Managing Concrete Structures Aging - One Approach

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Synopsis

Research providing guidance on management of aging reinforced concrete structures is summarized. Topics covered include a materials property database, an aging assessment methodology to identify critical structures and degradation factors that can potentially impact performance, guidelines and evaluation criteria for use in condition assessments, and a reliability-based methodology for current condition assessments and estimations of future performance. Applicability of nondestructive evaluation and repair-related technologies is addressed.

1. Introduction

1. Nuclear power plants (NPPs) are designed, built, and operated to standards that aim to reduce the releases of radioactive materials to levels as low as reasonably achievable.[1] The safety-related reinforced concrete structures in these plants are designed to withstand loadings from a number of low-probability external and internal events, such as earthquakes, tornadoes, and loss-of-coolant accidents. Loadings incurred during normal plant operation are generally not significant enough to cause appreciable degradation. Nuclear power plants, however, involve complex engineering structures and components operating in demanding environments that potentially can challenge the high level of safety (i.e., safety margins) required throughout the operating life of the plant. The effects of these processes may accumulate within these structures over time to cause failure under design conditions, or lead to repair. Ensuring that the structural capacity of the reinforced concrete structures has not deteriorated unacceptably due to aging or environmental effects is essential to reliable continued service and informed aging management decisions. Although major mechanical and electrical equipment items in a plant could be replaced, if necessary, replacement of most of the safety-related concrete structural components would be economically unfeasible.

2. Safety-Related Concrete Structures

2. All commercial NPPs contain concrete structures whose performance and function are necessary for protection of the safety of plant operating personnel and the general public, and the environment. Typical safety-related functions that the concrete structures provide include foundation, support, biological shielding, containment, and protection against internal and external hazards. Each boiling-water reactor (BWR) or pressurized-water reactor (PWR) unit in the U.S. is housed within a much larger metal or concrete containment.

3. Concrete containments for PWRs are fabricated from reinforced concrete, that in some cases may be post-tensioned. Containments enclose the primary circuit that includes the reactor pressure vessel, steam generators, etc. Three general categories of PWR containments exist: large dry, ice condenser, and subatmospheric. The large dry containment is designed to have a capacity to contain the energy of the entire volume of primary coolant fluid in the unlikely event of a loss-of-coolant-accident (LOCA). The ice condenser containments channel the steam resulting from a LOCA through ice beds to reduce the pressure buildup and thus the containment volume and pressure requirements. The subatmospheric containments are designed so that a slightly negative pressure is maintained in the containment to reduce the volume requirements. Leak-tightness of the containments is provided by a thin steel liner (e.g., 6 mm in thickness) that is anchored to the concrete. Material systems used to fabricate concrete containments include: moderate heat of hydration and sulfate-resistant portland cement, fine and coarse aggregate and water obtained primarily from local sources, carbon steel deformed bar reinforcement having a minimum yield strength of 415 MPa, and wire or strand post-tensioning systems having capacities to 10.7 MN. Depending on the functional design, concrete containments can be on the order of 40 to 50 m diameter and 60 to 70 m high, with dome and wall thicknesses from 0.9 to 1.4 m, and base slabs from 2.7 to 4.1 m. Figure 1 presents a cross section of a post-tensioned reinforced concrete containment.

4. Although the majority of BWR plants utilize a steel containment vessel, a number of units utilize either a prestressed- or reinforced-concrete containment. With only one exception, all BWR plants in the U.S. that utilize a

[1] Although the present application addresses reinforced concrete structures in NPPs, results generally also are applicable to general civil engineering reinforced concrete structures. Relative to general civil engineering structures the NPP concrete structures tend to be more massive and have increased steel reinforcement densities with more complex detailing.

steel containment have reinforced concrete structures that serve as secondary containments or reactor buildings to provide support and shielding functions for the primary containment. The containments are divided into two main compartments — wetwell and drywell. After a LOCA, the air and steam in the drywell are forced through a number of downcombers to a pool in the wetwell, where the steam condenses. Water spray systems are provided and the auxiliary systems are generally housed in the secondary containment.

5. A myriad of other concrete-based structures are also contained as part of a light-water reactor (LWR) plant (e.g., reactor pedestal/support, intake structures, and primary shield wall). More detailed descriptions of the safety-related concrete structures are provided elsewhere (1).

3. Aging and Durability Considerations

6. Reinforced concrete structures historically have been designed in accordance with national consensus codes and standards in force at the time (2). The rules in these documents were developed over the years by experienced people and are based on the knowledge that was acquired in testing laboratories and supplemented by field experience. Design principles have been dominated by analytical determinations based on strength principles. Durability considerations require that the time element be factored into the design of reinforced concrete structures. Associated with the design specifications developed for concrete structures in conformance with these calculations was a certain implied level of durability (e.g., minimum concrete cover requirements to protect embedded steel reinforcement under different anticipated environmental conditions).

7. Primary mechanisms or factors that can produce premature deterioration of concrete structures include those that impact either the concrete or reinforcing steel 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, sulfate attack, attack by acids and bases, salt crystallization, and alkali-aggregate reactions. Physical attack mechanisms for concrete include freeze/thaw cycling, thermal exposure/thermal cycling, abrasion/erosion/cavitation, irradiation, and fatigue or vibration. Degradation of mild steel reinforcing materials can occur as a result of corrosion, irradiation, elevated temperature, or fatigue effects. Post-tensioning systems are susceptible to the same degradation mechanisms as mild steel reinforcement, plus loss of prestressing force, primarily due to tendon relaxation and concrete creep and shrinkage.

