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Assessment of Aging of Nuclear Power Plant Civil Structures

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ABSTRACT

Research is being conducted by ORNL for the USNRC to address aging of civil structures in light-water reactor plants. The importance and operating experience of nuclear power plant (NPP) civil structures is reviewed. Factors that can lead to age-related degradation of reinforced concrete structures and containment metallic pressure boundaries (i.e., steel containments and liners of reinforced concrete containments) are identified and their manifestations described. Background information and data for improving and developing methods to assess the effects of age-related degradation on structural performance are provided. Techniques for detection of degradation are reviewed and research related to development of methods for inspection of inaccessible regions of the containment pressure boundary presented. Application of structural reliability analysis methods to develop condition assessment tools and guidelines is described.

INTRODUCTION

All commercial NPPs contain civil structures whose performance and function are necessary to protect the safety of plant operating personnel and the general public. Although these structures are essentially passive under normal operating conditions, they play a key role in mitigating the impact of extreme environmental events such as tornadoes, hurricanes, and earthquakes. As such, these structures are designed to withstand loadings from a number of low-probability external and internal events. Loadings incurred during normal plant operation therefore generally are not significant enough to cause appreciable degradation. Moreover, the importance of these structures in accident mitigation is amplified by the so-called "common cause" effect, in which failure of a structure may lead to failure or loss of function of appurtenant mechanical or electrical components and systems. Furthermore, in contrast to many mechanical and electrical components, replacement of many structures is impractical.

DEGRADATION AND OPERATING EXPERIENCE

As NPPs age, degradation incidences are starting to occur at an increasing rate, primarily due to environmental-related factors. There have been at least 66 separate occurrences of degradation in operating containments (some plants may have more than one occurrence of degradation) [1].

Degradation Factors

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 leak-tight 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 the containment metallic pressure boundary 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 leak-tight 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 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.

Operating Experience

There have been over 30 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).

IN-SERVICE INSPECTION

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.

Degradation Detection

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 margins (e.g., provide a baseline for evaluating the safety significance of any damage that may be present and defining in-service inspection programs and maintenance strategies).

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. Guidelines on conduct of a structural condition assessment are available [4-6]. Summarized below are several nondestructive examination techniques for use in assessment of the significance of metallic and reinforced concrete material degradation.

Metallic Materials

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 radiographic. 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 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 [7].

Reinforced Concrete Materials

Primary manifestations of distress that are present or can occur in reinforced concrete structures include cracking, voids, 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 [4]. 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 use of 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 of importance relative to assessment of reinforced concrete structures for corrosion occurrence include: half-cell potential (likelihood of corrosion activity), linear polarization (instantaneous corrosion current and corrosion rate), resistivity (likelihood of corrosion activity), and galvanostatic pulse (corrosion rate). Guidance on interpretation of results from reinforced concrete structure's inspections is available [10,11].

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 leak-tight integrity. Trends established by examinations performed at the specified intervals can provide indications that the following characteristics are acceptable at least until the time of the next scheduled inspection: lift-off force, wire/strand strength and ductility, sheathing filler chemical properties, and corrosion of metallic components. Determination of the level of prestresing force is performed primarily through lift-off force measurements. Lift-off force results are compared to design calculations of prestressing force versus time and if determined to be unacceptable, specific actions are required (e.g., increased inspection, retensioning, or replacement). Representative samples of the tendon materials are removed to monitor for any aging effects, notably corrosion. Finally, samples of the grease are taken at both ends of the tendons selected for examination and analyzed for free water content, reserve alkalinity, and presence of aggressive ions (i.e., chloride, sulfide, and nitrate ions). Post-tensioning system inspection acceptance standards are available [12].

Needed Improvements

Inspection of NPP structures can be difficult because there are a number of functionally different components in a variety of environments. Previously it was noted that there are many techniques, destructive, nondestructive and semi-destructive, that are available for indicating the condition of the basic components that comprise NPP structures. Application of these techniques is most effective when an approach is utilized in which the structures have been prioritized with respect to such things as aging significance, structural importance, environmental factors, and risk. Guidance on component selection is provided elsewhere [4,5].

