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Structural aging program to evaluate continued performance of safety-related concrete structures in nuclear power plants*

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The Structural Aging (SAG) Program is being conducted at the Oak Ridge National Laboratory (ORNL) for the United States Nuclear Regulatory Commission (USNRC). The SAG Program is addressing the aging management of safety-related concrete structures in nuclear power plants for the purpose of providing improved technical bases for their continued service. The program is organized into three technical tasks: Materials Property Data Base, Structural Component Assessment/Repair Technologies, and Quantitative Methodology for Continued Service Determinations. Objectives and a summary of recent accomplishments under each of these tasks are presented.

INTRODUCTION

1. Concrete structures play a vital role in the safe operation of all light-water reactor (LWR) plants since they provide foundation, support, shielding, and containment functions. History tells us that concrete can be a very durable material. However, a number of factors can compromise its performance, singly or in combination: (1) faulty design, (2) use of unsuitable materials, (3) improper workmanship, (4) exposure to aggressive environments, (5) excessive structural loads, and (6) accident conditions. Furthermore, aging of nuclear power plant concrete structures occurs with the passage of time and has the potential, if its effects are not controlled, to increase the risk to public health and safety.

BACKGROUND

2. In general, the performance of concrete materials and structures in nuclear power plants has been good. However, there have been several instances in nuclear power plants where the capability of concrete structures to meet future functional and performance requirements has been challenged due to problems arising from either improper material selection, construction/design deficiencies, or environmental effects. Examples of some of the potentially more serious instances include anchorhead failures, voids under vertical tendon bearing plates, dome delaminations, and corrosion of steel tendons and rebars. Other problems such as the presence of voids or honeycomb in concrete, contaminated concrete, cold joints, cadweld (steel reinforcement connector) deficiencies, concrete cracking, higher than code-allowable concrete temperatures, materials out of specification, misplaced steel reinforcement, lower than predicted prestressing forces, post-tensioning system buttonhead deficiencies, water contaminated corrosion inhibitors, water intrusion through basemat cracks, low tensile strength of post-tensioning tendon wire material, leaching of concrete in tendon galleries, and leakage of corrosion inhibitor from tendon sheaths also have been identified (refs 1-3).

STRUCTURAL AGING PROGRAM

3. Incidences of concrete component structural degradation indicate that there is a need for improved surveillance, inspection/testing, and maintenance to enhance the technical bases for assurances of continued safe operation of nuclear power plants. The Structural Aging (SAG) Program has the overall objective of preparing documentation that will provide the United States Nuclear Regulatory Commission license reviewers with the following: (1) identification and evaluation of the structural degradation processes; (2) issues to be addressed under nuclear power plant continued-service reviews, as well as criteria, and their bases, for resolution of these issues; (3) identification and evaluation of relevant in-service inspection or structural assessment programs; and (4) methodologies required to perform current assessments and reliability-based life predictions of safety-related concrete structures. To accomplish this objective, the SAG Program is conducting activities under three primary technical task areas: (1) materials property data base, (2) structural component assessment/repair technologies, and (3) quantitative methodology for continued service determinations.

<u>Materials property data base</u>

4. The objective of the materials property data base task is to develop a reference source which contains data and information on the time variation of material properties under the influence of pertinent environmental stressors and aging factors. This source will be used to assist in the prediction of potential long-term deterioration of critical structural components in nuclear power plants and to establish limits on hostile environmental exposure for these structures.

5. <u>Structural materials information center (SMIC)</u>. Formatting of the SMIC has been completed and results presented in a report (ref. 4). Contained in the report are detailed descriptions of the *Structural Materials Handbook* and the *Structural Materials Electronic Data Base* which form the SMIC.

6. The *Structural Materials Handbook*, when issued, will be an expandable, hard-copy reference document containing complete sets of data and information for each material. The handbook includes four volumes that will be provided in loose-leaf binders for ease of revision and updating. Volume 1 will contain performance and analysis information useful for structural assessments and safety margins evaluations, e.g., performance values for mechanical, thermal, physical, and other properties presented as tables, graphs, and mathematical equations. Volume 2 will provide test results and data used to develop the performance values in Volume 1. Volume 3 will contain material data sheets providing general information, as well as material composition and constituent material properties, for each material system contained in the handbook. Volume 4 will contain appendices describing the handbook organization, and updating and revision procedures.

7. The *Structural Materials Electronic Data Base*, when issued, will be an electronically accessible version of the *Structural Materials Handbook*. Due to software limitations, the electronic data base will not be as comprehensive as the handbook, but it will provide an efficient means for searching the various data base files to locate materials with similar characteristics or properties. The electronic data base is being developed on an IBM-compatible personal computer and employs two software programs: Mat.DB (ref. 5) and EnPlot (ref. 6). Mat.DB is a menu-driven software program for data base management that employs window overlays to access data searching and editing features. It is capable of maintaining, searching, and displaying textual, tabular, and graphical information and data contained in electronic data base files. EnPlot is a software program that incorporates pop-up menus for creating and editing engineering graphs. It includes curve-fitting and scale-conversion features for preparing engineering graphs and utility features for generating output files. The graphs generated

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with EnPlot can be entered directly into the Mat.DB data base files.