8. Although the vast majority of reinforced concrete structures associated with NPPs have met and continue to meet their functional and performance requirements, in several instances these structures have exhibited degradation. Examples of some of the degradation occurrences include cracking in basemats, failure of prestressing tendon wires, corrosion of steel reinforcement in water-intake structures, leaching of tendon gallery concrete, and freeze/thaw damage to containment dome concrete (1). Current aging concerns are related to inaccessibility of reinforced concrete basemats for inspection to detect potential degradation resulting from mechanisms such as leaching or sulfate attack, availability of proven nondestructive evaluation techniques for inspection of thick heavily-reinforced concrete sections, and corrosion of embedded portions of the steel pressure boundary (liner) due to a breakdown of the seal at the floor-to-liner interface.

4. Structural Aging Program

9. Incidences of structural degradation related to the concrete components in NPPs indicate a need for improved surveillance, inspection/testing, and maintenance to enhance the technical bases for assuring continued safe operation. The Structural Aging (SAG) Program was initiated in 1988 and had the overall objectives of providing background data and information for identification and evaluation of the potential structural degradation processes; identifying issues to be addressed during continued service reviews of NPP concrete structures, as well as criteria, and their bases, for resolution of these issues; assessing relevant in-service inspection, structural assessment or remedial measures programs; and formulating methodologies to perform current assessments and reliability-based life predictions of reinforced concrete structures. To meet these objectives, activities were conducted under three task areas: (1) materials property database, (2) structural component assessment/repair technologies, and (3) quantitative methodology for continued service determinations.

4.1 Material Properties Database

10. Development of a reference source that contains data and information on the time variation of material properties under the influence of pertinent environmental stressors and aging factors provides a means to assist in the prediction of potential long-term deterioration of critical concrete structural components and to establish limits on hostile environmental exposure for these structures. Also, by using a comparative approach, estimations of current and as well as future material properties can be made. Primary activities under this task included development of the Structural Materials Information Center (SMIC), assemblage of materials property data, and review and evaluation of service life models. In addition, durability assessments of concrete structures contained in several nuclear power stations located in the United Kingdom (UK) were conducted, and the performance of post-tensioning systems in both UK and U.S. nuclear power facilities was assessed.

11. The SMIC consists of the *Structural Materials Handbook* and the *Structural Materials Electronic Database* (3). The *Structural Materials Handbook* is an expandable, hard-copy reference document containing complete sets of data and information for each material (e.g., material composition, constituent material properties, and performance and analysis information useful for structural assessments and safety margins determinations). The *Structural Materials Electronic Database* is an electronically accessible version of the *Structural Materials Handbook* providing an efficient means for searching the various database files to locate materials with similar characteristics or properties. Reference sources and testing of prototypical concrete samples obtained from nuclear power plant facilities have been used to develop over 140 material databases for the SMIC. Summary descriptions of the material property database files contained in SMIC are provided elsewhere (4).

12. A review and evaluation was conducted of accelerated aging techniques and tests that can either provide data for service life models or that by themselves can be used to predict the service life or performance of reinforced concrete (5). The most promising approach for predicting the remaining service life of concrete involves the application of mathematical models of the degradation processes. Models were identified and evaluated for each of the degradation processes that can potentially impact the performance of concrete structures. A major conclusion of this study was that theoretical models need to be developed, rather than relying solely on empirical models, because predictions from theoretical models are more reliable, far less data are needed, and the theoretical models would have wider applications.

13. Durability assessments of concrete structures at nuclear power stations in the UK, and performance evaluations of post-tensioning systems in prestressed concrete pressure vessels (PCPVs) in the UK and post-tensioned containments in the U.S. were conducted (6-8). Results indicate that the performance of the concrete structures and PCPVs at UK nuclear power stations has been good with only minor incidences of concrete cracking and tendon corrosion (insignificant pits) reported. Primary degradation mechanisms considered under the U.S. aging study of post-tensioning systems were corrosion, loss of prestressing force, and (potential) loss of strength and ductility of the post-tensioning elements. Results indicate that deterioration of the system hardware has not been significant. Water has occasionally been found in the anchorage end caps but has been of no consequence or produced only minor surface staining of load-bearing elements. Leakage of corrosion-inhibiting grease has occurred at the tendon end caps due to overfilling or presence of defective gaskets. Grease has also been observed on exterior concrete surfaces, but it is uncertain if this is an aging-related problem.[2] Tendon end-anchorage forces generally were above the required limits, but some of the older plants have experienced tendon forces below these limits.