Once the components have been selected for inspection, however, there are several conditions in NPPs where performing the inspections may not be straightforward.

Examples of applications where the capabilities of inspection methods require improvements or development include: inaccessible areas of containment metallic pressure boundaries and thick heavily-reinforced concrete sections.

Inaccessible Area Considerations. Inspection of inaccessible portions of NPP containment metallic pressure boundary components (e.g., fully embedded or inaccessible containment shell or liner portions, the sand pocket region in Mark I and II drywells, and portions of the shell obscured by obstacles such as platforms or floors) requires special attention. Embedded metal portions of the containment pressure boundary may be subjected to corrosion resulting from groundwater permeation through the concrete; a breakdown of the sealant at the concrete-containment shell interface that permits entry of corrosive fluids from spills, leakage, or condensation; or in areas adjacent to floors where the gap contains a filler material that can retain fluids. Corrosion occurrence in inaccessible areas may challenge the containment structural integrity and, if through-wall, can provide a leak path to the outside environment. No completely suitable technique for inspection of inaccessible portions of containment metallic pressure boundaries has been demonstrated to date.

Exploratory analytical and experimental simulations have been conducted to investigate the feasibility of high frequency acoustic imaging techniques for detecting and locating thickness reductions in the metallic pressure boundaries of NPP containments [13,14]. The analytical study used an elastic layered media code (OASES) to perform a series of numerical simulations to determine the fundamental two-dimensional propagation physics. Analytical simulation suggests that for the case of steel-lined concrete containments, the thin steel liner with concrete backing contribute to give unacceptable loss of signal. For embedded steel containments, analytical simulation suggests that significant degradations (i.e., $\geq$2 mm) of containment thickness below the concrete/air interface provide reasonable backscatter signal levels of approximately –15 dB, and should be detectable. The experimental study utilized a commercial ultrasonic testing system to carry out several tests of steel plates 25 by 203 by 914 mm, some partially embedded in concrete. Scattered signals from simulated degradations of different size and shape (i.e., rectangular, semi-circle, and "V" shaped), as well as from a flaw embedded in concrete, were investigated. Overall, the experimental results showed that the measurement system displayed a dynamic range of 125 dB with measurement variability less than 1-2 dB. Based on these results, a 4-mm-deep round-faced degradation embedded in 30 cm of concrete has expected returns of –73 dB relative to input and should be detectable.

Magnetostrictive sensors launch guided waves and detect elastic waves in ferromagnetic materials electromagnetically to determine the location and severity of a defect based on timing and signal amplitude. The feasibility of applying this technology to inspection of plate-type materials and evaluating its potential for detecting and locating thickness reductions in the containment metallic pressure boundary has been investigated [15]. Limited analytical studies suggest that a low-frequency A_0 mode wave (below approximately 0.5 MHz-mm, that corresponds to approximately 40 kHz in a 12.7-mm-thick plate or 20 kHz in a 25.4-mm-thick plate) would be best suited for inspection of containment metallic pressure boundaries that are either backed on one or both sides by concrete. Pulse-echo sensor data were obtained experimentally for notches ranging from 10- to 30-cm long that had been placed into a 6.11–m-long by 1.23–m-wide by 6.35-mm-thick plate at a distance equal to 4.06 m from the probe end of the plate. Results indicate that guided waves provide an effective means of inspection of the metallic pressure boundary in a NPP and are capable of performing global, long-range inspection of plates, including areas that are difficult to access because of the presence of other equipment or attachments, or the presence of concrete on one or both sides. The effect of a flaw embedded in concrete was not investigated experimentally.

