STRUCTURAL AGING PROGRAM STATUS REPORT

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ABSTRACT

Research is being conducted at the Oak Ridge National Laboratory (ORNL) under U.S. Nuclear Regulatory Commission (USNRC) sponsorship to address aging management of safety-related concrete structures. Documentation is being prepared to provide the USNRC with potential structural safety issues and acceptance criteria for use in continued service evaluations of nuclear power plants. Program accomplishments have included development of the Structural Materials Information Center containing data and information on the time variation of 144 material properties under the influence of pertinent environmental stressors or aging factors, performance assessments of reinforced concrete structures in several United Kingdom nuclear power facilities, evaluation of European and North American repair practices for concrete, an evaluation of factors affecting the corrosion of metals embedded in concrete, and application of the time-dependent reliability methodology to reinforced concrete flexure and shear structural elements to investigate the role of in-service inspection and repair on their probability of failure.

1. INTRODUCTION

By the end of this decade, 63 of the 111 commercial nuclear power plants in the United States will be more than 20 years old, with some nearing the end of their 40-year operating license. Faced with the prospect of having to replace the lost generating capacity from other sources and the potential for substantial shutdown and decommissioning costs, many utilities are expected to seek extensions to their plant operating licenses. A major concern in evaluating such applications is ensuring that the capacity of the safety-related systems to mitigate extreme events has not deteriorated unacceptably due to either aging or environmental stressor effects during their previous service history. Although major mechanical and electrical equipment items in a plant could be replaced, if necessary, replacement of the containment and many other safety-related concrete structural components would be economically

*Research sponsored by the Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission under interagency Agreement 1886-8084-5B with the U.S. Department of Energy under Contract DE-AC05-84OR21400 with Martin Marietta Energy Systems, Inc.

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unfeasible. Approval for service life extension must be supported by evidence that these structures will continue to be capable of withstanding potential future extreme events.

2. BACKGROUND

In general, the performance of nuclear power plant concrete structures has been good. However, there have been some instances where the capacity of the containment and other safety-related structures to meet future functional and performance requirements has been challenged. Degradation mechanisms that can potentially impact the performance of nuclear power plant reinforced concrete structures include corrosion of steel reinforcing systems, chemical attack, alkali-aggregate reactions, sulfate attack, frost attack, leaching, salt crystallization, and microbiological attack. Some of the aging concerns identified to date include inaccessibility of reinforced concrete basemat for inspection to detect potential degradation resulting from mechanisms such as leaching or sulfate attack, corrosion of steel reinforcement contained in water-intake structures, and corrosion of embedded portion of steel pressure boundary (liner) due to a breakdown of the seal at the concrete floor-to-liner interface. Where degradation incidences have occurred, they have generally done so early in the life of the particular structure and have been corrected. Causes were primarily related to improper material selection, construction/design deficiencies, or environmental effects. Examples of some of the more serious instances are described elsewhere [1-4].

3. STRUCTURAL AGING PROGRAM

Incidences of structural degradation related to the concrete components in nuclear power plants indicate a potential need for improved surveillance, inspection/testing, and maintenance to enhance the technical bases for assurances of continued safe operation. The Structural Aging (SAG) Program was initiated in 1988 and has the overall objective of preparing documentation that will provide USNRC license reviewers with (1) identification and evaluation of the potential 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, structural assessment or remedial measures programs; and (4) methodologies to perform current assessments and reliability-based life predictions of safety-related concrete structures. To meet this objective, SAG Program activities are conducted under four task areas: (1) program management, (2) materials property data base, (3) structural component assessment/repair technologies, and (4) quantitative methodology for continued service determinations.

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3.1 PROGRAM MANAGEMENT

3.1.1 Objective and Scope

The overall objective of the program management task is to effectively manage the technical tasks undertaken to address priority structural safety issues related to nuclear power plant continued service assessments. Management duties include planning, integrating, monitoring, reporting, and technology transfer. A key part of the management function is integration of the technical objectives and efforts of program participants. In addition, efforts are continuously made to maintain effective technology transfer and to maintain liaison with peer groups, technical committees, and programs in foreign countries.

