typically, no other inspection methods are required to assess the condition of the repair.

Drs. Xi and Berke described several concrete remediation approaches. The type of remediation and repair method to be used, however, depends on the type of concrete damage. For instance, Drs. Xi and Berke indicated that if corrosion is the only observable degradation of concrete, sacrificial anodes can be embedded into the concrete to protect the reinforcing steel. The sacrificial anodes can deliver a cathodic potential so that the corrosion rate of the steel is reduced. If corrosion is identified at early stages, the prompt implementation of a cathodic protection system can help preserve the structure intended functions. Dr. Berke added that if moisture is affecting the performance of the concrete, moisture barriers applied as topical treatments can be effectively used. In addition, Drs. Berke and Popovics stated that electrochemical chloride extraction can be used to restore the alkalinity of the concrete and reduce the chloride concentration around the reinforcing steel. This method has been shown to arrest corrosion, but its long-term effectiveness is questionable.

7.3 Considerations for Determining Acceptance Criteria

NUREG–1927, Revision 1 (NRC, 2015c) references acceptable concrete repair guidelines for cracking, spalling, and scaling in ACI 224.1R (ACI, 2007a), ACI 364.1R (ACI, 2007b), ACI 506R (ACI, 2005b), and ACI 562 (ACI, 2013c). In addition, ACI 349.3R (ACI, 2010a) provides general guidelines before, during, and after conducting a repair, including the required documentation, evaluation, and execution of the repair procedure. Specifications for repair materials include ASTM C928 (ASTM, 2013d) and ASTM C881 (ASTM, 2014c). The integrity of concrete repairs could be tested following ASTM C1583 (ASTM, 2013e). The licensee's selection of repair method will depend on a number of factors, including the size of the area or volume affected by degradation; the location of degradation; its effect on the safety function of the structures, systems, and components (SSC); and the anticipated duration that the repair is expected to remain effective. A licensee may choose to implement a repair that would not last as long as the licensed life of the system, provided that there is a program in place to periodically assess the condition of the repair and implement corrective actions if it no longer meets certain acceptance criteria. As discussed previously, repairs are subject to the licensee's quality assurance requirements under 10 CFR Part 50 or Part 72 regulations, as appropriate.

8 AGING MANAGEMENT PROGRAMS

8.1 Technical Background

An aging management program (AMP) is defined in the Title 10 of the *Code of Federal Regulations* (CFR) 10 CFR 72.3 as a program for addressing aging effects that may include prevention, mitigation, condition monitoring, and performance monitoring. Descriptions of AMPs are required to be in the Safety Analysis Report accompanying a renewal application for a specific license or Certificate of Compliance (CoC), according to 10 CFR Part 72.240. Guidance on the form and content of AMPs for dry cask storage systems (DCSSs) is found in NUREG–1927, Revision 1 (NRC, 2015c), which is generally similar to that for reactor license renewal in NUREG–1800, Revision 2 (NRC, 2010d) and NUREG–1801, Revision 2 (NRC, 2010b). As discussed in NUREG–1927, Revision 1 (NRC, 2015c), an effective AMP prevents, mitigates, or detects the aging effects and helps predict the extent of the effects of aging and timely corrective actions before there is a loss of intended function. It is described as comprising 10 primary elements:

1. Scope of program
2. Preventive actions
3. Parameters monitored or inspected
4. Detection of aging effects
5. Monitoring and trending
6. Acceptance criteria
7. Corrective actions
8. Confirmation process
9. Administrative controls
10. Operating experience

For both DCSSs and reactor systems, many AMPs are based upon the implementation of consensus codes and standards that the U.S. Nuclear Regulatory Commission (NRC) has determined to be applicable to safety-related structures, systems, and components (SSCs). The use of consensus codes and standards, when practicable, is in keeping with NRC policy for implementation of the National Technology Transfer and Advancement Act of 1995, as discussed in further detail in NRC Management Directive 6.5. The generic AMP for reinforced concrete structures in NUREG–1927, Revision 1 (NRC, 2015c) relies on periodic visual examination of accessible surfaces, as described in ACI 349.3R (ACI, 2010a), and groundwater monitoring consistent with the acceptance criteria in the ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL (ASME, 2001). These are similar to the approaches for reactor concrete structures in NUREG–1801, Revision 2 (NRC, 2010b); namely, in AMPs XI.S2, “ASME Section XI, Subsection IWL” and XI.S6, “Structures Monitoring.”

