Assessment and Management of Aging of Nuclear Power Plant Safety-Related Structures

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ABSTRACT

Background information and data have been developed to assist in quantifying the effects of age-related degradation on the performance of nuclear power plant (NPP) safety-related structures. Factors that can lead to age-related degradation of safety-related structures are identified and their manifestations described. Current regulatory testing and inspection requirements are reviewed and a summary of degradation experience is presented. Techniques commonly used to inspect NPP concrete structures to assess and quantify age-related degradation are summarized. An approach for conduct of condition assessments of structures in NPPs is presented. Criteria, based primarily on visual indications, are provided for use in classification and assessment of concrete degradation. Materials and techniques for repair of degraded structures are noted and guidance provided on repair options available for various forms of degradation. A probabilistic methodology for condition assessment and reliability-based life prediction has been developed and applied to structures subject to combinations of structural load processes and to structural systems. The methodology has also been used to investigate optimization of in-service inspection and maintenance strategies to maintain failure probability below a specified target value as well as to minimize costs. Fragility assessments involving analytical solutions and finite-element methods have been utilized to predict the effect of aging degradation on structural component performance.

KEY WORDS: nuclear power plants , aging management, nondestructive evaluation, in-service inspection, condition assessment, degradation, structural engineering, time-dependent reliability assessments.

INTRODUCTION

The Atomic Energy Act and USNRC regulations limit commercial reactor power licenses in the U.S. to an initial 40-year period, but also provide the option of renewing the licenses. The original 40-year licensing period was primarily based on the amortization period generally used by electric utility companies for large capital investments, and not on safety, technical, or environmental issues. Due to this selected time period, however, some utilities may have engineered some of the structures and components based on an expected 40 year service life. Currently in the U.S. 103 NPP's are licensed for commercial operation and provide about 20% of all electrical power produced. About one-quarter of these plants are under 20 years old, but more than one-third are 30 or more years old. Starting in the year 2006, the first of the initial operating licenses for these plants will expire. By the year 2010 approximately 10% of the initial operating licenses are scheduled to expire and by the year 2015 over 40% of the current plants will have reached the end of their initial operating license period. In an effort to continue furnishing a timely and cost-effective solution to electricity production requirements as well as provide an environmentally clean energy source, most utilities are expected to seek a renewal of the initial operating license for their plants. Indeed, at least forty-eight of the currently operating units either have been through the license renewal process, submitted an application to renew their operating license, or announced that they intend to do so.

The importance of nuclear power and the necessity for ensuring continued satisfactory operation is clear. One of the key concerns that could affect the continued operation and development of nuclear power relates to the impact of aging of the plants on plant performance. Nuclear power plants are designed, built, and operated to standards that aim to reduce the likelihood of release of radioactive materials to levels as low as reasonably achievable. A NPP, however, involves complex engineering structures and components operating in demanding environments that potentially could challenge the high level of safety (i.e., safety margin) required of the plant throughout its service life. Safety of these structures is also a concern during decommissioning, in which a staged approach may be used that would return the sites to "green field" conditions. Throughout any decommissioning period, the safety-related structures must continue to meet several of their intended functions (e.g., leaktightness and shielding). Age-related degradation may affect the engineering properties, structural resistance/capacity, failure mode, and location of failure initiation that may in turn affect the ability of a structure to withstand challenges in service. It is necessary that safety issues related to plant aging and continued service be resolved through sound scientific and engineering understanding. Furthermore, in contrast to many mechanical and electrical components, replacement of many structures is impractical.

DEGRADATION AND OPERATING EXPERIENCE

Service-related degradation can affect the ability of a NPP civil structure to perform satisfactorily in the unlikely event of a severe accident by reducing its structural capacity or jeopardizing its leaktight integrity. The root cause for most degradation can generally be linked to a design or construction problem, inappropriate material application, a base metal or weld-metal flaw, maintenance or inspection activities, or excessively severe service conditions.

Steel structure degradation can be classified as either material or physical damage. Material damage occurs when the microstructure of the metal is modified causing changes in its mechanical properties. Material damage to a NPP containment metallic pressure boundary (i.e., steel containment and liner of reinforced concrete containment) is not considered likely, however. Physical damage occurs when the geometry of a component is altered by the formation of cracks, fissures, or voids, or its dimensions change due to overload, buckling, corrosion, erosion, or formation of other types of surface flaws. Changes in component geometry, such as wall thinning or pitting caused by corrosion, can affect structural capacity by reducing the net section available to resist applied loads. In addition, pits that completely penetrate the component can compromise the leaktight integrity of the component. Physical degradation due to either general or pitting corrosion represents the greatest potential threat to the containment metallic pressure boundary.