3.1.2 Summary of Recent Accomplishments

Recent activities under this task have included administration of four subcontracts, and preparation of a technical progress report [5] and two foreign trip reports [6,7]. Program overview presentations were made at the Workshop on Concrete Performance and Modeling of Low-Level and Radioactive Waste Disposal [8], the National Institute of Standards and Technology Symposium on Integrated Knowledge Systems for High-Performance Construction Materials and Systems [9], British Nuclear Energy Society-sponsored International Conference on Thermal Reactor Safety Assessment [10], Nuclear Electric [11], and Electricité de France [12]. Program personnel participated in technical committees of the American Concrete Institute (Service Life Prediction, Concrete Materials Property Data Base, and Radioactive and Hazardous Waste Management) and American Society of Mechanical Engineers (Section XI Working Group on Concrete Pressure Components, Subgroup on Containments, and Special Working Group on Plant Life Extension). In addition, program personnel participated in an International Atomic Energy Agency Coordinated Research Program on Aging of Concrete Containment Buildings, and in development of an International Union of Testing and Research Laboratories for Materials and Structures Technical Committee, Methodology for Life Prediction of Concrete Structures in Nuclear Power Plants.

3.2 MATERIALS PROPERTY DATA BASE

3.2.1 Objective and Scope

The objective of the materials property data base task is to develop 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. 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. Primary activities under this task have involved the development of the Structural Materials Information Center and assemblage of materials property data. In addition, survey data and a durability assessment review of several reinforced concrete structures contained in several nuclear power stations located in England were assembled.

3.2.2 Summary of Recent Accomplishments

Structural Materials Information Center. Initial development of the Structural Materials Information Center (SMIC) has been completed [13]. Contained in this report are detailed descriptions of the Structural Materials Handbook and Structural Materials Electronic Data Base that comprise the SMIC. In addition, a reassessment of software tools for building customized data bases was performed.

The Structural Materials Handbook is an expandable, four volume, hard-copy reference document containing complete sets of data and information for each material. Volume 1 contains performance and analysis information (i.e., mechanical, physical, and other properties) useful for structural assessments and safety margins evaluations. Volume 2 provides the data used to develop the performance curves in Volume 1. Volume 3 contains material data sheets (e.g., constituent materials, general information, and material composition). Volume 4 contains appendices describing the handbook organization and revision procedures.

The Structural Materials Electronic Data Base is an electronically-accessible version of the handbook that provides an efficient means for searching the data base files. It has been developed on an IBM-compatible personal computer and employs two software programs - Mat.DB [14] and EnPlot [15]. Mat.DB is a menu-driven program for data base management that was developed for metallic materials. It employs window overlays to access data searching and editing features. Textual, tabular, and graphical information and data can be maintained, searched, and displayed. EnPlot is a program that incorporates pop-up menus for creating and editing engineering graphs. This software includes curve-fitting and scale-conversion features for preparing engineering graphs and utility features for generating output files. The engineering graphs can be entered directly into the Mat.DB data base files. A new version of Mat.DB (Version 2.0) based on Microsoft Windows [16] is being developed by ASM International (Materials Park, Ohio).

Based on experience gained during development of SMIC, advances in personal computer hardware capabilities, and corresponding developments in software tools for building customized data bases, a reassessment of candidate systems was conducted [17]. It was concluded that custom software provides enough flexibility to meet SMIC requirements for a data base management system (Table 3.6 of Ref. 5) and also permits entry of existing data and information files. Existing object-oriented relational data base software provides a foundation for a new data base management system that could be designed and built locally.

Data Collection. In parallel with efforts to develop the SMIC, activities have been conducted to establish materials property data for input into the SMIC. Two primary approaches have been utilized - open-literature information and testing of prototypical samples. To date, 132 material data bases have been developed based on open-literature sources (i.e., 116 concrete, 12 metallic reinforcement, 1 prestressing tendon, 2 structural steel, and 1 rubber). Twelve data bases have been prepared based on testing of prototypical samples associated with nuclear power plant facilities. Reference [18] presents summary descriptions of the material property data base files contained in SMIC.

Material Behavior Modeling. Activities related to an evaluation of models for predicting performance of concrete materials, collection of survey data and durability assessments of reinforced concrete structures contained in United Kingdom (UK) nuclear power stations, and evaluations of the performance of post-tensioning systems in UK prestressed concrete reactor vessels were described previously [19].