8. To date, 139 material data bases (123 concrete, 12 metallic reinforcement, 1 prestressing steel, 2 structural steel, and 1 rubber material) have been developed. Concrete material property data and information files currently contained in the SMIC include: ultimate compressive strength, dynamic modulus of elasticity, and flexural strength versus time for several different concrete materials which had been cured under a variety of conditions (air drying, moist curing, or outdoor exposure) for periods up to 50 years; ultimate compressive strength and modulus ...lasticity versus temperature at exposures up to 600°C for durations up to four months; dynamic modulus of elasticity, ultimate compressive strength, flexural strength and weight change versus radiation exposure; ultimate compressive strength for stressed (maintained at 20 to 55% baseline compressive strength while heated) and unstressed specimens versus temperature for exposures up to 871°C; weight loss versus time for specimens subjected to sulfuric acid concentrations (by weight) of 0.0016 to 0.02%; length change versus time for specimens subjected to wet (2.1% Na_2SO_4 solution) - dry cycling; creep of sealed concrete exposed to temperatures of either 20°, 40°, or 70°C while loaded to ~0.2 the 28-day ultimate compressive strength; porosity versus time for thermal exposures to either 20°, 40°, or 70°C; Poisson's ratio versus time for thermal exposures to 232°C for periods up to 1198 days; and bond stress versus slip for reinforced concrete bond test specimens exposed for 14 days to either direct or alternating current (potential up to 20 volts). Metallic reinforcement data and information files include: ASTM A 615 uncoated, plain and uncoated, deformed carbon steel reinforcing bar material ambient and temperature-dependent engineering stress-strain performance curves (Grades 40, 60 and 75), and S-N (fatigue) performance curves (Grade 40 material); and ASTM A 15 uncoated, plain and uncoated, deformed billet steel ambient engineering stress-strain performance curves (all grades), and S-N (fatigue) performance curves and temperature-dependent stress-strain performance curves (Intermediate Grade material). Prestressing tendon data and information files (ASTM A 421, Type BA material) include a temperature-dependent engineering stress-strain performance curve, a tensile yield strength vs temperature performance curve, an ultimate tensile strength vs temperature performance curve, and an ultimate tensile elongation vs temperature performance curve. Structural steel data and information files (ASTM A 36 material) include a temperature-dependent engineering stress-strain performance curve, a tensile yield strength vs temperature performance curve, an ultimate tensile strength vs temperature performance curve, and an ultimate tensile elongation vs temperature performance curve. Rubber data and information files [ethylene propylene diene (EPDM), ASTM D 1418] include a temperature-dependent hardness vs time performance curve.

9. Data collection. Several U.S. organizations have been contacted to pursue the possibility of removing and testing concrete core samples from prototypical structures. These contacts have resulted in procurement of samples from the Shippingport Power Station; EBR-II site; Palisades, Midland, Dresden, Braidwood, Quad Cities, Zion, and LaSalle Power Stations; and Vallecitos Nuclear Center. Specimens have been tested and material property data bases developed for heavyweight and normal-weight concrete materials obtained from the EBR-II site, and for normal-weight concrete materials obtained from the Vallecitos Nuclear Center and Midland Power Station (ref. 7). Specimens obtained from the Palisades, Midland, Dresden, Braidwood, Quad Cities, Zion, and LaSalle Power Stations have been tested, but material property data bases have not been completed because baseline data and information have not been completely assembled.

10. In a related activity, prototypical nuclear power plant concrete materials have been tested under a subcontract with Taywood Engineering Ltd. (ref. 8). Twenty-nine specimens cast in conjunction with the nuclear power stations at Wylfa, Heysham, Hartlepool, Torness, and Sizewell "B" were tested. The specimens had ages from 4 to 24 years and had been stored in a sealed condition at temperatures ranging from 10° to 95°C while either loaded to 13.8 MPa or unloaded. Variables investigated included age of specimen, concrete mix design, loaded or unloaded while curing, and storage temperature. Prior to compression testing each specimen was submitted to a series of nondestructive tests (density, ultrasonic pulse velocity, Schmidt hammer, surface hardness, and dynamic modulus of elasticity). Results indicate that, with only one or two exceptions, there was a consistent trend for the concrete moduli of elasticity and compressive strength to increase with age for each of the mixes. From the limited results which were available for specimens continuously stored at 10° to 20°C, increases in modulus of elasticity ranged from 3% for the Sizewell "B" concrete (test age of 4.8 years) to 112% for the Wylfa concrete (test age of 21 years).

11. Material behavior modeling. A review and evaluation was conducted of to identify and evaluate models and accelerated aging techniques and methodologies which can be used in making predictions of the remaining service life of concrete in nuclear power plants (ref. 9). Methods which are often used for predicting the service lives of construction materials include: (1) estimates based on experience, (2) deductions from performance of similar materials, (3) accelerated testing, (4) applications of reliability and stochastic concepts, and (5) mathematical modeling based on the chemistry and physics of degradation processes. The most promising approach for predicting the remaining service life of concrete involves the application of mathematical models of the degradation processes. Each of the degradation processes was reviewed based on its mechanism, likelihood of occurrence, manifestations, and detectability. Models identified for each process were evaluated considering: (1) their basis (e.g., theoretical, empirical, or some combination), (2) correctness of assumptions used in their derivation, (3) availability of data to perform an evaluation, (4) their applicability to the problem, and (5) degree of quantitativeness of their predictions. A major conclusion of the study was that theoretical models need to be developed, rather than relying solely on empirical models. Predictions from theoretical models are more reliable, far less data are needed, and the theoretical models would have wider applications, e.g., applicable to a broad range of environmental conditions. Deterministic and stochastic models should be combined to give realistic predictions of the service life of an engineering material. Purely stochastic models have limited application because of the lack of adequate data to determine the statistical parameters. Accelerated tests do not provide a direct method for making the life predictions, but can be useful in obtaining data required to support the use of analytical models. In comparison to predicting the life of new concretes, few studies were identified on predicting the remaining service life of in-service concretes exposed to environmental stressors potentially encountered in nuclear power plant facilities, i.e., corrosion of steel reinforcement, sulfate attack, alkali-aggregate reactions, frost attack, leaching, irradiation, salt crystallization and microbiological attack.