Primary mechanisms that can produce premature deterioration of reinforced concrete structures include those that impact either the concrete or steel reinforcing materials (i.e., mild steel reinforcement or post-tensioning system). Degradation of concrete can be caused by adverse performance of either its cement-paste matrix or aggregate materials under chemical or physical attack. Chemical attack may occur in several forms: efflorescence or leaching; attack by sulfate, acids, or bases; salt crystallization; and alkali-aggregate reactions. Physical attack mechanisms for concrete include freeze/thaw cycling, thermal expansion/thermal cycling, abrasion/erosion/cavitation, irradiation, and fatigue or vibration. Degradation of mild steel reinforcing materials occurs mainly as a result of corrosion. Post-tensioning systems are susceptible to corrosion plus loss of prestressing force, primarily due to tendon relaxation and concrete creep and shrinkage.

As NPPs age, degradation incidences are starting to occur at an increasing rate, primarily due to environmental-related factors [1]. There have been at least 32 reported occurrences of corrosion of steel containments or liners of reinforced concrete containments. In two cases, thickness measurements of the walls of steel containments revealed areas that were below the minimum design thickness. Two instances have been reported where corrosion has completely penetrated the liner of reinforced concrete containments. Examples of specific problems identified include corrosion of the steel containment shell in the drywell sand cushion region (Oyster Creek), shell corrosion in ice condenser plants (Catawba and McGuire), corrosion of the torus of the steel containment shell (Fitzpatrick, Cooper, and Nine Mile Point Unit 1), and concrete containment liner corrosion (Brunswick, Beaver Valley, North Anna 2, Brunswick 2, and Salem). Transgranular stress corrosion cracking in bellows has also occurred (Quad Cities 1 and 2, and Dresden 3).

With respect to concrete structures, at least 34 occurrences of degradation have been reported. Causes were primarily related to improper material selection, construction/design deficiencies, or environmental effects. Age-related degradation occurrence examples include failure of prestressing wires (Calvert Cliffs); corrosion of steel reinforcement in water-intake structures (Turkey Point and San Onofre); leaching of tendon gallery concrete (Three Mile Island); and low prestressing forces (Ginna, Turkey Point 3, Zion, and Summer).

TESTING AND INSPECTION

One of the conditions of all operating licenses for water-cooled power reactors is that the primary reactor containment shall meet the containment leakage test requirements set forth in Appendix J ("Primary Reactor Containment Leakage Testing for Water-Cooled Power Reactors") to 10 CFR Part 50 [2]. These test requirements provide for preoperational and periodic verification of the leaktight integrity of the primary reactor containment, and systems and components that penetrate containment of water-cooled power reactors, and establish the acceptance criteria for such tests. On September 26, 1995, the USNRC amended Appendix J to provide a performance-based option for leakage-rate testing. The amendment is aimed at eliminating prescriptive requirements that are marginal to safety and providing licensees greater flexibility for cost-effective implementation methods for regulatory safety objectives. Thus, either Option A—*Prescriptive Requirements* or Option B — *Performance-Based Requirements* can be chosen by a licensee to meet the requirements of Appendix J. Option B allows licensees with good integrated leakage-rate test performance histories to reduce the Type A (i.e., primary reactor containment overall integrated leakage rate) testing frequency from three tests in ten years to one test in 10 years. However, a general inspection of accessible interior and exterior surfaces of the containment, structures, and components must be performed prior to each Type A test and during two other refueling outages before the next Type A test.