3.3 STRUCTURAL COMPONENT ASSESSMENT/REPAIR TECHNOLOGY

3.3.1 Objective and Scope

The objectives of this task are to: (1) develop a systematic methodology that can be used to make quantitative assessments of the presence, magnitude, and significance of any environmental stressors or aging factors that adversely impact the durability of safety-related concrete structures in nuclear power plants; and (2) provide recommended in-service inspection or sampling procedures that 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 that have experienced an unacceptable degree of deterioration, and the identification and evaluation of techniques for mitigation of any environmental stressors or aging factors that may act on critical concrete components.

Assessment of the ability of concrete components to meet their functional and performance characteristics is an important consideration in continuing the service life of nuclear facilities. Given the complex nature of the various environmental stressors and aging factors that potentially can exert deteriorating influences on the concrete components, a systems approach is probably best in addressing an evaluation of a structure for continued service. Basic components of such an approach would encompass the development of (1) a classification scheme for structures, elements, and deterioration causes and effects; (2) a methodology for conducting a quantitative assessment of the presence of active deteriorating influences; and (3) remedial measure considerations to reestablish the capability of degraded structures or components to meet potential future requirements, such as a loss-of-coolant accident (LOCA).

LWR Critical Concrete Component Classification. An aging assessment methodology for identifying structural components of most importance to aging and the degradation factors that can potentially impact the performance of these components was described previously [19].

NDE Sampling/Inspection Technology. Detection of age- or environmental stressor-related degradation, as well as its magnitude and rate of occurrence, is a key factor in maintaining the readiness of safety-related concrete components to continue their functions in the unlikely event of a severe accident. Basic activities under this subtask have addressed evaluation of destructive and nondestructive testing techniques, development of statistical data for selected nondestructive testing techniques, identification of potential aging concerns associated with post-tensioning systems used in prestressed concrete containments, and preparation of guidelines for in-service inspection of nuclear power plant concrete structures [19].

The ability of a prestressed concrete nuclear power plant containment to withstand the loadings that could develop as a result of a loss-of-coolant accident depends on the continued integrity of the prestressing tendons. In the U.S., the condition and functional capability of unbonded post-tensioning systems must be periodically assessed. This is accomplished, in part, through an in-service inspection program that must be developed and implemented for each containment. Requirements for containment tendon surveillance programs are provided in documents such as Regulatory Guide 1.35, Regulatory Guide 1.35.1, ASME Section XI Subsection IWL, and the U.S. Standard Technical Specification for Tendon Surveillance. Although the overall performance of the post-tensioning systems has been very good, there have been several instances of degradation. Examples include voids under tendon bearing plates resulting from improper concrete placement, cracking of anchorheads due to stress-corrosion cracking or embrittlement, containment dome delaminations due to low quality coarse aggregate material and absence of radial reinforcement or unbalanced prestressing forces, and low prestressing forces. A report is being prepared in which potential structural issues related to aging of post-tensioning systems in nuclear power plant containments are discussed. An overview of current requirements associated with in-service inspection of the post-tensioning systems will be used as the basis for development of a life management program for these systems. Potential aging- and environmental-stressor related items that can impact the performance of these systems are being identified (e.g., corrosion; loss of prestressing force due to relaxation, concrete creep, concrete shrinkage; etc.). The effectiveness of current life management programs in identifying problem areas is being assessed and recommendations are being prepared on how post-tensioning system-aging issues can be addressed in the future. For example, can post-tensioning tendons with prestressing force levels approaching the lower bound of acceptable performance merely be retensioned? What are the long-term effects on the mechanical performance of the post-tensioning system of being under

load? That is, does the tendon ultimate tensile strength and elongation capacity decrease with age under load? This activity is scheduled for completion in December 1994.