12. A collation of survey data and a durability assessment review of reinforced concrete structures contained as a part of several nuclear power stations located in England has been completed (ref. 10). The nuclear power station surveys were aimed primarily at providing data which can be used to predict the onset of corrosion of steel reinforcement due to carbonation or chloride ingress. Corrosion of steel reinforcement was recognized to be a two-stage process: activation and propagation. The present state-of-the-art for modeling carbonation and chloride migration into concrete was reviewed along with factors which affect the time to corrosion activation, e.g., concentration level, environmental conditions, and diffusion coefficient. Survey data obtained at selected locations in the Windscale Advanced Gas-Cooled Reactor, and Hinkley Point "A," Bradwell and Wylfa Nuclear Power Stations included chloride content, carbonation depth, compressive strength, resistivity, moisture content, sorptivity, oxygen diffusivity, and petrography. Environmental conditions and survey results were used to predict the onset of steel reinforcement corrosion due to either carbonation or chloride ingress at selected locations in these stations. Predicted maximum age for steel reinforcement corrosion activation due to carbonation or chloride ingress ranged from 31 to >140 years and 43 to >140 years, respectively. Visual surveys at the stations indicated that for interior structures the steel reinforcement and concrete were sound with no corrosion, whereas the external concrete in most instances exhibited only a few localized areas of cracking and spalling. The exception to this was at Wylfa where the cooling plant exhibited severe rust staining and spalling due to chloride ion penetration. Other survey results were that the depths of carbonation reached up to 50 mm for uncoated internal concrete, chloride ingress was generally low (<0.05% in surface 10 mm) except for Wylfa, and the concretes were free of alkali-silica reactivity and sulfate attack.

Structural component assessment/repair technology

13. The objectives of this task are to: (1) develop a systematic methodology which can be used to make quantitative assessments of the presence, magnitude, and significance of any environmental stressors or aging factors which adversely impact the durability of safety-related concrete structures in nuclear power plants; and (2) provide recommended in-service inspection or sampling procedures

which can be utilized to develop the data required both for evaluating the current condition of concrete structures and for trending the performance of these components. Associated activities include an assessment of techniques for repair of concrete components which have experienced an unacceptable degree of deterioration, and the identification and evaluation of techniques for mitigation of any environmental stressors or aging factors which may act on critical concrete components.

14. LWR critical concrete component classification. A methodology has been developed which provides a logical basis for identifying the critical concrete structural elements in a nuclear plant and the degradation factors which can potentially impact their performance (ref. 11). Numerical rating systems are used to establish the relative importance of the structure's subelements, safety significance of each Category I structure, and the influence of environmental exposure. Because of the variability in likelihood of occurrence of degradation to concrete structures in U.S. light-water reactor plants due to design differences, material utilization, geographical location, etc., the grading system for degradation factors is stated in terms of a range of possible values. Determination of the relative ranking of Category I structures and their subelements is based on the weighted contributions of the (1) structural importance of subelements, (2) safety significance, (3) environmental exposure, and (4) degradation factor significance.

15. NDE/sampling inspection technology. A review of direct and indirect techniques that can be used to detect degradation of concrete materials and structures was conducted (ref. 12). Capabilities, accuracies and limitations of available nondestructive evaluation testing techniques were assessed. Also included in the review was a discussion of methodologies used to indicate the condition of concrete structures and emerging methods under development which potentially have application to detection of concrete degradation, i.e., leakage flux, polarization resistance, ultraviolet radiation, etc.

16. Correlation curves and statistical data were developed for selected nondestructive testing techniques (ref. 13). This information is required where destructive and nondestructive tests can not be conducted in tandem at noncritical locations to develop a regression relation between the two tests. Monovariant linear regression analyses were applied to data obtained from publications on selected nondestructive testing techniques that are commonly used to indicate concrete compressive strength, e.g., break-off, pullout, rebound hammer, ultrasonic pulse velocity, and probe penetration. Figure 1 presents an example of the relationship between concrete cylinder compressive strength and rebound number that was obtained from data for gravel-aggregate concretes having water-cement ratios from 0.37 to 0.56, by weight, and coarse aggregate contents from 0.45 to 0.49, by mass.