Appendix J also requires a general inspection of the accessible interior and exterior surfaces of the containment structures and components to uncover any evidence of structural deterioration that may affect either the containment structural integrity or leaktightness. On August 8, 1996, the USNRC published an amendment to 10 CFR Part 50.55a ("Codes and Standards") to require that licensees use portions of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code [3] for containment in-service inspection. The regulations were amended to assure that critical areas of containments are routinely inspected to detect and to take corrective action for defects that could compromise structural integrity. The amended rule became effective September 9, 1996 with a five-year implementation period. Specifically, the rule requires that licensees incorporate the 1992 Edition of the ASME Code, Section XI, with the 1992 Addenda of Subsection IWE, "Requirements for Class MC and Metallic Liners of Class CC Components of Light-Water Cooled Power Plants," and Subsection IWL, "Requirements for Class CC Concrete Components of Light-Water Cooled Power Plants,"into their in service inspection plans. In addition, several supplemental requirements with respect to the concrete and metal containments were included in the rule (e.g., inspection of inaccessible areas, and prevention of duplicate examinations required by both the periodic routine and expedited examination program). Subsequently on August 3, 2001 the USNRC announced that it intends to amend 10 CFR Part 50.55a to incorporate by reference the 1997 Addenda, the 1998 Edition, the 1999 Addenda, and the 2000 Addenda of Section XI of the ASME Code. Comments on the proposed amendment are presently being addressed.

ASSESSMENT AND REMEDIATION OF AGED/DEGRADED CIVIL STRUCTURES

Operating experience has demonstrated that periodic inspection, maintenance, and repair are essential elements of an overall program to maintain an acceptable level of reliability for the civil structures over their service life. Knowledge gained from conduct of an in-service condition assessment can serve as a baseline for evaluating the safety significance of any degradation that may be present, and defining subsequent in-service inspection programs and maintenance strategies. Effective in-service condition assessment of civil structures requires knowledge of the expected type of degradation, where it can be expected to occur, and application of appropriate methods for detecting and characterizing the degradation.

The stability and durability of a civil structure can only be guaranteed when it has an appropriate safety margin against expected loads and environmental influences during its intended lifetime. In-service inspection programs for safety-related NPP civil structures have the primary goal of ensuring that these structures have sufficient structural margins to continue to perform in a reliable and safe manner [2,3]. A secondary goal is to identify environmental stressor or aging factor effects before they reach sufficient intensity to potentially degrade structural components.

Determining the existing performance characteristics and extent and causes of any observed distress is accomplished through a structural condition assessment. Routine observation, general visual inspections, leakage-rate tests, and destructive and nondestructive examinations are techniques used to identify areas of the NPP that have experienced degradation. Techniques for establishing time-dependent change, such as section thinning due to corrosion or changes in component geometry and material properties, involve monitoring or periodic examination and testing. Knowing where to inspect and what type of degradation to anticipate often requires information about the design features of the NPP as well as the materials of construction and environmental factors. Knowledge gained from a condition assessment serves as a baseline for evaluating the safety significance of any damage present and defining in-service inspection programs and maintenance strategies. Guidelines on conduct of a structural condition assessment of metallic and concrete structures are available [4,5]. Assessment of the magnitude and rate of occurrence of age- or environmental stressor-related degradation is often accomplished using nondestructive examination methods.

Nondestructive examination methods for metallic materials involve surface and volumetric inspections to detect the presence of degradation (i.e., coating deterioration, loss of section due to corrosion, or presence of cracking). The surface examination techniques primarily include visual, liquid penetrant, and magnetic particle methods. Volumetric methods include ultrasonic, eddy current, and radiography. Provisions are also included in the ASME Code for use of alternative examination methods provided results obtained are demonstrated to be equivalent or superior to those of the specified method. Acceptance standards are defined in Article IWE-3000 of the ASME Code. In order to obtain repeatable and reproducible nondestructive examination results using any of the methods noted above, several factors must be understood and controlled: material evaluated, evaluation procedure utilized, environment, calibration/baseline reference, acceptance criteria, and human factors. Brief descriptions of each of the above methods and a summary of applicability by flaw type and important material characteristics are provided elsewhere [6]. A suitable technique for inspection of inaccessible regions of the containment metallic pressure boundary needs to be developed. Some preliminary work in this area has been conducted [7].