Key attributes of ACI 349.3R (ACI, 2010a) include the establishment of inspection frequencies and evaluation criteria. With respect to the former, the code specifies a 5-year inspection interval for visual examination of exposed or accessible structural surfaces, and a 10-year interval for below-grade structures. Inspection is intended to identify conditions such as leaching, chemical attack, erosion, voiding, cracking, reinforcing bar corrosion, settlement, or other evidence of physical degradation. If there is no indication of such degradation, the structure is characterized as being in a Tier 1 condition, suitable for continued service with no further evaluation needed. If there is evidence of degradation, the structure may be characterized as being in a Tier 2 condition if the degradation appears to be inactive. More frequent inspections or structural integrity analyses may be warranted. If the structure cannot meet the Tier 2 criteria, analyses may be needed to determine whether repair or replacement of the affected structure is necessary. The intent of the groundwater acceptance criteria in ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL (ASME, 2001) is to distinguish between those sites that could manage the effects of the groundwater species by a generic AMP and those that would need a site-specific AMP because of a particularly aggressive condition.

8.2 Summary of Workshop Discussion and Expert Panel Assessment

The current approach in the AMPs to evaluating the accessible (outer and remote access) concrete surfaces in DCSSs is by performing visual inspection every 5 years [ACI 349.3R (ACI, 2010a)]. As part of the routine walkdowns conducted periodically, the licensee examines the overall appearance of the outer concrete surfaces. If during those periodic inspection activities concrete damage is found that does not meet the ACI 349.3R (ACI, 2010a) acceptance criteria, it is entered into the licensee's Corrective Action Program (CAP) for evaluation and disposition. To that end, Drs. Xi, Berke, and Jacobs indicated that, while the concrete may still be functionally adequate, repair (ACI, 2007a; NIST, 2013) and mitigation [ACI 349.3R (ACI, 2010a)] activities may be needed to prevent further degradation in the future. Drs. Berke, Popovics, and Jacobs indicated that periodic visual inspection should be able to capture any degradation mechanism before the structure is deemed to lose the intended functions. Mr. James mentioned that the strategy for visual inspection should include determining the time at which more detailed assessments should be done.

Dr. Xi provided an example of more frequent visual inspection for bridges, indicating that the Federal Highway Administration (FHWA) requires bridge owners to entirely inspect their bridges every 2 years. This FHWA requirement is based on a law from Congress and corresponding regulations (23 CFR Part 650), not technical assessment, which would have likely determined a longer inspection interval to be adequate. Drs. Xi and Jacobs indicated that it is important to realize that bridges are considered active systems where mechanical fatigue and load changes are frequent, whereas for the passive DCSSs, there is generally no mechanical fatigue or changes in load.

However, Dr. Xi indicated that the 5-year inspection interval may be appropriate for a new DCSS. Dr. Xi indicated that if the cask is already 20 or more years old, another 5 years may cause extensive cracking and related damages. On the other hand, Dr. Berke suggested that if the outside concrete surface shows absence of degradation, then the inside concrete surface would likely be in good condition assuming that gamma and neutron radiation is within the limits established by ACI 349.3R (ACI, 2010a). Per these statements, Dr. Berke recommended inspections every 5 years but suggested the inclusion of opportunistic inspections when inspecting other concrete components.

Drs. Xi, Berke, and Jacobs stressed that after the initial visual signs of concrete damage, a reactive program has to be in place to repair and mitigate the damaged concrete. Likewise, Drs. Xi, Berke, and Jacobs indicated that internal concrete degradation may not be visible from the outside concrete surface and that performing only visual inspection will most certainly miss the degradation initiation and propagation inside the concrete. However, Drs. Xi, Berke, Jacobs, and Popovics indicated that visual inspection is essential for concrete structures. They also stated visual inspection would adequately detect concrete degradation before there is a loss of intended function, even though by itself visual inspection could not assess and quantify concrete damage in sufficient detail. As such, these panelists suggested complementing visual inspection with some other monitoring techniques to assess and quantify concrete damage for DCSSs.