4.2 Structural Component Assessment/Repair Technologies

14. New structures can be designed for improved durability based on operating experience (e.g., use of high performance concrete materials). Existing structures, however, have already been designed and constructed, so apart from possibly the addition of barrier materials and sealants to accessible surfaces to prevent ingress of hostile

[2] Results of a limited investigation at the Trojan NPP indicate that the grease on the external containment surface was due to leakage from the conduits that migrated through cracks in the concrete that probably formed due to shrinkage and confinement (9). Concrete cores indicated that the grease was confined to the cracks with no perceptible movement into the concrete to affect its properties.

environments, the most prudent approach to maintaining adequate structural performance is through an aging management program that involves application of in-service inspection and maintenance strategies. Primary activities under this task included development of a structural aging assessment methodology, review and evaluation of in-service inspection and structural integrity assessment methods, development of nondestructive evaluation methods for inspection of inaccessible portions of metallic liners that provide leak-tight barriers for reinforced concrete structures, formulation of in-service inspection guidelines, establishment of criteria for use in condition assessments, and reviews of repair practices for degraded reinforced concrete structures.

15. Structural Aging Assessment Methodology. A methodology has been developed that provides a logical basis for identifying the critical concrete structural elements and the degradation factors that potentially can impact the performance of these structures (10). A numerical rank is computed for each subelement by summing the weighted contributions of the subelement's structural importance, safety significance environmental exposure, and potential degradation factor significance. Using the sum of ranks for the individual subelements, the final rank of each structure is provided. Selection of structural components for evaluation can also be based on an evaluation of the impact on plant risk due to structural aging. This approach has been applied to NPP concrete structures through the use of plant logic models to identify structural components of most importance. The impact of aging on the fragility parameters is evaluated through changes in the cumulative distribution function of the estimated probability of core damage; the high-confidence, low-probability-of-failure value of demand; and a point estimate of risk. (11). The listing of critical elements and subelements generated using either of these two approaches (or both approaches in concert where applicable) can be utilized as part of an aging management program to prioritize in-service inspections so that they can focus on a selected subset of structural components that have the potential to impact performance and safety.

16. In-service Inspection Methods. Nondestructive test methods are used to determine hardened concrete properties and to evaluate the condition of concrete in structures.[3] Application of these methods for detection of degradation in reinforced concrete structures involves either a direct or indirect approach. The direct approach generally involves a visual inspection of the structure, removal/testing/analysis of material, or a combination of the above. Indirect approaches measure some property of concrete (e.g., rebound number or ultrasonic pulse velocity) and relate it to strength, elastic behavior, or extent of degradation through correlations that have been established previously. Many of the nondestructive test methods are based on the indirect approach, in which a small number of destructive and nondestructive tests are conducted in tandem at noncritical locations in a structure to develop the required correlation curve(s). However, destructive tests may not be possible in many areas of a structure to develop the required curves so assessment of in-place strength must be based on published relations. Regression analyses applied to data obtained from publications have been used to develop correlation curves and other statistical data for selected nondestructive testing techniques (i.e., break-off, pullout, rebound hammer, ultrasonic pulse velocity, and probe penetration) (12). Environment-specific methods are used where surfaces of structures are not accessible for direct inspection due to the presence of soils, protective coatings, or portions of adjacent structures. These methods provide an indirect assessment of the physical condition of the structure (i.e., potential for degradation) by qualifying the aggressiveness of the environment adjacent to the structure (e.g., air, soil, and groundwater). If results of these tests indicate that the environment adjacent to the structure is not aggressive, one might conclude that the structure is not deteriorating. However, when conditions indicate that the environment is potentially conducive to degradation, additional assessments are required that may include exposure of the structure for visual or limited destructive testing.

17. Inspection of Embedded Metallic Liners. Preliminary assessments of candidate techniques (i.e., conventional ultrasonics, magnetostrictive sensors, electromagnetic acoustic transducers, and multimode guided waves) for use in inspection of inaccessible portions of metallic liners that provide leak-tight barriers for reinforced concrete structures have been completed with encouraging results (13). Of the techniques investigated, the guided wave technique (multi-mode guided plate waves) appears to be the most promising. A limited investigation has been conducted in which both horizontal shear and Lamb guided waves have been used to interrogate 25-mm thick by 203-mm wide by 914-mm long plate specimens (14). Three plates were investigated: a bare plate with two defects

[3] Descriptions and principles of operation, as well as applications, for nondestructive test methods most commonly used to determine material properties of hardened concrete in existing construction and to determine structural properties and assess conditions of concrete are available (15).

(i.e., machined notches), a plate with concrete but no defects, and a plate embedded in concrete with one defect. Results indicate that horizontal shear waves can be used to detect defects in plates embedded in concrete. Investigations are presently underway to try to develop criteria for sizing the defects.