17. Remedial/preventative measures. A report has been prepared providing an overview of the European perspective on concrete repair, e.g., methods of damage assessment and repair strategies (ref. 14). Specific topics covered include: (1) criteria used in selection of a repair procedure, (2) descriptions of various repair materials and procedures

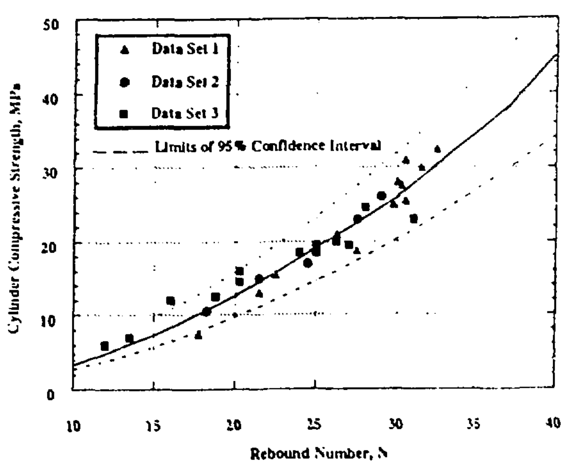

Fig. 1 Relationship between cylinder compressive strength and rebound number for selected gravel aggregate concretes

currently utilized, (3) an assessment of the effectiveness of the various repair techniques as determined by in-situ evaluation (testing) and performance history, and (4) an assessment of the future direction of concrete repairs within Europe. Although there are no European standards governing repair of concrete, there are several documents that provide guidelines, with the most widely developed regulations having been prepared by the German Committee on Reinforced Concrete (ref. 15). Steel reinforcement corrosion occurring as a result of concrete carbonation or chloride presence are the most important sources of reinforced concrete distress in Europe. For carbonation, the emphasis has been placed on anti-carbonation surface treatments, protective properties of patch materials, and the durability/compatibility of these materials. For chloride attack, efforts are underway to provide an improved understanding of the corrosion mechanisms, the mechanism of incipient anode development, and the use of cathodic protection to overcome the problem.

18. A complementary activity to the one described above has reviewed North American practices for concrete repair. The distinction between the two efforts is that European repair activities have primarily (but not exclusively) addressed those for corrosion damaged reinforced concrete in building and general civil engineering structures, while in North America, repair activities have concentrated on roads and bridge structures. Basic components of the report which has been prepared (ref. 16) include discussions of: when a specific repair technique is applicable, e.g. specific crack sizes; how the techniques or materials are used, e.g., injection, grouting; how to evaluate and test a repair; how to maintain the repair after it has been installed; the expected life of the repair technique; methods for determining when a repair has failed; and methods for re-repair. Information specifically addressing repair of reinforced concrete structures in light-water reactor plants was assembled through responses to a questionnaire which was sent to the utilities. In addition to a general description of the particular plant, the questionnaire requested each utility to provide information on inspection procedures utilized, types of deterioration that have occurred, the deterioration mechanism(s), repair actions that have been undertaken, research investigations on repair materials, and performance history of repair procedures utilized. Responses to the questionnaire provided by 29 sites representing 41 units indicate that the majority of the plants perform inspections of the concrete structures only in compliance with the integrated leak-rate test requirements and surveillances of the prestressed concrete containment post-tensioning systems. The 12 plants which do conduct regular inspections specifically addressing the concrete structures do so at intervals ranging from one to five years and rely mainly on visual techniques. The performance of the concrete structures, in general, has been good with the primary forms of degradation that have occurred being concrete cracking, spalling, and staining, and steel reinforcement corrosion. The most common causes of deterioration were drying shrinkage, acid/chemical attack, thermal movement, freeze/thaw cycles and sea water exposure. The most common locations of deterioration for the pressurized-water reactor plants were the containment dome and in the walls and slabs of auxiliary structures. For the boiling-water reactors, the locations of deterioration were primarily in the walls, slabs, and equipment supports or pedestals of the reactor buildings and auxiliary structures. Most of the repair activities were associated with problems during initial construction (cracks, spalls, and delaminations), with the repairs performed on an as-needed basis. Little information was available on the materials used for repair, the repair procedure, or the durability of the repair. Where the performance of a repair was evaluated, visual inspection was used.

19. A review of corrosion of reinforced concrete structures, with an emphasis on stray electrical current-induced corrosion and use of cathodic protection to control the occurrence of corrosion in these structures has been completed (ref. 17). Types of corrosion that can occur on metals embedded in concrete include uniform, bimetallic, fretting, crevice, pitting, selective leaching, intergranular, stress corrosion cracking, corrosion fatigue, and hydrogen embrittlement. Methods which are available to detect corrosion include visual observations, half-cell potential measurements, delamination detection, electrolyte chemistry, corrosion monitors, acoustic emission, radiography, ultrasonics, magnetic perturbation, metallurgical properties, and electrical resistance. Remedial methods to control corrosion include damage repair, cathodic protection, inhibitors, chloride removal, membrane sealers, stray current shielding, dielectric isolation, coatings, structural modifications to prevent water accumulation, material selection, and environmental modification. Stray electrical current is any current flowing in a path other than its

intended circuit and it has a magnitude inversely proportional to the resistance of the path(s). With the exception of prestressing tendons, the portion of the structure picking up stray current will not experience corrosion damage. In the case of prestressing tendons, hydrogen embrittlement corrosion may result. For the nuclear power industry stray current can be anticipated from a variety of sources, with the most significant being cathodic protection systems, high voltage DC systems, and DC welding operations. Half-cell potential versus time study, half-cell potential versus distance study, and cooperative (interference) testing are techniques commonly used to detect stray current. Mitigation measures for stray current include prevention or elimination of the current source, installation of cathodic protection, draining the current from the affected source, and shielding the structure from the source. Cathodic protection mitigates the corrosion reaction by imposing direct current flow between an anode placed on the concrete surface and the metal to be protected. To be most effective, cathodic protection requires electrical continuity of all metallic components within a concrete structure. Since cathodic protection systems are DC, they have the potential to cause stray current corrosion in other structures, and application of cathodic protection to high-strength steel used in prestressing wires or strands may result in embrittlement due to generation of hydrogen at the cathode.