In-service inspection requirements are imposed on nuclear plants through documents such as the following: 10CFR50, NRC Regulatory Guides, Plant Technical Specifications, Inspection and Enforcement Bulletins, NRC letters, and the American Society of Mechanical Engineers Boiler and Pressure Vessel Code [20]. However, because each nuclear plant has unique construction permit and operating license issue dates, each plant could potentially have a different set of minimum in-service inspection requirements. Therefore, to simplify continued service evaluations, it would be advantageous to have a standardized in-service inspection program that could be used to identify and also to quantify any deteriorating influences. The availability of such a standardized inspection program would also help ensure that a uniform set of evaluation criteria is applied to each plant. Limited information on criteria, inspection, and testing requirements for development of such a procedure is available (e.g., Refs. [21-22]). However, the application requirements presented in these documents to nuclear safety-related concrete structures requires evaluation with respect to items such as accessibility, service history, functional requirements, construction materials, etc. A report is being prepared that has the overall objective of providing a suggested in-service inspection approach for reinforced and prestressed concrete structures in nuclear power plants. Criteria are being established to assess the current structural reliability of the safety-related concrete structures and to develop data for use in assessments of future performance. Specific activities include (1) a review and assessment of current NRC and industry-related in-service inspection requirements for reinforced concrete structures; (2) an evaluation of the applicability of available information on criteria, inspection, and testing requirements for general civil engineering concrete structures provided through organizations such as the American Concrete Institute and American Society for Testing and Materials; and (3) development of a structural component condition assessment methodology that establishes criteria for relating damage state and environmental exposure in terms of a "three tiered hierarchy" (e.g., acceptance "as-is," acceptance after review, and acceptance after additional evaluations). Visual acceptance criteria, per this hierarchy, are being developed to parallel the effort of American Concrete Institute Committee 349 [23]. Basic criteria for acceptance without further evaluation and acceptance after review based on visual inspections have been developed for (1) exposed concrete surfaces; (2) lined concrete surfaces; (3) areas around embedments in concrete; (4) joints, coatings, and non-structural components; and (5) prestressing steel systems. Any condition outside the criteria for these two conditions is considered unacceptable and requires additional nondestructive testing, destructive testing, analytical assessment, or a combination of the three. Degradation-based acceptance criteria are being established for concrete cracking, loss of concrete section, conventional and prestressing steel corrosion, and loss of

prestressing force. When completed in December 1994, the structural component condition assessment methodology will provide guidance for dispositioning of conditions or findings from in-service inspections.

Remedial/Preventative Measures Considerations. The life of reinforced concrete components in nuclear power plants is expected to be greater than any likely period for which the plant would operate, provided neither environmental factors, applied load, nor a combination of load and environmental factors compromise its integrity [24]. When concrete structures have been fabricated with close attention to the details related to fabrication of good quality concrete, the concrete should exhibit extended durability. A breakdown in any of these factors or occurrence of an extreme environmental stressor or adverse aging factor can result in significant distress requiring preventative maintenance or structural repair. Basic activities under this subtask have included reviews of European and North American repair practices for degraded reinforced concrete, and an assessment of corrosion of metals embedded in concrete [19].

3.4 QUANTITATIVE METHODOLOGY FOR CONTINUED SERVICE DETERMINATIONS

3.4.1 Objective and Scope

The goal of this task is to develop a methodology that will facilitate quantitative assessments of current and future structural reliability and performance of concrete structures in nuclear plants. Specific objectives associated with accomplishing this goal include (1) identification of models to evaluate changes in strength of concrete structures over time in terms of initial conditions, service load history, and aggressive environmental factors; and (2) formulation of a methodology to predict structural reliability of existing concrete structures during future operating periods from a knowledge of initial conditions of the structure, service history, aging, nondestructive condition assessment techniques, and inspection-maintenance strategies.

Prior activities have developed a probability-based methodology to estimate strength degradation of a component and to evaluate the effect of periodic maintenance from a reliability point of view [25]. This methodology has been extended to consider cases where several defects or zones of damage may contribute to a reduction in strength of a structural member.

Time-Dependent Reliability Analysis. Structural loads, 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 regulatory level.

The strength, $R(t)$, of the component and the applied loads, $S(t)$, both are random (or stochastic) functions of time. At any time, t , the margin of safety, $M(t)$, is

$$M(t) = R(t) - S(t). \quad (1)$$

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

$$P_f(t) = P[M(t) < 0] = \int_0^\infty F_R(x) f_S(x) dx. \quad (2)$$

in which $F_R(x)$ and $f_S(x)$ are the probability distribution function of R and density function of S . Equation 2 provides one quantitative measure of structural reliability and performance, provided that P_f can be estimated and validated. The numerical evaluation of Eqn. 2 and the development of supporting statistical data remain research challenges. However, significant progress has been made in this regard during the past several years.