In terms of the number of casks to be inspected after loading, the expert panel discussed progression of priorities. The first priority presented by Dr. Jacobs is to examine a cask that has experienced some unusual circumstances (e.g., fire or flooding) over its life on a particular site. In addition, inspection has to focus on a cask with the longest service time with stored fuel. Another priority presented by Dr. Jacobs would be to examine the cask with the largest internal temperature or water ponding as water is known to affect most of the concrete degradation mechanisms. The number of casks to be inspected depends on several factors, such as the origin and time of acquisition of the cask. Dr. Jacobs also stated that irrespective of the similarities in environmental conditions and concrete design, cask inspections should be conducted at each site. In addition, Drs. Jacobs and Popovics suggested that construction of a database with information about the condition of the casks at each site along with environmental and other information would be useful to better identify concrete degradation and prognosticate its durability.

For below-grade concretes, the proposed monitoring frequency is set to 10 years [ACI 349.3R, (ACI, 2010a)]. Dr. Berke indicated that if slab degradation occurs, the below-grade areas more susceptible for damage are the foundation corners due to larger exposed surface area. As mentioned in Chapter 6 of this report, Drs. Berke and Popovics and Mr. James indicated that to perform inspection of below-grade slabs, partial excavation may be required as the first step in the monitoring of these underground structures. For instance, removal of the top metric [3 in] of soil may be sufficient to obtain a degradation assessment if a more extensive excavation is required. Drs. Berke and Popovics and Mr. James also indicated that extensive soil excavation should not be performed every 10 years. Instead, if subgrade concrete damage is noted, then extensive excavation could be necessary after a shallow soil inspection surrounding the slab is performed.

As indicated in Section 6.3 of this report, Drs. Berke, Popovics, and Xi indicated that it would be valuable to broaden the monitoring of aggressive species in soil or groundwater. They agreed that groundwater and soil chemistry should be monitored for chloride, sulfate, phosphate, and magnesium ions. In terms of the threshold concentrations of aggressive species in the groundwater, the ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL (ASME, 2001) establishes an acceptance criterion for a maximum chloride concentration of 500 ppm (as long as the concrete does not carbonate) and a maximum sulfate concentration of 1,500 ppm. These panelists agreed that these concentrations are appropriate for concrete degradation in soil. However, Drs. Berke, Popovics, and Xi did not discuss concentration limits for magnesium and phosphate in soil and groundwater nor the suitable sampling locations and sampling size. For instance, an increase in groundwater sampling size and frequency in addition to monitoring additional groundwater species (e.g., $FeCl_3$, $MgCl_2$, $CaCl_2$) may be required if an aggressive groundwater is detected.

8.3 Considerations for Aging Management Programs

The intent of the NRC staff review of the licensees' AMPs in the renewal application review is to confirm that the licensee has adequately identified the range of degradation phenomena that could potentially affect the SSCs, that the licensee will implement inspection or monitoring programs that will actually detect the presence of degradation prior to the loss of the intended safety function, that the licensee will implement corrective actions to mitigate further damage or to repair or replace an affected component, and that the licensee will consider facility-specific or industry-wide operational experience to inform enhancements or modifications to the AMP. Specific technical issues related to potential degradation phenomena, inspections, monitoring, and repairs are largely discussed in Chapters 4 through 7 of this report.

The actual implementation of the AMP is not directly addressed in NUREG–1927, Revision 1 (NRC, 2015c) but may involve a proposal to inspect only one or a small fraction of the cask systems at a particular site, rather than each one, or even to assign credit to inspections performed at other sites. Such an approach would necessarily involve analyses to demonstrate that the systems which would be inspected bound the conditions of entire set of systems. In other words, the systems that are inspected should be those which are most likely to experience degradation. Factors to consider when assessing the potential for degradation may include the age, the extent of moisture exposure, the extent of exposure to chlorides or other aggressive species, the temperature history, and the experience of any unusual events, such as flooding or seismic activity. If different conditions favor different modes of degradation (e.g., if one mode occurs preferentially at higher temperatures and another mode at lower temperatures), a single system may not be able to bound the conditions at a particular site. Further, given variations in system designs and environments at different sites, it may prove even more challenging to demonstrate that inspections at one site can serve as a surrogate for another. In any event, the licensees will be required to provide an adequate technical basis to support whatever is their proposed approach for the AMP implementation. Industry may issue its own generic guidance as, for example, set forth in Nuclear Energy Institute 14-03 (NEI, 2014) or as Electric Power Research Institute has done for a number of aging management approaches for reactor components. Such guidance may be submitted to NRC for review and endorsement.