Primary manifestations of distress that are present or can occur in reinforced concrete structures include cracking, voids, and delaminations, and strength losses. Reviews of the performance of NPP reinforced concrete structures indicates that concrete cracking and corrosion of embedded steel reinforcement are the primary manifestations of degradation reported [5]. Methods used to detect discontinuities in concrete structures generally fall into two categories: direct and indirect. Direct methods involve a visual inspection of the structure, removal/testing/analysis of material(s), or a combination of the two. Indirect methods generally measure a parameter from which an estimate of the extent of degradation can be made through existing correlations. Most nondestructive testing methods for concrete are indirect and quite often evaluation of concrete structures requires a combination of test methods, as no single testing technique is available that will detect all potential degradation factors. Indirect methods are effective in indicating the relative quality of concrete and identifying concrete cracking, voids, and delaminations, but tend to be more qualitative when it comes to determination of mechanical properties of in-place concrete. Information on nondestructive test methods for determining concrete material properties and assessing conditions of concrete is available [8,9]. Methods available to detect corrosion occurrence include visual observations, half-cell potential measurements, delamination detection, electrolyte chemistry, corrosion monitors, acoustic emission, radiography, ultrasonics, magnetic perturbation, metallurgical evaluations, and electrical resistance. Guidance on interpretation of results from reinforced concrete structure's inspections is available [0]. Maintaining the required prestressing force levels in post-tensioned concrete containments is important in helping assure that the containment retains adequate margins with respect to structural and leaktight integrity. Inspection requirements and acceptance standards for post-tensioning system inspections are available [ ]. A suitable technique for inspection of thick heavily-reinforced concrete structures requires development.

Whenever damage is detected, corrective actions are taken to identify and eliminate the source of the problem and thereby halt the degradation process. When significant containment metallic pressure boundary wall thinning, cracking, surface defects, or leakage is detected and the containment structural or leaktight integrity is potentially jeopardized, defective areas are either evaluated, repaired, or replaced before the plant is returned to service. The primary mechanism of concern to the containment metallic pressure boundary is corrosion. Methods to prevent the occurrence of corrosion primarily include the application (or maintenance) of coatings to exposed steel that is at risk, and use of cathodic protection systems (i.e., impressed current or sacrificial anode). Repair methods generally include: (1) defect removal by mechanical means in which the unacceptable flaw is reduced and the resultant section thickness created by the removal process remains equal to at least the minimum design thickness; (2) repair welding in which the design section thickness is reestablished (e.g., cladding); and (3) component replacement with items that meet acceptance standards [ ]. Repair options for restoring damaged bellows include replacement of penetration assembly, bellows replacement, installation of new enveloping bellows, in-place welding repairs, removal of severe dents, and blending the surface. Basic repair solutions for reinforced concrete structures include: (1) realkalization by either direct replacement of contaminated concrete with new concrete, use of a cementitious material overlay, or application of electrochemical means to accelerate diffusion of alkalis into carbonated concrete; (2) limiting the corrosion rate by changing the environment (e.g., drying) to reduce the electrolytic conductivity; (3) steel reinforcement coating (e.g., epoxy); (4) chloride extraction by passing an electric current (DC) from an anode attached to the concrete surface through the concrete to the reinforcement (chloride ions migrate to anode); and (5) cathodic protection. Detailed information on repair of NPP civil structures is available [ ].

RELIABILITY-BASED CONDITION ASSESSMENT

Time-dependent reliability analysis methods provide a framework for performing condition assessments of existing structures and for determining whether in-service inspection and maintenance are required to maintain reliability and performance at the desired level. The duration of structural loads that arise from rare operating or environmental events, such as accidental impact, earthquakes, and tornadoes, is short and such events occupy a negligible fraction of a structure's service life. Such loads can be modeled as a sequence of short-duration load pulses occurring randomly in time. The occurrence in time of such loads is described by a Poisson process, with the mean (stationary) rate of occurrence, λ, random intensity, S_j, and duration, τ. The number of events, $N(t)$, to occur during service life, t, is described by the probability mass function,

$$P\left[N(t)=n\right]=\frac{(\lambda t)^n \bullet \exp(-\lambda t)}{n!};n=0,1,2,\ldots \tag{1}$$

The intensity of each load is a random variable, described by the cumulative distribution function (CDF) $F_s(x)$. In general, the load process is intermittent and the duration of each load pulse has an exponential distribution,

$$F_{t_d} = 1 - \exp[-t/\tau];\ t \geq 0 \tag{2}$$

in which τ = average duration of the load pulse. The probability that the load process is nonzero at any arbitrary time is $p = \lambda\tau$. Loads due to normal facility operation or climatic variations may be modeled by continuous load processes. A Poisson process with rate λ may be used to model changes in load intensity if the loads are relatively constant for extended periods of time.