18. In-Service Inspection Guidelines. Determining the existing performance characteristics and extent and causes of any observed distress is accomplished through a condition assessment. Common in the condition assessment approaches is the conduct of a field survey, involving visual examination and application of nondestructive and destructive testing techniques, followed by laboratory and office studies. Guidelines and direction on conduct of surveys of existing general civil engineering buildings such as summarized below are available (16,17). The condition survey usually begins with a review of the "as-built" drawings and other information pertaining to the original design and construction so that information, such as accessibility and the position and orientation of embedded steel reinforcing and plates in the concrete, is known prior to the site visit. Next is a detailed visual examination of the structure to document easily obtained information on instances that can result from or lead to structural distress. Visual inspections are one of the most valuable of the condition survey methods because many of the manifestations of concrete deterioration appear as visible indications or discontinuities on exposed concrete surfaces. Visual inspections encompass a variety of techniques (e.g., direct and indirect inspection of exposed surfaces, crack and discontinuity mapping, physical dimensioning, environmental surveying, and protective coatings review). To be most effective, the visual inspection should include all exposed surfaces of the structure; joints and joint materials; interfacing structures and materials (e.g., abutting soil); embedments; and attached components (e.g., base plates and anchor bolts). Degraded areas of significance are measured. The condition of the surrounding structures should also be examined to detect occurrence of differential settlement or note aggressiveness of the local operating environment. Results obtained should be documented and photographs or video images taken of any discontinuities and pertinent findings. A crack survey is usually done by drawing the locations and widths of cracks on copies of project plans. Cracking patterns may appear that suggest weaknesses in the original design, construction deficiencies, unanticipated thermal movements, chemical reactivity, detrimental environmental exposure, restrained drying shrinkage, or overloading. Distress associated with cracks such as efflorescence, rust stains, or spalling is noted. After the visual survey has been completed, the need for additional surveys such as delamination plane, corrosion, or pachometer is determined. The delamination plane survey is used to identify internal delaminations that are usually caused by corrosion of embedded metals or internal vapor pressure. Results of the visual and delamination surveys are used to select portions of the structure that will be studied in greater detail. To locate areas of corrosion activity within reinforced concrete, copper-copper sulfate half-cell studies can be performed. By taking readings at multiple locations on the concrete surface, an evaluation of the probability of corrosion activity of embedded reinforcing steel (or other metals) can be made. Where significant chloride penetration is suspected, concrete powder samples or cores should be removed from several depths extending to and beyond the embedded outer layer of reinforcing steel. Also, a pachometer survey may be performed as part of the detailed study to confirm the location of steel reinforcement. Where there is evidence of severe corrosion, the steel bar should be uncovered to allow visual inspection and measurement of cross-sectional area loss. Upon return to the office, results of the field survey are evaluated in detail. A crack survey map is prepared and studied for meaningful patterns. Half-cell data are studied and isopotential lines are drawn to assist in determining active corrosion sites. Samples of concrete and steel obtained from areas exhibiting distress are tested in the laboratory. Chloride ion results are plotted versus depth to determine the profile and the chloride content at the level of the steel. Any elements that appear to be structurally marginal, due either to unconservative design or deterioration effects, are identified and appropriate calculation checks made. These analyses may identify distress in the structure that has been caused by structural overload and indicate safety factors. If the calculations are inconclusive, suitable load testing may be indicated. After all of the field and laboratory results have been collated and studied and all calculations have been completed, a report is prepared.

19. Condition Assessment Criteria. Cracking is a very common damage by-product from a large number of concrete degradation mechanisms. Active concrete cracking is difficult to assess in terms of impact on structural behavior and is difficult to repair. Thus, inspection methods that support the early identification, sizing, and cause of cracking in concrete structures are of primary interest for future inspections. Also, the primary concern for all metallic constituents of concrete structures is corrosion and corrosion-related damage. Inspections that identify early signs of corrosion cell initiation and indicate the rate of propagation are similarly valuable. Two approaches based primarily on the results of visual inspections have been developed for assistance in the classification and treatment of conditions or findings that might emanate from in-service inspections of reinforced concrete structures:

visual-based and damage-based.

20. The visual-based approach uses a "three-tiered" hierarchy so that through use of different levels of acceptance, minor discontinuities can be accepted and more significant degradation in the form of defects can be evaluated in more detail (18,19). The three acceptance levels include acceptance without further evaluation, acceptance after review, and additional evaluation required. Table 1 provides criteria for acceptance without further evaluation. Criteria associated with acceptance after review and additional evaluation required are presented elsewhere (18). Evaluations under these acceptance levels will generally involve extensive application of both nondestructive and destructive testing methods and detailed analytical evaluations frequently may be required to better characterize the current condition of the structure and provide the basis for formulation of a repair strategy (if needed). Even if the analysis results indicate that the component is acceptable at present, additional assessments should be conducted to demonstrate that the component will continue to meet its functional and performance requirements during the desired service life (i.e., take into account the current structural condition and use service life models to estimate the future impact of pertinent degradation factors on performance).

21. The damage-based approach is founded on the concept that the degradation of a component in service is manifested in physical evidence or signs (e.g., measurable values), and that these signs can be categorized or classified into distinct stages or conditions in accordance with their potential impact on performance. Cracks are a frequent manifestation of degradation in reinforced concrete structures and are significant from the viewpoint that they may indicate major structural problems (active cracks); provide an important avenue for ingress of hostile environments (e.g., chloride ions and sulfate solutions); and inhibit a structure from meeting its performance requirements (e.g., water retaining). Both the width of concrete cracks and the environmental exposure are important. There have been a number of studies over the years that related maximum permissible crack widths to environmental factors through specified limits to reduce the potential for enhanced degradation through ingress of contaminants, primarily leading to corrosion of steel reinforcement (20). Some work has been done in classifying environmental exposure conditions in terms of their degree of aggressivity, degree of chemical attack of concrete by soils and water containing aggressive agents, and information is available on the influence of the moisture condition on several durability processes (e.g. carbonation, corrosion, frost attack, and chemical attack) (21). Based on this information, two approaches have been developed for assistance in the classification and treatment of conditions or findings that might emanate from in-service inspections of reinforced concrete structures. These approaches primarily are based on the results of visual inspections since these inspections provide the cornerstone of any condition assessment program for concrete structures and the approaches are related to parameters that can be measured associated with corrosion of steel embedded in concrete. Figure 2 provides a relationship between environmental exposure in terms of extent of carbonation or chloride ion content of the environment, the width of cracks present, and the necessity for additional evaluation or repair. As noted in the figure, the extent of action required increases as the severity of environmental exposure increases or the width of cracks present increases. Figure 3 provides a relationship between environmental exposure, half-cell potential readings, and necessity for further evaluation or repair. Superimposed on the half-cell potential axis are visual inspection results that might be anticipated for different degrees of severity of corrosion of steel reinforcement. It should be noted that information presented in these two figures is provided only as examples of what might be developed for specific applications, as all factors required for a detailed assessment have not been incorporated (e.g., in assessing the potential for corrosion the current density may be a more useful parameter than the absolute value of potential). Application of an approach of this type should utilize the judgement of a suitably qualified and experienced responsible engineer.