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. 2. Indeed, it is difficult to use reliability analysis for engineering decision analysis without having some time period (say, 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 $S_1, S_2, \dots, S_n$ occur at times $t_1, t_2, \dots, t_n$ during $(0,t)$, the reliability function becomes,

$$L(0,t) = P[R(t_1) > S_1, \dots, R(t_n) > S_n]. \quad (3)$$

If the load process is continuous rather than discrete, there is an analogous but more complex expression.

The conditional probability of failure within time interval $(t, t+dt)$, given that the component has survived during $(0,t)$, is defined by the hazard function:

$$h(t) = -d \ln L(0,t)/dt. \quad (4)$$

Solving for $L(0,t)$ yields,

$$L(0,t) = \exp \left[- \int_0^t h(x) dx \right]. \quad (5)$$

The hazard function is especially useful in analyzing structural failures due to aging or deterioration. For example, the probability that time to structural failure, T_f , occurs prior to a future maintenance operation scheduled at $t+\Delta t$, given that the structure has survived to t , can be evaluated as,

$$P[T_f \leq t + \Delta t | T_f > t] = 1 - \exp \left[- \int_t^{t+\Delta t} h(x) dx \right]. \quad (6)$$

The hazard function for pure chance failures (case 1 in next section) is constant. When structural aging occurs and strength deteriorates, $h(t)$ characteristically increases with time. In-service inspection and maintenance impact the hazard function, causing it to change discontinuously at the time an inspection is performed. The main difference between time-dependent reliability of undegrading and degrading structural components can be characterized by their hazard functions. Much of the challenge in structural reliability analysis involving deteriorating structures lies in relating the hazard function to specific degradation mechanisms, such as corrosion.

It is assumed that significant structural loads can be modeled as a sequence of load pulses, the occurrence of which is described by a Poisson process with mean rate of occurrence λ , random intensity S_j , and duration τ . Such a simple load process has been shown to be an effective model for extreme loads on structures, since normal service loads challenge the structure to only a small fraction of its strength. With this assumption, the reliability function becomes

$$L(0, t) = \int_0^\infty \exp \left(-\lambda t \left[1 - t^{-1} \int_0^t F_S(rg) dt \right] \right) f_R(r) dr, \quad (7)$$

in which $f_R(r)$ is the probability density function of initial strength, $R(0)$, and $g(t)$ equals the mean of $R(t)/R(0)$, a function describing the degradation of strength in time (see Fig. 1). The limit state probability, or probability of failure during $(0, t)$, can be determined as $F(t) = 1 - L(0, t)$; $F(t)$ is not the same as $P_f(t)$ in Eqn. 2.

Service Life Predictions for Reinforced Concrete Slab. Time-dependent reliability concepts are illustrated with a simple example of a concrete slab drawn from recent research on aging of concrete structures in nuclear plants [26-27]. This slab was designed using the requirements for flexure strength found in ACI Standard 318 [28]:

$$0.9 R_n = 1.4 D_n + 1.7 L_n. \quad (8)$$

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. The strength of the slab changes in time, initially increasing as the concrete matures and then decreasing due to (unspecified) environmental attack. This situation is illustrated conceptually by the sample functions $r(t)$ and $s(t)$ for strength and load in Fig. 1. The behavior of the resistance over time must be obtained from mathematical models describing the degradation mechanism(s) present.

Fig. 1 Mean degradation functions of one-way slab.

Figure 2 presents a comparison of limit state probabilities for intervals $(0, t)$ for t ranging up to 60 years. Three cases are presented (see Fig. 1): (1) no degradation in strength, i.e., $R(t) = R(0)$, a random variable (this case is analogous to what has been done in probability-based code work to date [29]); (2) $R(t)$ strength initially increasing with concrete maturity and then decreasing; and (3) $R(t)$ strength decreasing linearly over time to 90% of its initial strength at 40 years. The statistics used in the illustrations that follow are summarized in Table 1. The basis for these statistics is given elsewhere [30]. Neglecting strength degradation entirely in a time-dependent reliability assessment can be quite unconservative, depending on the nature of the time-dependent behavior.

Service Life Predictions for Reinforced Concrete Shear Wall. As shown above, the failure probability of a structural component or system under stationary random loading can be evaluated as a function of time if the strength degradation and the probabilistic characteristics of the initial strength are known. In previous papers, a probability-based method to estimate the strength degradation of a component and to evaluate the effect of periodic maintenance from a reliability point of view was provided [26,27]. In the method developed, it was assumed that strength degradation at any section is caused by one randomly occurring defect of random intensity. Such a model is reasonable when the degradation is such that at most one defect or

Fig. 2 Failure probability of one-way slab.