20. A systematic rating system for application in the repair of degraded reinforced concrete structures in nuclear power plants is being developed. Similar computerized rating systems have been developed for evaluating general civil works concrete structures, e.g., BRAIN (Building Rating Analysis and Investigation System). Existing damage assessment and repair prioritization systems for bridge and building structures were reviewed to determine their applicability to nuclear power plant concrete structures and an approach was identified for development of a rating system for nuclear power plant concrete structures (ref. 18). The approach used in development of the damage rating system is similar to that for the aging assessment methodology, except that instead of using four factors to develop a rating (repair prioritization number), two factors are considered, i.e. the environmental exposure and the damage significance.

21. A qualitative presentation of the repair rating system for reinforced concrete structures subjected to chloride environments is shown in Fig. 2. Criteria are being developed so that a quantitative assessment of the damage to a structure can be made, i.e., environmental exposure and damage values are being developed so that scales can be applied to Fig. 2 and regions indicating testing and evaluation or repair actions delineated. When completed, results

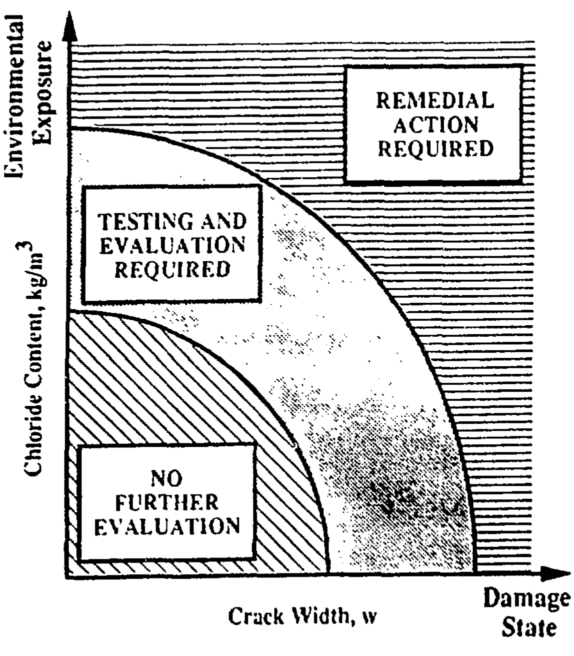

Fig. 2 Model for development of repair prioritization methodology

developed under this phase will provide a logical connection between the structural aging assessment methodology and the repair activities noted above.

Quantitative methodology for continued service determinations

22. The objective of this task is to develop a methodology to facilitate quantitative assessments of current and future structural reliability and performance of concrete structures in nuclear power plants. Time-dependent reliability analysis provides a framework for performing condition assessments of existing structures and for determining whether in-service inspection/maintenance is required to maintain reliability and performance at the desired level. The methodology integrates information on degradation and damage accumulation, environmental factors, and load history into a decision tool that provides a quantitative measure of structural reliability and performance under projected future service conditions based on an assessment of a new or existing structure.

23. Reliability-based condition assessment. The strength of structural members and components can be described statistically by data gathered in research over the past decade to develop improved bases for structural design of new reinforced concrete structures (refs 19-20). Time-dependent changes in concrete strength due to aging phenomena were not considered in developing these statistics, and they are not directly applicable to the evaluation of existing, possibly degraded, structures with a given service history (ref. 21). The statistical descriptions of concrete structure strength must account for such aging effects. This can be done by modeling the structural resistance as a time-dependent function,

$$R(t) = R_0\, g(t) \tag{1}$$

in which R_0 is the initial resistance and $g(t)$ is a time-dependent degradation function defining the fraction of initial strength remaining at time, t. Conceptually, a function $g(t)$ can be associated with each environmental stressor (ref. 9), and most significant degradation mechanisms have been identified, at least qualitatively.

24. Structural loads occur randomly in time and are random in their intensity. If the load intensity varies slowly during the load event, its effect on the structure is essentially static. Moreover, the duration of significant load events usually is short, and such events occupy only a small fraction of the total life of a structure. With these assumptions, structural loads can be modeled as a sequence of pulses, the occurrence of which is described by a Poisson process with mean rate of occurrence, λ, and duration, τ. The pulse intensities S_j are assumed to be identically distributed and statistically independent random variables described by the cumulative distribution function $F_S(x)$. Many of the loads for which nuclear power plant structures are designed can be modeled by such processes (ref. 22). A summary of the parameters describing several load processes is given in Table 1. Some of these parameters were determined through a consensus estimation survey (ref. 22).

25. Time-dependent reliability analysis. The reliability function, $L(t)$, is defined as the probability that the structure survives during interval of time $(0,t)$. If n events occur within time interval $(0,t)$, the reliability function for a structural component can be represented as:

$$L(t) = P\left[R(t_1) > S_1 \cap R(t_2) > S_2 \cap \cdots \cap R(t_n) > S_n\right]. \tag{2}$$

Taking into account the randomness in the number of loads and the times at which they occur as well as in the initial strength, and assuming that $g(t)$ is deterministic, the reliability function becomes (refs 23-24),

Table 1. Load process parameters.