Experimental studies utilizing the multimode guided wave technique were conducted to demonstrate its feasibility for identification and location of thickness reductions in the metallic pressure boundary of NPP containments [16]. Test specimens included a bare plate with two defects, a plate with concrete but no defects, and a plate embedded in concrete with one defect. Each plate was 25 by 203 by 914 mm. The specimens provided a benchmark for studying several aspects of guided wave inspection - sensitivity, transmission ability across defects, inspection reliability, and penetration ability. The plates were interrogated using both horizontal shear [electromagnetic acoustic transducer (EMAT)] and Lamb (piezoelectric transducer) guided waves. Horizontal shear (SH) guided waves have particle displacements in the shear horizontal direction, which is perpendicular to the propagation direction. The grid distance of the EMATs was 17.7 mm, which determines that the corresponding frequency for generating the non-dispersive SH wave mode is around 200-250 kHz. Reflected echoes for the bare plate with two defects and the plate containing a defect embedded in concrete, indicates that all defects can be detected by using the SH waves. Unlike SH waves, Lamb waves have particle displacements that are both parallel and perpendicular to the propagation direction. The frequency and wedge angle determines the generated Lamb wave mode. In order to obtain a fairly uniform energy distribution across the plate thickness for Lamb waves, a 38° wedge angle was utilized. The tone burst frequency was 565 kHz. Lamb waves were also able to detect the defects for both scenarios. However, for the plate embedded in concrete but without defects, multiple echoes were received from the plate-concrete interface before the backwall echo (BWE). This indicates one of the disadvantages of the Lamb wave mode in that it is sensitive to the plate-concrete interface.

Thick Heavily-Reinforced Concrete Sections. Inspection of NPP reinforced concrete structures presents challenges different from conventional civil engineering structures in that wall thicknesses can be in excess of one meter; the structures often have increased steel reinforcement density with more complex detailing; there can be a number of penetrations or cast-in-place items present; and accessibility may be limited due to the presence of liners and other components, harsh environments, or the structures may be located below ground. Techniques are required for characterization, inspection, and monitoring of thick heavily-reinforced concrete structures to provide assurances of their continued integrity. Methods that can be used to inspect the basemat without the requirement for removal of material and techniques that can detect and assess corrosion are of particular interest. The present status of work in this area is available in proceedings of a workshop held specifically to develop nondestructive evaluation priorities for concrete structures in NPPs [17]. Radar, acoustic, and radiography methods were identified as having the greatest potential to meet needs related to inspection of these structures. Application and qualification of these techniques to NPP structures of interest, however, requires demonstration.

RELIABILITY-BASED CONDITION ASSESSMENTS

Time-Dependent Reliability

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,

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

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

$$F_{T_d} = 1 - \exp[-t/\tau];\ t \geq 0 \qquad (2)$$

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

$$R = B \bullet R_m(X_1, X_2, \ldots, X_m) \qquad (3)$$

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 usually 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

$$M(t) = R(t) - S(t). \qquad (4)$$

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

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

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 [18]. 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). Indeed, it is difficult to use reliability analysis for engineering decision analysis without having some time period in mind (e.g., an in-service maintenance interval). 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,

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

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 [19]

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

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

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

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

Intervals of inspection and maintenance that may be required as a condition for continued operation can be determined from the time-dependent reliability analysis. 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,

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

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 $f_R(r)$. The updating causes the hazard function to be discontinuous.

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. Applications of the time-dependent reliability methodology to a ring-stiffened shell and concrete components are available [20-22].

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 [23]. The recently issued Regulatory Guide 1.174 [24] 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. With the limit states identified, the limit state probability is expressed as,

$$P[LS] = \sum P[LS|D = x]\, P[D = x] \qquad (14)$$

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,

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

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 [25]. If these median strengths are used in structural analysis in lieu of specified nominal strengths, one often can obtain a reasonable estimate of the median capacity, m_C, in Eq. (15) [26]. 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. The role of epistemic uncertainty on fragility can be displayed in one of two ways. In the first, a family of fragilities is generated, one for each modeling assumption. In the second, the aleatory and epistemic uncertainties are combined in the form $\beta_C^2 = \beta_R^2 + \beta_U^2$, and only one (mean) fragility curve is generated. The second approach is taken herein.