Table 1 Statistical properties of strength and load.

Parameter	Rate of Occurrence	Duration	Main	C.O.V.	Pd _f
Flexure Strength	—	—	1.12M _n	0.14	Lognormal
Shear Strength	—	—	1.7V _n	0.18	Lognormal
Dead Load	—	—	1.0D _n	0.07	Normal
Live Load	0.5/yr	3 mo.	0.4L _n	0.50	Type I
Earthquake Load	0.05/yr	30 sec.	0.08E _{ss} e	0.85	Type II

zone of damage is likely to occur within a given cross section. The strength degradation of a reinforced concrete beam or column due to corrosion of reinforcement can be estimated by such modeling. However, there are cases where several defects or zones of damage may contribute in reducing strength. For example, the strength of a reinforced concrete wall in flexure and/or shear might degrade due to the combined effects of expansive aggregate reactions at several points along a given cross section of the wall. The

evaluation of the (random) residual strength of the wall requires that the cumulative effect of defects in a cross section be considered. Recent research has provided a method whereby the impact of randomly occurring multiple defects on structural capacity can be considered [33]. Some results are summarized in the following.

The wall considered is a low-rise wall with a height-to-width ratio equal one, and is subjected to vertical load, D , which is uniformly distributed on the top of the wall, and in-plane lateral load, V , which is concentrated at the top of the wall. The shear strength of concrete walls can be estimated from empirical models [28,34]. These models are not sufficient to analyze the strength of deteriorating low-rise shear walls. Although finite-element analysis is versatile and able to provide detailed information on the shear resistance mechanisms, it requires lengthy computational effort, especially when adapted to reliability analysis. A recent theoretical approach for evaluating shear strength of reinforced concrete components [35-37] determines the ultimate shear strength as the sum of the forces sustained by a truss mechanism, V_t , and by an arch mechanism, V_a . It is assumed that the wall fails if all the reinforcing bars yield in tension and the concrete arch crushes in compression. According to the lower bound theorem of plasticity [38], this approach provides a conservative estimate of the shear strength. These models have been modified for the reliability analysis of a degrading low-rise concrete shear wall [33]. Figure 3 shows that the strength predicted by this method compares well to experimental tests of low-rise shear walls.

Wall in Shear

A wall subjected to expansive aggregate reaction or chemical attack suffers a loss of concrete section. If the wall is not heavily reinforced in the transverse direction, the contribution of the truss mechanism is small. Thus, it can be assumed that only the strength of the arch mechanism decreases due to the loss of concrete section while the strength attributed to the truss mechanism is independent of the degradation. If the wall is reinforced in the longitudinal direction, the vertical reaction is sustained by the longitudinal reinforcement, and degradation of concrete outside the concrete strut in the arch mechanism can be neglected. Assume that the stress in the concrete strut is uniform. Then the degradation function of the shear wall can be given by

$$\begin{aligned} g(t) &= \frac{V_t + V_a(t)}{V_{u0}} \\ &= \frac{V_t + G_a(t) V_a(0)}{V_{u0}} \end{aligned} \tag{9}$$

in which V_{u0} is the initial shear strength of the wall, $V_a(t)$ is the shear strength of the arch mechanism at time t , and $G_a(t)$ is the degradation function of the shear strength of the arch mechanism.

Fig. 3 Comparison of measured and calculated shear strength.

Wall in Flexure and Compression

The ultimate flexural capacity of a cross section is expressed as

$$M_u = T_s \left(\frac{b}{2} - d_c \right) + C_c \left(\frac{b}{2} - k_2 c_u \right) + C_s \left(\frac{b}{2} - d_c \right) \quad (10)$$

in which T_s and C_s are the total force transferred to reinforcement in the tension and compression zone, respectively, d_c is the concrete cover, c_u is the distance from the compressive face to the neutral axis, and $k_2 c_u$ locates the compressive resultant, C_c .

For illustration, assume that

- The wall is subjected to time-invariant dead load, D , which is uniformly distributed on the wall, and intermittent lateral load V , which is concentrated at the top of the wall and may act either in-plane or out-of-plane.