	Mean*	C.O.V	pdf	λ (yr^{-1})	τ
Dead Load	1.00 D_n	0.07	Normal	...	40 years
Live Load	0.40 L_n	0.50	Type I	0.5	3 months

*D_n and L_n are nominal loads.

$$L(t)=\int_0^{\infty}\exp\left[-\lambda t\left[1-\frac{1}{t}\int_0^t F_S\left(r\cdot g(t)\right)dt\right]\right]f_{R_O}(r)\,dr \qquad (3)$$

in which $f_{R_O}(r)$ = probability density function (pdf) of the initial strength R_O. The limit state probability or probability of failure during (0,t) is,

$$F(t)=1-L(t). \qquad (4)$$

26. The hazard function, or failure rate, h(t), is defined as the probability of failure within time interval (t,t+dt), given that the component has survived up to time t. This conditional probability can be expressed as,

$$h(t)=-\frac{d}{dt}\ln L(t). \qquad (5)$$

The reliability function can be determined from h(t) as,

$$L(t)=\exp\left[-\int_0^t h(\xi)d\xi\right]. \qquad (6)$$

When structural failure occurs due to aging, h(t) increases with time. The common assumption in some time-dependent reliability studies that the failure rate increases linearly with time gives rise to a Rayleigh distribution for the limit state probability, F(t). This assumption generally is not valid for concrete structures in nuclear plants.

27. The failure probability of a structural component under stationary random loading can be evaluated as a function of time if the degradation function defining the fraction of initial strength remaining at time, t, and the probabilistic characteristics of the initial strength and loads modeled as stochastic processes are known (ref. 25). In order to evaluate the effect of periodic inspection and maintenance on the failure probability of a structure, it is necessary to relate the strength degradation to the damage intensities and to determine the impact of various repair strategies on strength.

28. <u>Degradation function based on individual damage intensities.</u> The damage intensity is modeled in the abstract as a state variable taking a value within the interval [0, 1]; the values 0 and 1 indicate no damage and no residual strength, respectively. An example of this state variable would be the ratio of area of reinforcement lost due to corrosion to the original area. The following assumptions are made:

(a) Initiation of damages in a component is described by a Poisson process in which the expected number of damages in time interval $(t, t+\Delta t]$ is $\int_t^{t+\Delta t}\nu(\tau)d\tau$ for $t>0$. $\nu(\tau)$ is dependent on the surface area or volume of the component.

(b) Damages initiate homogeneously over the surface area or volume of the component.

(c) Once damage initiates at location j, it grows according to,

$$X_j(t)=\begin{cases}0 & ;0\le t<T_{I_j}\\ C_j\left(t-T_{I_j}\right)^{\alpha} & ;t\ge T_{I_j}\end{cases} \qquad (7)$$

in which $X_j(t)$'s are the intensity of damage at time t, T_{I_j}'s are the random times at which damage initiates, C_j's are damage growth rates which are assumed to be identically distributed and statistically independent random variables described by a cumulative distribution function (cdf) $F_C(c)$, and α is a deterministic parameter. Parameters C and α depend on the degradation mechanism [e.g., (ref. 26)].

(d) The degradation function, $G(t)$, for a component, defining the fraction of initial strength remaining at time t, can be given in terms of damage intensities as,

$$G(t)=1-\max_{all\ j}\left\{X_j(t)\right\}. \qquad (8)$$

29. Consider damages which initiate within interval $(t_1, t]$. Given that the number of these damages equals n, the rank-ordered initiation times, $T_{I_1}, \ldots, T_{I_n}$ are n order statistics of random variables $W_{I_1}, \ldots, W_{I_n}$ which are statistically independent and identically distributed with probability density function (pdf) expressed as (ref. 27),

$$f_{W_I}(w)=\begin{cases}\dfrac{\nu(w)}{\int_{t_1}^{t}\nu(\tau)d\tau} & ;t_1\le w\le t\\ 0 & ;\text{Otherwise}\end{cases} \qquad (9)$$

From assumption (c), the cdf of $X_j(t)$, $F_X(x;\, t_1, t)$, is

$$F_X(x;\, t_1,\, t)=\int_{t_1}^{t}F_C\left(\frac{x}{(t-\tau)^{\alpha}}\right)f_{W_I}(\tau)d\tau. \qquad (10)$$

The cdf of $X_{max}\,(t_1;t) = \max_j\left\{X_j(t)\text{ initiating within }(t_1, t]\right\}$ (ref. 28), is

$$F_{X_{max}}(x;\, t_1,\, t)=\exp\left[-\int_{t_1}^{t}\nu(\tau)d\tau\left\{1-F_X(x;\, t_1, t)\right\}\right]. \qquad (11)$$

From assumption (d), the mean of the degradation function is evaluated by,

$$E[G(t)]=1-\int_0^1\left[1-F_{X_{max}}(x;\, 0,\, t)\right]dx. \qquad (12)$$

In the course of the analysis, it was found that the variability in $G(t)$ has a secondary effect on the time-dependent reliability of a component, and thus the reliability can be evaluated considering only the mean of $G(t)$, defined as $g(t)$ (ref. 27).

30. <u>Degradation function after repair</u>. No nondestructive evaluation (NDE) method can detect a given defect with certainty. The imperfect nature of NDE methods must be described in statistical terms. Figure 3 illustrates conceptually the probability, $d(x)$, of detecting a defect of size x. Such a relation exists, at least conceptually, for each in-service inspection technology.