The strength, R, of a structural component is described by

$$R = B \bullet R_m(X_1, X_2, \ldots, X_m) \tag{3}$$

in which X_1, X_2, ... are basic random variables that describe yield strength of steel, compressive or tensile strength of concrete, and structural component dimensions or section properties. The function $R_m(\ldots)$ describes the strength based on principles of structural mechanics. Modeling assumptions invariably must be made in deriving $R_m(\ldots)$ and the factor

B describes errors introduced by modeling and scaling effects. The probability distribution of B describes bias and uncertainty that are not explained by the model $R_m(\ldots)$ when values of all variables X_i are known. The probability distribution of B can be assumed to be normal. A more accurate behavioral model leads to a decrease in the mean and variability in B and thus in R. Probabilistic models for R in most structures must be determined from the statistics of the basic variables, X_i, since it seldom is feasible to test a sufficient sample of structural components to determine the cumulative distribution function (CDF) of R directly.

The failure probability of a structural component can be evaluated as a function of (or an interval of) time if the stochastic processes defining the residual strength and the probabilistic characteristics of the loads at any time are known. The strength, R(t), of the structure and applied loads, S(t), are both random functions of time. Assuming that degradation is independent of load history, at any time t the margin of safety, M(t), is

$$M(t) = R(t) - S(t). \tag{4}$$

Making the customary assumption that R and S are statistically independent random variables, the (instantaneous) probability of failure is,

$$P_f(t) = P\left[M(t) < 0\right] = \int_0^{\infty} F_R(x) f_S(x) dx \tag{5}$$

in which $F_R(x)$ and $f_S(x)$ are the CDF of R and probability density function (PDF) of S. Equation (5) provides an instantaneous quantitative measure of structural reliability, provided that $P_f(t)$ can be estimated and/or validated [13]. It does not convey information on how future performance can be inferred from past performance.

For service life prediction and reliability assessment, one is more interested in the probability of satisfactory performance over some period of time, say (0,t), than in the snapshot of the reliability of the structure at a particular time provided by Eq. (5). The probability that a structure survives during interval of time (0,t) is defined by a reliability function, L(0,t). If, for example, n discrete loads S_1, S_2, ..., S_n occur at times t_1, t_2, ..., t_n during (0,t), the reliability function becomes,

$$L(t) = P\left[R(t_1) > S_1, \ldots, R(t_n) > S_n\right] \tag{6}$$

in which $R(t_i)$ = strength at time of loading S_i.

Taking into account the randomness in the number of loads and the times at which they occur as well as initial strength, the reliability function becomes [14]

$$L(t) = \int_0^{\infty} \exp\left(-\lambda t\left[1 - t^{-1}\int_0^t F_s(g_i r) dt\right]\right) f_{R_0}(r) dt \tag{7}$$

in which f_{R_0} = PDF of the initial strength R_0 and g_i = fraction of initial strength remaining at time of load S_i. The probability of failure during (0,t) is

$$F(t) = 1 - L(t). \tag{8}$$

The conditional probability of failure within time interval (t, t+Δt), given that the component has survived up to t, is defined by the hazard function which can be expressed as

$$h(t) = -d \ln L(t)/dt. \tag{9}$$

The reliability and hazard functions are integrally related

$$L(t) = \exp\left[-\int_0^t h(x) dx\right]. \tag{10}$$

The hazard function is especially useful in analyzing structural failures due to aging or deterioration. For example, if the structure has survived during the interval (0, t_1), it may be of interest in scheduling in-service inspections to determine the probability that it will fail before t_2. Such an assessment can be performed if h(t) is known. If the time-to-failure is T_f, this probability can be expressed as

$$P\left[T_f < t_2 \middle| T_f > t_1\right] = 1 - \exp\left(-\int_{t_1}^{t_2} h(x) dx\right). \tag{11}$$

In turn, the structural reliability for a succession of inspection periods is

$$L(0,t) = \prod_i L(t_{i-1}, t_i) \exp\left\{-\int_{t_i}^{t} h(x)dx\right\} \tag{12}$$

in which $t_{i-1} = 0$ when $i = 1$.

Forecasts of reliability enable the analyst to determine the time period beyond which the desired reliability of the structure cannot be assured. At such a time, the structure should be inspected. The density function of strength, based on prior knowledge of the materials in the structure, construction, and standard methods of analysis, is indicated by $f_R(r)$. The information gained during scheduled inspection, maintenance and repair causes the characteristics of strength to change; this is denoted by the (conditional) density $f_R(r|B)$, in which B is an event dependent on in-service inspection. Information gained from the inspection usually involves several structural variables including dimensions, defects, and perhaps an indirect measure of strength or stiffness. If these variables can be related through event B, then the updated density of R following in-service inspection is,

$$f_R(r|B) = P[r < R \le r + dr, B] / P[B] = cK(r)f_R(r) \tag{13}$$

in which $f_R(r)$ is termed the prior density of strength, K(r) is denoted the likelihood function, and c is a normalizing constant. The time-dependent reliability analysis then is re-initialized following in-service inspection/repair using the updated $f_R(r|B)$ in place of $f_R(r)$. Applications of the time-dependent reliability methodology to a ring-stiffened shell and concrete components are available [15-17].