The fragility assessment is illustrated using a PWR ice condenser steel containment which has been modeled to be somewhat similar to one of the reference plants in the NUREG-1150 risk study [27] and has been thoroughly analyzed in several independent studies.* The steel containment is designed for an internal pressure of 74 kPa, has an internal diameter of 35 m, springline height of 35 m, and the elevation of the apex of the spherical dome is 53 m. The steel in the shell is A516/GR 60 plate, varying in thickness from 35 mm at the basemat to 12 mm at the springline. Vertical and circumferential stringers are welded to the exterior of the shell at approximately 1.2 m vertical and 3 m horizontal intervals.

The containment must confine radioactive material in the event of an accident, so the performance limit is loss of shell integrity or ability to perform this essential function [29]. This performance limit must be related to structural limit states that can be identified from nonlinear finite-element analysis, along with local or general structure or material failure criteria. Tests of internally pressurized scaled model containments have indicated that the governing failure mode invariably is one of tensile instability. For this study, the tensile instability limit state is defined by,

$$\varepsilon_p = \varepsilon_f\, f_1\, f_2\, f_4 \qquad (16)$$

in which ε_p = effective plastic strain, ε_f = uniaxial limit strain, f_1 = factor to correct uniaxial limit strain for triaxiality effects, f_2 = factor that accounts for bias and uncertainty in the finite element analysis, and f_4 = factor to account for the reduction in steel ductility as a result of corrosion. The commercially available nonlinear finite-element program ABAQUS was used to perform the numerical experiments of the pressurized containment leading to the fragilities. This analysis is described in more detail elsewhere [26]. Four cases are illustrated: the uncorroded (as-built) case, which is used as a benchmark, cases where there is postulated 10% and 25% loss of containment

* 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 [28].

shell thickness behind the ice basket, and the case where there is 50% loss adjacent to an upper floor.

The median capacity of the containment in the as-built condition is 455 kPa, and the logarithmic standard deviation β_X is 0.04. The median is consistent with values obtained elsewhere by other investigators. The estimated 5-percent and 2-percent exclusion limits are 427 kPa and 421 kPa, respectively. For comparison, median fragilities based on simplified criteria such as first yielding or 2% strain in the circumferential direction, which can be modeled by simple yield analysis of the shell, are 290 kPa and 365 kPa, respectively. These simplified analyses lead to conservative estimates of the margin of safety. In comparison, for a postulated 25% loss of shell thickness behind the ice basket the median capacity is 386 kPa, a decrease of 15%, and the logarithmic standard deviation has increased to 0.06. The 5-percent and 2-percent exclusion limits for this postulated degraded condition have decreased to 352 kPa and 345 kPa, respectively, or by approximately 18% from the as-built condition.

SUMMARY AND DISCUSSION

Activities that address aging of civil structures in light-water reactor plants are summarized. Current and emerging nondestructive examination techniques and a degradation assessment methodology for characterizing and quantifying the amount of damage present are noted. The use of time-dependent structural reliability analysis methods to provide a framework for addressing the uncertainties attendant to aging in the decision process are discussed (i.e., methods help provide assurances that degraded civil structures will be able to withstand future extreme loads during the desired service period with a level of reliability that is sufficient for public safety). The impact of aging (i.e., loss of shell thickness due to corrosion) on steel containment fragility for a pressurized water reactor ice-condenser plant is presented. Results indicate that values at the 2- and 5-percentiles exclusion limits for this postulated degraded condition are well in excess of the design basis of 74 kPa (i.e., by factors of 5.7 in the as-built condition and 4.7 under the postulated degradation). These large margins of safety are due to a number of factors. Design material strengths are substantially less than the likely values in service; design is based on the assumption of elastic behavior, which does not account for additional capacity beyond yielding that is provided by the large ductility of carbon steels; conservative assumptions are made regarding structural response; and factors of safety in the range 1.5 to 2.0 are used, depending on the safety check. Thus, even in a deteriorated condition, the containment still may retain sufficient capacity to withstand challenges from events at or beyond the original prescriptive design basis with a high level of confidence. Other studies have shown that a decrease in the median fragility of 15% is likely to lead to an increase in the limit state probability of a factor of two or less [30]. Such increases would not mandate immediate corrective action according to the recent regulatory guideline RG 1.174 on risk-informed decision making, but would require that a periodic inspection program be initiated to track cumulative impacts of such degradation over time [24].

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