- The wall is designed for in-plane shear based on the current design requirement [39].

$$0.9R_n = E_{ss} \quad (11)$$

in which R_n is the nominal shear strength and E_{ss} is the structural action due to safe-shutdown earthquake. The statistical characteristics of the shear strength and the earthquake load are shown in Table 1. It is assumed that $E_{ss} = 3D = 3.21MN$.

- The mean initiation rate of local damage per unit surface area, v_u , due to expansive aggregate reaction is time invariant and is $0.1/m^2/year$.
- The defect intensity is modeled as,

$$Y(t) = C(t - T_I)^2 \quad (12)$$

in which C is a time-invariant random variable described by a lognormal distribution with mean value, m_C , of $2.22 \times 10^{-6}/year$ and coefficient of variation, V_C , of 0.5. This value results in an average defect size that is large enough after several years following its initiation to be found by visual inspection.

- The 28-day specified compressive strength of concrete equals 27.6 MPa. The corresponding mean compressive strength at 28 days is 28.7 MPa [40]. The specified yield strength of the reinforcement is 414 MPa and the mean is 465 MPa.
- Compressive strength of the concrete increases during the first 10 years but does not change thereafter. The mean compressive strength (in units of MPa) at time t is evaluated by [41]

$$E[f_c(t)] = \begin{cases} 15.51 + 3.95 \ln t, & t < 10 \text{ years} \\ 47.91, & t \geq 10 \text{ years} \end{cases} \quad (13)$$

in which t is in days. The concrete section area decreases with time as damage accumulates. Other engineering properties of the wall are assumed to be time-invariant.

The mean degradation in shear strength of the wall in which expansive aggregate reactions occur in the concrete is illustrated in Fig. 4, assuming $v_u = 0.1/m^2/year$. The mean degradation in shear strength evaluated ignoring the cumulative effect of multiple defects in a section on the strength degradation of the wall is also illustrated in the figure. The gain in shear strength due to the continuous hydration of concrete more than compensates for

the strength degradation due to the loss of section area up to about 50 years. Ignoring the cumulative effect of defects provides an overly optimistic degradation function.

The failure probabilities and the hazard functions associated with the strength degradation illustrated in Fig. 4 are presented in Figs. 5 and 6, respectively. The increase in failure probability due to the strength degradation is small because of the large variability in earthquake load intensity [42]. However, the hazard function increases rapidly after about 50 years when the cumulative effect of defects is considered.

The mean degradation in flexure/compression strength of the wall is more sensitive to the loss of the outer part of the cross section area than is the shear strength, as shown in Fig. 7. Since the loss of the outer part of the wall leads to a reduction in the internal moment arm, the flexural strength degrades more rapidly than the shear strength, which decreases linearly as a function of the loss of cross-section area. Thus, if the governing limit state of the wall is flexure, special attention should be given to the potential for degradation when performing a condition assessment.

Fig. 4 Mean degradation function of wall in shear without repair.

Fig. 5 Failure probability of wall in shear without repair.

Fig. 6 Hazard function of wall in shear without repair.

Condition Assessment and In-Service Inspection. Forecasts of reliability of the type illustrated in Fig. 2 enable the analyst to determine the time period beyond which the desired reliability of the structure cannot be ensured. At such a time, the structure should be inspected. Intervals of inspection and maintenance that may be required as a condition for continued operation can be determined from the time-dependent reliability analysis. In-service inspection and maintenance are a routine part of managing aging and deterioration in many engineered facilities; work already has been initiated to develop policies for offshore platforms [43] and aircraft [44] using probabilistic methods.

Fig. 7 Mean degradation function of wall in flexure/compression.

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. The information gained from 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,

$$f_R(r|B) = P[r < R \leq r + dr, B] / P[B] = c K(r) f_R(r), \quad (14)$$

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 (e.g., Fig. 6) to be discontinuous.

Uncertainties in methods of in-service inspection/repair affect the density $f_R(r|B)$. Using a combination of methods usually is more effective from a reliability point of view than using one method. When there are limited resources, it often is most effective to select a few safety-critical elements and concentrate on them [30,45]. 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 [42]. 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.

4. APPLICATION OF PROGRAM RESULTS

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 that 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 that could be referenced by standard review plans.

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