22. Remedial/Preventative Measures Considerations. Reinforced concrete structures almost from the time of construction start to deteriorate due to exposure to the environment (e.g., temperature, moisture, and cyclic loading) (22). The rate of deterioration is dependent on the component's structural design, materials selection, quality of construction, curing, and aggressiveness of its environmental exposure. Figure 4 presents the relationship between concrete performance and time. Termination of a component's service life occurs when it can no longer meet its functional and structural requirements. Results provided through periodic application of in-service inspection techniques as part of a condition assessment program can be used to develop and implement a remedial action prior to the structure achieving an unacceptable level of performance.

23. Corrosion resulting from either carbonation or the presence of chlorides is the dominant type of distress that impacts reinforced concrete structures. Corrosion mechanisms and types (e.g., uniform, pitting, bimetallic, crevice,

etc.) as well as conditions that affect the corrosion rate (e.g., oxygen, electrolyte conductivity, ion concentration, and temperature.) have been summarized (23). 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. Remedial measures include damage repair, cathodic protection, inhibitors, chloride removal, membrane sealers, stray current shielding, dielectric isolation, coatings, and environmental modifications. Stray electrical current resulting from any of a number of sources (e.g., cathodic protection systems, high voltage direct current systems, and welding operations) could also lead to corrosion. Techniques to detect stray current include half-cell potential versus time measurements, half-cell potential versus distance measurements, and cooperative (interference) testing. Mitigation measures for stray current include prevention or elimination of the current source, installation of cathodic protection, draining the current from the source, and shielding the structure from the source. Use of sacrificial or impressed current cathodic protection systems as both a rehabilitation technique for corroding structures and a corrosion prevention technique for steel that may lose its inherent passivity at a later date were investigated. Design considerations, advantages and disadvantages, and commentary on when cathodic protection should and should not be used were also addressed.

24. Damage repair practices commonly used for reinforced concrete structures in Europe and North America have been reviewed. In Europe, activities have concentrated on repair of damage resulting from corrosion of steel reinforcement (24). Basic repair solutions 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. Repair strategies and procedures were developed in the form of flow diagrams. In North America, activities have primarily addressed repair of infrastructure-related facilities (e.g., highways and bridges). A report has been prepared that summarizes techniques and materials for repair of damaged concrete (19). Mitigation and repair methods are provided for cracking, spalling, delaminations, water seepage, honeycomb and voids, alkali-aggregate attack, external sulfate attack, and corrosion damage.

5.2 Quantitative Methodology for Continued Service Determinations

25. Evaluation of structures for continued service should provide quantitative evidence that their capacity is sufficient to withstand future demands within the proposed service period with a level of reliability sufficient for public safety. Structural aging will cause the integrity of structures to evolve over time (e.g., a hostile service environment may cause structural strength and stiffness to degrade). Uncertainties that complicate the evaluation of aging effects arise from a number of sources: inherent randomness in structural loads, initial strength, and degradation mechanisms; lack of in-service inspection measurements and records; limitations in available databases and models for quantifying time-dependent material changes and their contribution to structural capacity; inadequacies in nondestructive evaluation; and shortcomings in existing methods to account for repair. Any evaluation of the reliability of a reinforced concrete structure during its service life must take into account these effects, plus any previous challenges to the integrity that may have occurred.

26. <u>Time-Dependent Reliability Approach</u>. Structural loads, variations in engineering material properties, and strength degradation mechanisms are random in nature. 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 loads (impulses) is described by a Poisson process, with the mean (stationary) rate of occurrence, λ, random intensity, S_j, and duration, τ (25). The number of events, N(t), to occur during service life, t, is described by the probability mass function,

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

The intensity of each load is a random variable, described by cumulative distribution function (CDF) $F_i(x)$. This process can be generalized to one in which 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 duration of each load is exponential, with average duration $\tau = 1/\lambda$. Loads that fluctuate with sufficient rapidity in time that they cannot be modeled by a sequence of discrete pulses can be modeled as continuously parametered stochastic processes.

27. The strength, R, of a reinforced concrete component is described by (26)

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

in which X_1, X_2, ... are basic random variables that describe yield strength of reinforcement, compressive or tensile strength of concrete, and structural component dimensions or section properties. The function $R_m(\bullet)$ describes the strength based on principles of structural mechanics. Modeling assumptions invariably must be made in deriving $R_m(\bullet)$, 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 $R_m(\bullet)$ when values of all variables X_i are known. The probability distribution of B can be assumed to be normal (27). A more accurate behavioral model leads to a decrease in the variability in B and thus in R. Probability 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.