Optimal intervals of inspection and repair for maintaining a desired level of reliability can be determined based on minimum life cycle expected cost considerations. Preliminary investigations of such policies have found that they are sensitive to relative costs of inspection, maintenance, and failure [16]. If the cost of failure is an order (or more) of magnitude larger than inspection and maintenance costs, the optimal policy is to inspect at nearly uniform intervals of time. However, additional research is required before such policies can be finalized as part of an aging management plan.

FRAGILITY ASSESSMENTS

A probabilistic safety assessment (PSA) is a structured framework for evaluating uncertainty, performance, and reliability of an engineered facility. The move toward quantitative risk assessment has accelerated in recent years as the benefits have become increasingly apparent in many fields [18]. The recently issued Regulatory Guide 1.174 [19] defines the USNRC's position on risk-informed decision-making regarding proposed changes to the licensing bases of operating NPPs.

The PSA process is initiated with the identification of limit states (LS) or conditions in which the system ceases to perform its intended function(s) in some way. For structural components and systems in NPPs, such limit states may be either strength or deformation-related, as large (inelastic) deformations affect the integrity or operability of mechanical or electrical systems that are attached to or otherwise interface with the structure. The limit state probability then is expressed as,

$$P[LS] = \Sigma\, P[LS|D = x]\, P[D = x] \tag{14}$$

in which D describes the intensity of demand on the system (hazard), and $P[LS|D = x]$ is the conditional limit state probability, or the fragility, of the system.

The fragility displays, in probabilistic terms, the capability of an engineered system to withstand a specified event with intensity x (sometimes referred to as a review-level event), one that often is well in excess of the design-basis event. Thus, it defines safety margins probabilistically against specific identified events for decision and regulatory purposes in a manner that effectively uncouples the system analysis from the hazard analysis. The fragility modeling process leads to a median-centered estimate of system performance, coupled with an estimate of the uncertainty in performance. The fragility of a structural component or system often is modeled by a lognormal CDF, described by,

$$F_R(x) = \Phi\,[\ln(x/m_C)/\beta_C] \tag{15}$$

in which $\Phi[\ ]$ = standard normal probability integral, m_C = median capacity (expressed in units that are consistent with the demand, x, in Eq. (14), and β_C = logarithmic standard deviation, which is approximately equal to the coefficient of variation (COV) in capacity, V_C, when $V_C < 0.3$ and provides a measure of uncertainty in capacity.

The strengths of steel and concrete structural materials and components are random variables, and their median (or mean) strengths are well in excess of the nominal values specified for NPP design [20]. If these median strengths are used in a finite element-based structural analysis with nonlinear analysis capabilities in lieu of specified nominal strengths, one often can obtain a reasonable estimate of the median system capacity, m_C, in Eq. (15) [21]. The uncertainty in capacity displayed by Eq. (15) arises from numerous sources. Some of these uncertainties (denoted by COV β_R) are inherent (aleatory) in nature, and are essentially irreducible under current engineering analysis procedures. Other uncertainties (denoted by COV β_U) arise from assumptions made in the analysis of the system and from limitations in the supporting databases. Such knowledge-based (epistemic) uncertainties depend on the quality of the analysis and data, and generally can be reduced, at the expense of more comprehensive (and costly) analyses. A nonlinear finite element-based fragility assessment of a pressurized-water reactor ice condenser steel containment having postulated losses of shell thickness due to corrosion at key locations of potential corrosion has been performed [[illegible]]. Results for a reinforced concrete flexural member and a shear wall experiencing loss of steel cross-sectional area, concrete spalling, or a combination of the two, are also available [[illegible]]. For the steel containment and the reinforced concrete structures analyzed to date, the fragilities indicate that even in the degraded conditions addressed, these structures maintain sufficient structural integrity to withstand challenges from events at or beyond the original prescriptive design basis with a high level of confidence.

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