9 TIME-LIMITED AGING ANALYSES

9.1 Technical Background

A time-limited aging analysis (TLAA) is defined in Title 10 of the *Code of Federal Regulations* (CFR) 10 Part 72.3 as a licensee or certificate holder calculation and analysis that

1. Involves structures, systems, and components (SSCs) important to safety within the scope of the license renewal
2. Considers the effects of aging
3. Involves time-limited assumptions defined by the current operating term (for example, 40 years)
4. Was determined to be relevant by the licensee or certificate holder in making a safety determination
5. Involves conclusions or provides the basis for conclusions related to the capability of SSCs to perform their intended safety functions
6. Is contained or incorporated by reference in the design bases

Descriptions of TLAAs are required to be in the safety analysis report accompanying a renewal application for a specific license or Certificate of Compliance (CoC), according to 10 CFR Part 72.240. In other words, NUREG–1927, Revision 1 (NRC, 2015c) describes TLAAs as calculations or analyses used to demonstrate that in-scope SSCs will maintain their intended function throughout the period of extended operation. A common example is a fatigue life evaluation.

NUREG–1927, Revision 1 (NRC, 2015c) does not prove specific guidance on the form and content of a TLAA, but generic TLAAs for license renewal of reactor systems are found in NUREG–1800, Revision 2 (NRC, 2010d) and NUREG–1801, Revision 2 (NRC, 2010b). The TLAAs are described as comprising the same 10 elements that make up an aging management program (AMP). However, rather than prescribing inspection practices to detect the effects of aging, the TLAA provides analytical methodologies to demonstrate that inspection is not needed, provided that certain acceptance criteria are met. Further, consistent with the approach for AMPs, TLAAs are often based upon the use of consensus codes and standards. For example, in the case of metal fatigue of reactor components, the TLAA references the design fatigue curves in the ASME Boiler and Pressure Vessel Code, Section III (ASME, 2007a) that is endorsed by current U.S. Nuclear Regulatory Commission (NRC) guidance provided in NUREG—1536 (NRC, 2010a). Potential uses for TLAAs in dry cask storage system (DCSS) concrete structures include the evaluation of irradiation effects and thermal fatigue, among others. With respect to the former, as discussed in Section 4.8, ACI 349.3R (ACI, 2010a) refers to a limit of 1×10^{17} neutrons/m^2 to prevent irradiation-induced degradation. For the latter, fatigue life curves for concrete and reinforcing bars are given in ACI 215R (ACI, 1997), “Considerations for Design of Concrete Structures Subjected to Fatigue Loading.”

9.2 Summary of Workshop Discussion and Expert Panel Assessment

As stated in Section 9.1, TLAAs have already been proposed for dispositioning the potential for radiation-induced damage, elevated temperature effects, and thermal cyclic effects on concrete structures (Calvert Cliffs, 2010; AREVA TN Americas, 2014). Drs. Popovics and Xi indicated that the lifetime limit of 1×10^{17} neutrons/m^2 provided in ACI 349.3R (ACI, 2010a) is conservative. The panelists identified some coupled effects that may be considered for performing the TLAAs. Dr. Berke indicated that radiation coupled with concrete temperature or other degradation modes can be a candidate for TLAAs. Concrete temperatures greater than 80 °C [176 °F] and high concrete pH (~13.0) can promote accelerated corrosion of the reinforcing steel, according to Dr. Berke. Dr. Berke also discussed the coupling between concrete cracks and corrosion of reinforcing steel in concrete, indicating that small cracks have a greater effect on the transport processes for high performance concrete than for normal concretes. Dr. Xi believes that accelerated carbonation due to neutron radiation can promote a decrease in pH as well as a change in the concrete microstructure. As a result, radiation effects and their interactions with other degradation modes may be accounted for in the form of a TLAA. Dr. Xi also discussed the effect of cations on the diffusion coefficient of chloride ions, indicating that the companion cation (e.g., Mg^{+2}, Ca^{+2}, and Na^{+}) of chloride salts plays a significant role in the overall diffusion of chlorides into concrete. Mr. James also suggested a TLAA to cover the effects of alkali-silica reactions (ASRs) coupled with radiation on the long-term concrete performance.