28. 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,

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

in which $F_R(x)$ and $f_S(x)$ are the cumulative distribution function of R and probability density function (PDF) of S (28). Equation (5) provides an instantaneous quantitative measure of structural reliability, provided that $P_f(t)$ can be estimated and/or validated (29). It does not convey information on how future performance can be inferred from past performance.

29. 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 Eqn. (5). Indeed, it is difficult to use reliability analysis for engineering decision analysis without having some time period (e.g., an in-service maintenance interval) in mind. 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,

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

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

30. 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 (30)

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

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 $(t, t+\Delta t)$, 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 $i = 1$.

31. 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. When a structure is inspected and/or repaired, something is learned about its in-service condition that enables the probability distribution of strength to be updated. The density function of strength, based on prior knowledge of the materials in the structure, construction, and standard methods of analysis, is indicated by $f_R(r)$. Scheduled inspection, maintenance and repair cause 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,

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

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.

32. 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 (31). 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.

33. Application Examples. Time-dependent reliability concepts are illustrated with a simple conceptual example of a reinforced concrete slab designed using the requirements for flexure strength found in ACI Standard 318 (2)

$$0.9\,R_n = 1.4\,D_n + 1.7\,L_n, \tag{14}$$

in which R_n is the nominal or code resistance, and D_n and L_n are the code-specified dead and live loads, respectively. Three scenarios are considered: (1) the strength of the slab changes with time, initially increasing as the concrete matures and then decreasing due to (unspecified) environmental attack, (2) the strength degrades linearly to 90% of its initial strength at 40 years, and (3) the strength remains constant with time (Fig. 5). In general, the behavior of resistance over time must be obtained from mathematical models describing the degradation mechanism(s) present . The statistics used in this example are provided elsewhere (1).

34. Figure 6 compares the limit state probabilities [F(t) = 1 - L(t) in Eqn. (7)] obtained for the three degradation models considered in Fig. 5 for service lives (0,t) ranging up to 60 years. When R(t) = R(0) and no degradation of strength occurs, a result is obtained analogous to what has been done in probability-based code work to date (29). Neglecting strength degradation entirely in a time-dependent reliability assessment can be quite unconservative, depending on the time-dependent characteristics of strength.

35. If we now consider the following which involves a single inspection/repair:

a. Every part of the structure is fully inspected and all detected damages are repaired completely.

b. The initiation of damages is described by a stationary Poisson process with a parameter ν = 5/yr that is dependent on the surface area or volume of the structure.

c. Damage grows linearly with time as described below with α = 1 (i.e., linear)

$$X_j(t) = \begin{cases} 0 & ; 0 \le t < T_{I_j} \\ C_j\left(t - T_{I_j}\right)^{\alpha} & ; t \ge T_{I_j} \end{cases}$$

in which $X_j(t)$ j = 1, 2, ... is the intensity of damage at time t; T_{Ij}, j = 1, 2, ... are the random initiation times of damage; Cj's are damage growth rates that are identically distributed and statistically independent random variables described by a CDF $F_C(c)$; and α is a deterministic parameter. The assumption of independent Cj's provides a conservative estimate of failure probability. Parameters C and α depend on the degradation mechanism.

d. The degradation rate, C, is lognormally distributed with mean value, $\mu_C = 0.00125$, that corresponds to $E[X(40)|T_I = 0] = 0.05$, and with a coefficient of variation, $V_c = 0.5$.

36. The effect on the mean degradation function of inspection/repair described by several detectability functions is illustrated in Fig. 7. The first detectability function considered is a step function in which $x_{th} = 0.03$; in the second, X_{th} is uniformly distributed [i.e., $d(x)$ is linear between x_{min} and x_{max}, where $d(x_{min}) = 0$ and $d(x_{max}) = 1$]; in the third and fourth, X_{th} is lognormally distributed with mean, $\mu_{X_{th}}$, equal to 0.03, and coefficient of variation, $V_{X_{th}}$, equal to 0.3 or 0.5. It is assumed that inspection/repair is carried out at $t_{Rm} = 20$ years. The mean degradation function decreases as $V_{X_{th}}$ increases (that would result in lower reliability); however, the effect of the general shape of d(x) is not significant and decreases with time elapsed since inspection. This insensitivity of the mean degradation to the choice of detectability function suggests that a general detectability function might be approximated for practical purposes by a step function with $x_{th} = \mu_{X_{th}}$. This would be advantageous for NDE technologies currently used for reinforced concrete structures because information on $\mu_{X_{th}}$ may be more readily available than information on d(x).

37. The effect of multiple inspection/repair and the mean degradation function is illustrated in Fig. 8, assuming a step detectability function and the same assumptions as used in the previous example. Inspection/repairs are carried out at 20, 30, 40, and 50 years with $x_{th} = 0.05$ when $E[X(40)|T_I = 0] = 0.05$. For comparison, the mean degradation function for a component without repair and for a component with one repair at 30 years with $x_{th} = 0.01$ is also presented in the figure. With multiple inspections/repairs, the mean degradation function can be kept within a narrow range during the service life of the structure. This suggests the existence of an optimum inspection/repair strategy in which the failure probability of the component is kept below an established target probability during its service life and the total expected cost, defined as the sum of the cost of inspections/repairs and expected cost (loss) due to failure, is minimized.