31. Assume that during inspection/maintenance the entire component is inspected, that all detected damages are repaired immediately and completely, and that the repaired parts of the component are restored to their initial strength levels. Then the effect of inspection/maintenance on $g(t)$ depends on the detectability function, $d(x)$, associated with the NDE method. The inspection with higher $d(x)$ makes repair more likely and, accordingly, leads to higher values of the degradation function, $g(t)$. In the limit, if an inspection is perfect, i.e., $d(x) = 1$ for $x > 0$, then the component is restored to its original condition by the repair.

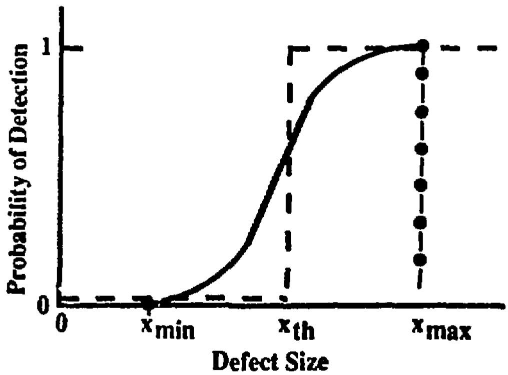

Fig. 3 Probability of detection of defect of size x

32. First assume that the detectability function, $d(x)$, is defined as,

$$d(x)=\begin{cases}0 & ; \quad 0\le x< x_{th}\\ 1 & ; \quad x_{th}\le x\le 1\end{cases} \tag{13}$$

where x_{th} is the minimum detectable value of damage (see Fig. 3). The same detection threshold values are assumed for all inspections. Following m inspections at $t_R = \{t_{R_1},\ldots,t_{R_m}\}$, some of the damages are repaired and the cdf describing $X(t)$ and the number of damages existing at time $t > t_{R_m}$, $N(t)$, changes. The intensities of damages that initiate after t_{R_m} are independent of repair, and only the pdf of the intensities of damages initiating before t_{R_m} is updated. Let us consider damages which exist at time t and initiate with (A) $(0, t_{R_m}]$ and (B) $(t_{R_m}, t]$. The number of damages left unrepaired after t_{R_m}, $N(A)$, can be described by a filtered Poisson process with a parameter $p\cdot\nu(w)$ where,

$$p = P\left[A \text{ damage is not repaired by } t_{R_m}\right] = F_X\left(x_{th}, 0, t_{R_m}\right) \tag{14}$$

while the number of damages initiating within $(t_{R_m}, t]$, $N(B)$, is described by a Poisson process with a parameter $\nu(w)$. In other words, the number of defects existing at time t can be described by a nonstationary Poisson process within a parameter $\nu''(w)$ given by,

$$\nu''(w)=\begin{cases}p\cdot\nu(w) & ;0< w\le t_{R_m}\\ \nu(w) & ;t_{R_m}< w\le t.\end{cases} \tag{15}$$

Therefore, the procedure to estimate the degradation function for a component before an inspection/repair can be used to estimate the function after multiple inspection/repair, replacing $\nu(w)$ by $\nu''(w)$, and $F_X(x; 0, t)$ by the updated cdf $F''_X(x; t_{R_m}, t)$.

33. By the theorem of total probability, $F''_X(x; t_{R_m}, t)$ can be expressed as,

$$F_X^{*}\left(x; t_{R_m}, t\right) = F_{X_{(A)}}\left(x; t_{R_m}, t\right)\cdot P(A) + F_{X_{(B)}}\left(x; t_{R_m}, t\right)\cdot P(B) \tag{16}$$

in which $F_{X_{(A)}}\left(x; t_{R_m}, t\right)$ and $F_{X_{(B)}}\left(x; t_{R_m}, t\right)$ are the cdf of intensity of damages in groups (A) and (B), respectively, and $P(A)$ and $P(B)$ are the probabilities that a defect belongs to group (A) or (B),

$$P(A) = P\left[W_i\le t_{R_m}\right] = \int_0^{t_{R_m}} f_{W_i}(w)\,dw \tag{17}$$

$$P(B) = 1 - P(A) \tag{18}$$

in which $f_{W_i}(w)$ is evaluated by Eqn. (9) replacing $\nu(w)$ with $\nu''(w)$. $F_{X_{(A)}}\left(x; t_{R_m}, t\right)$ is expressed as,

$$F_{X_{(A)}}\left(x; t_{R_m}, t\right) = P\left[X(t)< x \,\middle|\, X\left(t_{R_m}\right)< x_{th}\right]$$

$$=\frac{\int_0^{t_{R_m}} F_C\left(\min\left\{\frac{x}{(t-\tau)^{\alpha}}, \frac{x_{th}}{\left(t_{R_m}-\tau\right)^{\alpha}}\right\}\right) f_{W_i}(\tau)\,d\tau}{\int_0^{t_{R_m}} F_C\left(\frac{x_{th}}{\left(t_{R_m}-\tau\right)^{\alpha}}\right) f_{W_i}(\tau)\,d\tau} \tag{19}$$