9.3 Considerations for Time-Limited Aging Analyses

The intent of the NRC staff review of the licensees' TLAAs is to verity that the assumptions, calculations, and analyses are adequate, bound the environment, and bound aging mechanisms or aging effects for the pertinent SSCs. The nature of this review will depend upon the aging effect in question, but will likely take into consideration the technical basis for any model or equation used for the analysis, the selection of inputs used for the analysis, the reliability of any computer code or program used to make calculations, and the applicability of any acceptance criteria to the specific design bases of the DCSS. As discussed in Section 9.1, the technical basis for the model can often be referenced from the consensus codes and standards that apply to the subject SSC. Beyond these, the licensee may use other information from the pertinent technical literature that can be readily reviewed by NRC staff. The selection of model inputs may involve the quantification of parameters that define the system conditions, such as the radiation flux, temperature, and stress state. The licensee should be able to justify the selection of values based on analyses of the DCSS design and to describe associated uncertainties. If a computer code or program is used, the licensee may provide supporting references to show that it has been verified and validated for the calculations which will be performed, for instance, following ANSI/ANS-10.4 (ANSI, 2008). Finally, the licensee should demonstrate that there is a safety margin between the acceptance criteria and the conditions under which the SSC will no longer be able to perform the intended safety function. This will allow sufficient time for further analyses or other corrective actions to be implemented.

10 SUMMARY

10.1 Summary and Key Observations

In the context of extended operation up to 300 years of independent spent fuel storage installations (ISFSIs), the U.S. Nuclear Regulatory Commission (NRC) Office of Nuclear Material Safety and Safeguards (NMSS) in collaboration with the Office of Nuclear Regulatory Research (RES) and the Center of Nuclear Waste Regulatory Analyses (CNWRA®) organized and conducted an expert panel to provide insight into questions relating to degradation, inspection, monitoring, and analysis of concrete structures, systems, and components (SSCs). Experts were identified from industry and from academia in an effort to capture insights from the latest research as well as the practical perspective of constructing and maintaining concrete structures. The panel activities included two approximately 90-minute conference calls and one 2-day workshop. Two sets of written questions were posed to the experts, and the responses to these questions helped to formulate the discussion topics for the workshop.

The outcomes of the expert panel activities are documented in this report and can be topically summarized as:

- Concrete degradation mechanisms
- Prevention and mitigation strategies
- Monitoring and inspection
- Repair and remediation
- Aging management programs
- Time-limited aging analyses

While reviewing concrete degradation mechanisms, several useful conclusions were identified. First, all degradation mechanisms identified are expected to be operative and potentially significant to extended operation. Furthermore, because many of the mechanisms are temporally correlated either through chemical reaction or diffusion kinetics, the extended operation period increases the likelihood of degradation of the concrete SSCs. In addition, for the mechanisms identified, the appearance of visual distress (usually some sort of surface cracking or discoloration) would precede any significant loss of structural, shielding, or heat-removal function. There was some debate regarding the time between the emergence of visual indication and the significant loss of structural function for a few mechanisms mainly stemming from the nonuniformity and situation-specific details surrounding each degradation mode. This time is important because inspection intervals must be adequate to ensure detection of the degradation prior to loss of function.

Complementary to the degradation mechanisms identified, prevention and mitigation strategies were discussed for addressing degradation concerns. For the specific problem of extended storage in existing dry cask storage systems (DCSSs), many prevention strategies would be challenging and likely impractical because the systems are already in operation. However, strategies involving the removal or exclusion of moisture through barriers were identified as likely feasible beneficial strategies for mitigating concrete degradation. For new DCSSs, the experts recommended that the concrete constituents be selected and mix designs be formulated with long-term durability (as opposed to simply strength or shielding performance) in mind.

The expert panel provided useful insight into inspection and monitoring techniques for DCSSs. The importance of visual inspection was highlighted during the course of expert panel activities. No other technique provides as much useful information for evaluating the health of concrete structures. Various imaging and scanning techniques were discussed, and it was generally concluded that no one technique is suitable for all operative degradation modes. As such, careful consideration should be given to the relevance and validity of the data provided by the numerous nondestructive evaluation techniques.

Repair and remediation of concrete components in DCSSs was addressed in a general sense as the details of a particular degradation or aging effect greatly influence the particular repair techniques and effectiveness. It was noted that the efficacy of concrete repairs in other industries is typically assessed in a performance-based sense where capacity or performance are added back to a degraded structure. It was noted that any repairs must be closely monitored to ensure enduring performance of the repair materials.