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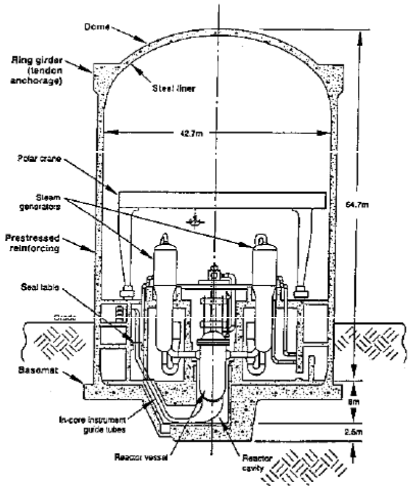
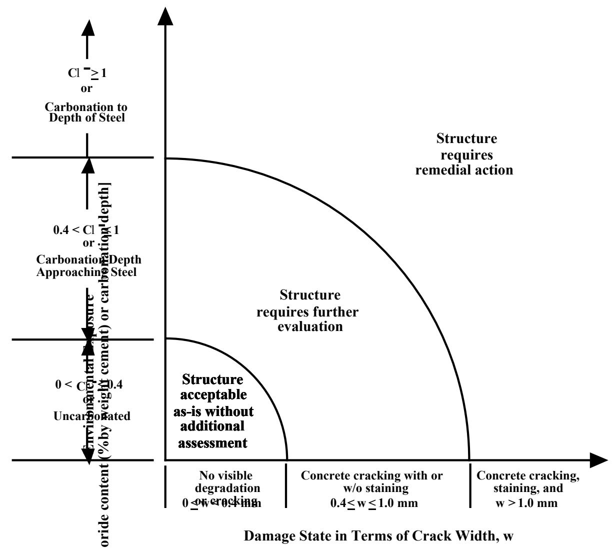
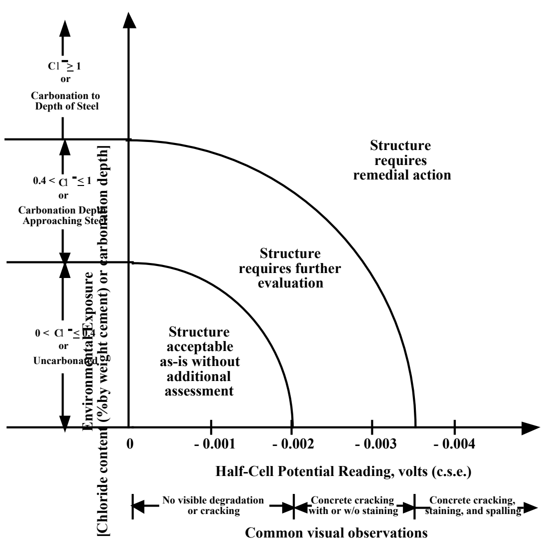
Table 1. Condition Assessment Results Considered not to Require Further Evaluation.•

1. **Unlined concrete surfaces** – Concrete surfaces that are exposed for inspection are generally acceptable without further evaluation if the following criteria are met:
   a. Absence of leaching and chemical attack;
   b. Absence of abrasion, erosion, and cavitation;
   c. Absence of drummy areas (poorly consolidated with paste deficiencies);
   d. Popouts and voids less than 20 mm in diameter or equivalent surface area;
   e. Scaling less than 5 mm in depth;
   f. Spalling less than 10 mm in depth and 100 mm in any dimension;
   g. Absence of any signs of corrosion in reinforcing steel system or anchorage components (including concrete staining or spalling);
   h. Passive cracks less than 0.4 mm in maximum width ("passive cracks" are defined as those having an absence of recent growth and absence of other degradation mechanisms such as leaching at the crack);
   i. Absence of excessive deflections, differential settlements, or other physical movements that may affect structural performance; and
   j. Absence of cement-aggregate reactions, chemical attack, fire damage, or other active degradation mechanism.
2. **Concrete surfaces lined by metal or plastic** – Concrete structures with inner surfaces protectively lined with either a metallic or plastic (non-metallic) system are judged to be acceptable without further evaluation if the following criteria are met:
   a. Without active leak-detection system
      1. Absence of bulges or depressions in liner plate (those that appear age-related as opposed to being created during construction);
      2. Absence of corrosion or other liner damage; and
      3. Absence of cracking in liner weld or base metal.
   b. With active leak-detection system
      1. No detectable leakage observed in leak detection system;
      2. Absence of any liner damage, such as noted in 2(a) above; and
      3. Absence of fluid penetration indications by other detection systems.
3. **Areas around embedments in concrete** – The condition of the concrete around embedments is acceptable without further evaluation if the following criteria are met:
   a. Concrete surface condition attributes of Criteria 1 above are met;
   b. Absence of corrosion on the exposed surfaces of embedded metal members and corrosion staining around the embedded metal;
   c. Absence of detached embedments or loose anchorages; and
   d. Absence of degradation due to vibratory loads from piping and other attached equipment.
4. **Joints, coatings, and non-structural components** – The condition of joints, protective coatings, waterproofing membranes, and other non-structural elements is acceptable without further evaluation if the following criteria are met:
   a. No signs of separation, environmental degradation, or water in-leakage are present in coatings, joints, or joint sealant material;
   b. Loss or degraded areas of coatings for structures that do not serve as a barrier to aggressive chemical flows are limited in surface area to 4000 square millimeters or less at one area, and 0.01 square meters over the gross surfaces of the structure;
   c. Absence of degradation in any waterproofing membrane protecting below-grade concrete surfaces (within the inspected area); and
   d. Non-structural components such as dewatering systems are serving their intended function.
5. **Post-Tensioning Systems** – Components of post-tensioning systems are acceptable if specific requirements are met [e.g., (32)].