The cdf $F_{X_{(B)}}(x; t)$ is given by,

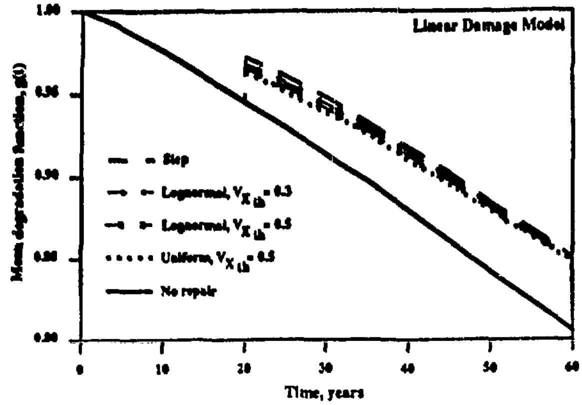

Fig. 4 Effect of several detectability functions on mean degradation function of inspection

$$F_{X_{(B)}}(x; t) = F_X\left(x; t_{R_m}, t\right). \tag{20}$$

34. In general, the detectability function, $d(x)$, is not a step function but rather a non-decreasing function of damage intensity (see Fig. 3). Procedures for dealing with this more general detectability function and for partial inspections have been developed and described elsewhere (refs 27-28). 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 $d(x)$.

35. Effect of inspection/repair operations on reliability. In order to study a simple case, assume that:

(a) The initiation of damages is described by a stationary Poisson process with $\nu = 5$/yr.

(b) Damage grows linearly as a function of time as described by Eqn. (7) with $\alpha = 1$; moderate reinforcement corrosion is typified by such growth (ref. 27).

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

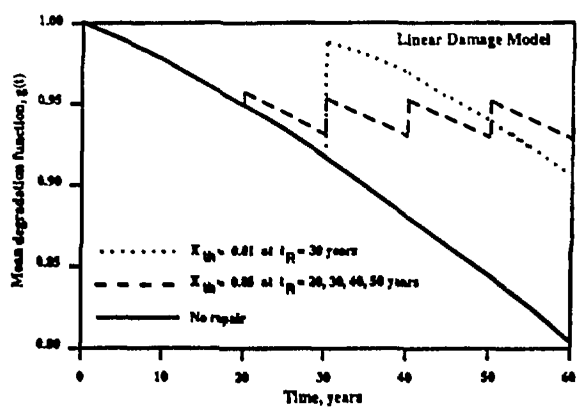

Fig. 5 Effect of multiple inspections/repairs on mean degradation function

36. The effect on the mean degradation function of inspection/repair described by several detectability functions is illustrated in Fig. 4. The first detectability function considered is a step detectability 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 $t_{R_m} = 20$ years. The mean degradation function decreases as $V_{X_{th}}$ increases (which would result in lower reliability).

37. The effect of multiple inspections/repairs on the mean degradation function is illustrated in Fig 5, considering the step detectability function. Inspections/repairs are carried out at 20, 30, 40 and 50 years with $x_{th_j} = x_{th} = 0.05$, or at 30 years with $x_{th} = 0.01$. With multiple inspection/repair, the mean degradation function can be kept within a narrow range during the service life of a structure; the width and the location of the range can be controlled by the frequency and the threshold value of the inspections.

38. Failure probabilities for a service life of 60 years evaluated with the degradation functions illustrated in Fig. 5 are presented in Fig. 6 for a reinforced concrete component designed for flexure using the design load combination for dead plus live load in ACI Standard 318 (ref. 29),

$$0.9R_n = 1.4D_n + 1.7L_n \tag{21}$$

in which R_n is the nominal strength, and D_n and L_n are the nominal dead and live loads, respectively The probabilistic models of load intensity used in this illustration are summarized in Table 1 (refs 25-27). It is assumed that the initial strength has a lognormal distribution with $\mu_R = 1.15R_n$ and coefficient of variation (c.o.v.) $V_R = 0.15$, and that $D_n = L_n$. The slope of $F(t)$ changes at the time of repair. The combined effect of multiple repairs with $x_{th} = 0.05$ leads to a failure probability during 60 years that is comparable to that of the single more intensive repair.

39. Results such as shown in Fig. 6 suggest the existence of an optimum inspection/maintenance strategy in which the failure probability of a 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 the expected loss due to failure, is minimized (ref. 27). In situations where the cost of failure dominates the expected total cost, the optimum policy involves inspection/repair at nearly uniform intervals during the service life of the structure.

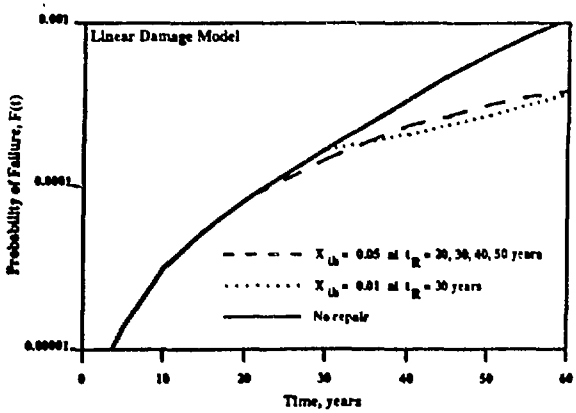

Fig. 6 Failure probability with multiple inspections/repairs

APPLICATION OF PROGRAM RESULTS

40. Potential regulatory applications of this research include: (1) improved predictions of long-term material and structural performance and available safety margins at future times, (2) establishment of limits on exposure to environmental stressors, (3) reduction in total reliance by licensing on inspection and surveillance through development of a methodology which will enable the integrity of structures to be assessed (either pre- or post-accident), and (4) improvements in damage inspection methodology through potential incorporation of results into national standards which could be referenced by standard review plans.

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