The panelists considered that an aging management program (AMP) for concrete components of a DCSS that relies on periodic visual examination of accessible surfaces, in accordance with American Concrete Institute (ACI) 349.3R (ACI, 2010a), is generally adequate for detecting the progression of multiple degradation modes in DCSS concrete components. However, some panelists expressed concern about its adequacy for identifying the onset of degradation for some of the operative degradation modes. In addition to visual inspection, the panelists considered that an AMP requiring periodic groundwater chemistry monitoring can serve to identify aggressive subgrade conditions. While the panelists agreed with the acceptance criteria in the generic AMP of NUREG–1927 (NRC, 2015c) for aggressive groundwater conditions, concern was raised about the lack of identified solution limits for other aggressive species in addition to chlorides and calcium sulfate. The panelists suggested that an increase in groundwater sampling size and frequency in addition to monitoring additional groundwater species (e.g., $FeCl_3$, $MgCl_2$, $CaCl_2$) could be corrective actions to be taken if the acceptance criteria in the generic AMP are not met. One key takeaway for NRC staff that is discussed in greater detail in Section 8.2 is the need to engage with the relevant code committee to ensure the identified limits are appropriate based on the latest research and to capture all potentially significant species.

Certain types of concrete degradation were identified as most likely suitable for time-limited aging analyses (TLAAs). Radiation-induced concrete degradation was identified as a clear candidate for TLAAs. Many other degradation modes are temporally correlated and therefore could be considered for TLAAs; however, the lack of robust phenomenological models for some degradation types and difficulty in characterizing future environmental stimuli limit predictive capability. Careful scrutiny should be applied to using TLAAs for degradation modes other than radiation effects.

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APPENDIX A
QUESTIONNAIRE FOR EXPERT PANEL MEMBERS—LIST OF GENERIC QUESTIONS FOR EACH TOPICAL AREA

APPENDIX A: QUESTIONNAIRE FOR EXPERT PANEL MEMBERS—LIST OF GENERIC QUESTIONS FOR EACH TOPICAL AREA

Please place your responses into the boxes provided. Don't be overly concerned with the format of any supporting references, but do ensure that there is enough information to locate the source.

Author (Last Name; First Name):

Topic 1: Evaluation of Degradation Mechanisms

Question 1.1: Is the list of degradation mechanisms for concrete structures in the body and Appendix A8 of the TIN report [ML14043A402] complete and correct? If not, what should be added or removed? Why?

Response:

References:

Topic 2: Detection and Monitoring of Degradation Mechanisms

Question 2.1: Which degradation mechanisms listed in the TIN report (or others that you think are relevant) can be detected by active/online monitoring or periodic inspection of the DCSS?

Response:

References:

Question 2.2: What are the currently available techniques for active monitoring or periodic inspection of the DCSS, and which ones should be avoided (if any) for each degradation mechanism? Consider the constraints imposed by the system design and environmental

conditions (e.g., geometry, space limitations, and high ionizing radiation of DCSS) and operational limitations (e.g., temperature and radiation tolerances). Indicate if the methodology is incorporated in a consensus standard (e.g., ASME, ACI) or if the standard would need to be modified for applicability to the DCSS.

Response:

References:

Question 2.3: Do the active monitoring or periodic inspection techniques provide information only about whether degradation is occurring (i.e., it is or is not), or information about the extent of progression?

Response:

References:

Question 2.4: If a degradation mechanism cannot be detected by active monitoring or periodic inspection, why not?

Response:

References:

Question 2.5: Are any of the degradation mechanisms detectable by indirect measurements (e.g., groundwater chemistry)? Are the acceptance criteria in ASME Section XI,

Subsection IWL (NUREG 1801 AMP XI.S6) sufficient, or in what ways should it be augmented?

Response:

References:

Topic 3: Repair/Remediation/Replacement

Question 3.1: Are there standard methods and acceptance criteria for mitigation and/or repair of the degradation mechanisms listed in the TIN report (or others that you think are relevant)? Also consider if a combination of degradation phenomena might influence repair options.

Response:

References:

Question 3.2: Are there standard criteria for determining when repair is no longer sufficient (i.e., replacement of the structure is required)?

Response:

References:

APPENDIX B
QUESTIONNAIRE FOR EXPERT PANEL MEMBERS—LIST OF SPECIFIC QUESTIONS

APPENDIX B: QUESTIONNAIRE FOR EXPERT PANEL MEMBERS—LIST OF SPECIFIC QUESTIONS

Please place your responses into the boxes provided. Don't be overly concerned with the format of any supporting references, but do ensure that there is enough information to locate the source.