• Criteria associated with "Acceptance After Review" and "Additional Evaluation Required" are available elsewhere (18).







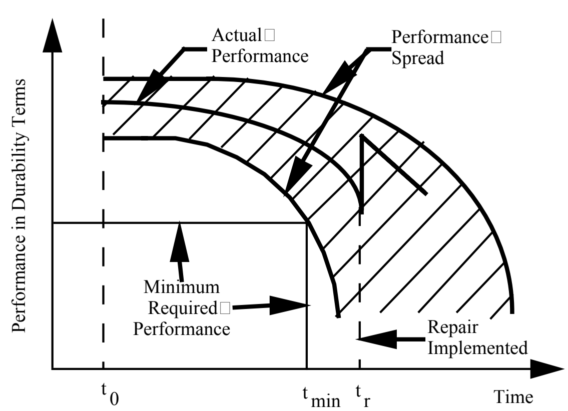


Fig. 4 Relationship between concrete performance and time.

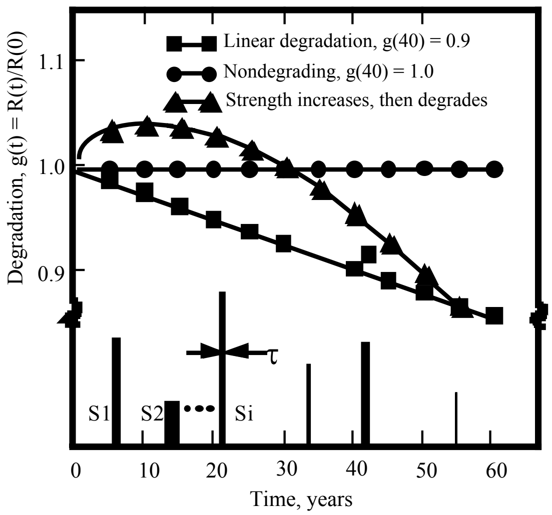


Fig. 5 Mean degradation function of one-way slab.

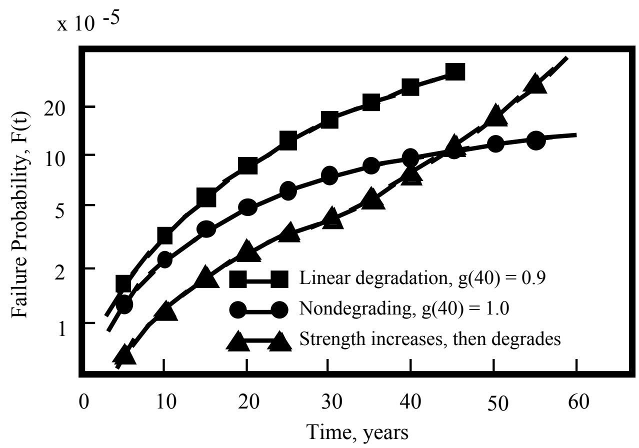


Fig. 6 Failure probability of one-way slab.

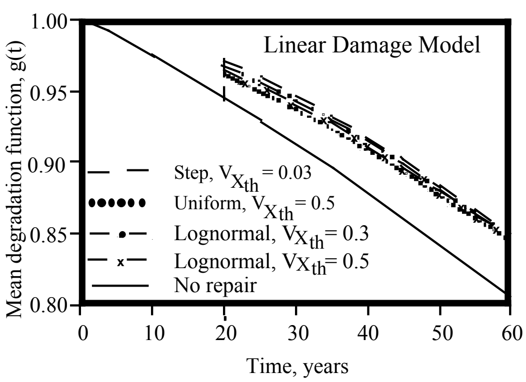


Fig. 7 Effect of several detectability functions on mean degradation function of inspection/repair.

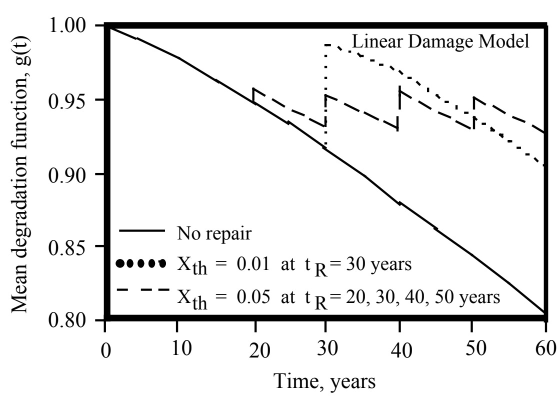


Fig. 8 Effect of multiple inspection/repairs on mean degradation function.