Also, with each question, consider whether your response would be different, for any reason, in the differing time periods of interest, 20-60 years and 60-300 years.

Author (Last Name; First Name):

Question 1: NRC guidance suggests that at the end of initial licensing period, a cask system (e.g., metallic fuel canister and concrete shielding structure) may be inspected to confirm that the components are still performing their intended safety function. The results of the inspection can be used to support the licensee's safety basis for license renewal. To minimize expense and dose exposure, the licensee may wish to select a single system thought to be the most susceptible to degradation, for instance because it is the oldest or hottest, thereby bounding the performance of the other systems on site. Inspections to date for concrete structures have relied on visual examination.

Considering potential concrete degradation phenomena, what criteria would you suggest for selecting a cask system or systems for inspection to best ensure that relevant concrete degradation mechanisms would be identified? Is likely that a single cask system could be identified that satisfies all of the preferred criteria or may there be a need to select multiple systems to satisfy the criteria for different degradation phenomena?

Response:

References:

Question 2: Can radiation effects on concrete be addressed through TLAA?

- Are the ACI 349-3R lifetime limits of 1×10^{17} neutrons/m^2, 10^{10} Gy adequate to ensure safe operation?

- Should any additional parameters be considered for the lifetime limit of neutron fluence (e.g., range of neutron energies, type of fuel)?

Response:

References:

Question 3: For concrete exposed to temperatures in excess of the ACI 349 design limits (150F general and 200F local) for extended periods (years), what degradation would be expected?

Response:

References:

Question 4: Determine if an AMP relying on (i) visual inspection (per ACI 349.3R tier acceptance criteria, or more restrictive), (ii) groundwater chemistry monitoring, and (iii) radiation surveys is adequate (and sufficient) to manage the following aging mechanisms for both above-grade and below-grade areas:

- Freeze-thaw
- Chemical attack (Cl, SO4 induced)
- Aggregate reactions/expansion
- Corrosion of embedded steel
- Leaching of Ca(OH)2
- Long term settlement

Response:

References:

Question 5: Determine if a groundwater chemistry program is sufficient for managing below-grade (underground) effects related to corrosion of embedded steel and chemical attack (pH, Cl, SO_4 induced).

Response:

References:

Question 6: Determine if further clarification is needed for the definition of below-grade structures, e.g. structure in direct contact with soil, underground, or any alternative criteria.

Response:

References:

Question 7: Identify monitoring techniques that could be useful for monitoring of below-grade structures.

Response:

References:

Question 8: Given the large concrete surface area in a typical ISFSI, an applicant may propose inspecting a subset of the concrete SSCs. This could include "opportunistic inspection" for very difficult to inspect concrete SSCs. Are there examples from other industries and structures where sampled inspections are effectively used? Please comment on sampled inspection for each "type" of ISFSI concrete:

- Above grade, accessible
- Above grade inaccessible
- Below grade

Response:

References:

Question 9: Critique or help define the technical basis for inspection frequency (ACI 349.3R):

- Determine if 5 year inspections for above-grade inaccessible areas is adequate.

- Determine if 10 year inspections for below grade structures is adequate.

- Determine if "opportunistic inspection" could be appropriate in contrast to frequency based.

- What is an acceptable frequency for groundwater monitoring?

Response:

References:

Question 10: Determine if specific areas require increased attention during inspections (e.g., where steel attachments might tie into the concrete, increased probability of freeze-thaw degradation)

Response:

References:

Question 11: Determine if additional guidance is required for qualified groundwater chemistry characterization method, including requirements for periodic calibration.

Response:

References:

Question 12: Are there any degradation modes that could be considered “geographically regionalized?”

Response:

References:

Question 13: In the case that an aggressive groundwater/soil environment is determined (Low pH or high deleterious ion concentration) and an enhanced/modified AMP is required:

- What would be a suitable inspection frequency for these focused inspections? What areas should be evaluated?
- What critical reinforced concrete parameters should be monitored?
- Should there be additional parameters to evaluate when monitoring aggressive groundwater (e.g. corrosion products, calcium). What would be suitable threshold values for these parameters (refer to specific codes and standards when possible)?
- Define suitable NDT/NDE or destructive methods for ensuring SSC is able to maintain safety functions.

Response:

References: