NUREG/CR-6424
ORNL/TM-13148

Report on Aging of Nuclear Power Plant Reinforced Concrete Structures

Prepared by
D. J. Naus, C. B. Oland, ORNL
B. R. Ellingwood, JHU

Oak Ridge National Laboratory

The Johns Hopkins University

Prepared for
U.S. Nuclear Regulatory Commission

AVAILABILITY NOTICE

Availability of Reference Materials Cited in NRC Publications

Most documents cited in NRC publications will be available from one of the following sources:

1. The NRC Public Document Room, 2120 L Street, NW., Lower Level, Washington, DC 20555-0001

2. The Superintendent of Documents, U.S. Government Printing Office, P. O. Box 37082, Washington, DC 20402-9328

3. The National Technical Information Service, Springfield, VA 22161-0002

Although the listing that follows represents the majority of documents cited in NRC publications, it is not intended to be exhaustive.

Referenced documents available for inspection and copying for a fee from the NRC Public Document Room include NRC correspondence and internal NRC memoranda; NRC bulletins, circulars, information notices, inspection and investigation notices; licensee event reports; vendor reports and correspondence; Commission papers; and applicant and licensee documents and correspondence.

The following documents in the NUREG series are available for purchase from the Government Printing Office: formal NRC staff and contractor reports, NRC-sponsored conference proceedings, international agreement reports, grantee reports, and NRC booklets and brochures. Also available are regulatory guides, NRC regulations in the *Code of Federal Regulations*, and *Nuclear Regulatory Commission Issuances*.

Documents available from the National Technical Information Service include NUREG-series reports and technical reports prepared by other Federal agencies and reports prepared by the Atomic Energy Commission, forerunner agency to the Nuclear Regulatory Commission.

Documents available from public and special technical libraries include all open literature items, such as books, journal articles, and transactions. *Federal Register* notices, Federal and State legislation, and congressional reports can usually be obtained from these libraries.

Documents such as theses, dissertations, foreign reports and translations, and non-NRC conference proceedings are available for purchase from the organization sponsoring the publication cited.

Single copies of NRC draft reports are available free, to the extent of supply, upon written request to the Office of Administration, Distribution and Mail Services Section, U.S. Nuclear Regulatory Commission, Washington, DC 20555-0001.

Copies of industry codes and standards used in a substantive manner in the NRC regulatory process are maintained at the NRC Library, Two White Flint North, 11545 Rockville Pike, Rockville, MD 20852-2738, for use by the public. Codes and standards are usually copyrighted and may be purchased from the originating organization or, if they are American National Standards, from the American National Standards Institute, 1430 Broadway, New York, NY 10018-3308.

NUREG/CR-6424
BNL-NUREG-13148

Report on Aging of Nuclear Power Plant Reinforced Concrete Structures

Manuscript Completed: March 1996
Date Published: March 1996

Prepared by
D. J. Naus, C. B. Oland, ORNL
B. R. Ellingwood, JHU

Oak Ridge National Laboratory
Operated by Martin Marietta Energy Systems, Inc.

Oak Ridge National Laboratory
Oak Ridge, TN 37831

Subcontractor:
Department of Civil Engineering
The Johns Hopkins University
3400 North Charles Street
Baltimore, MD 21218-2699

H. L. Graves, NRC Project Manager

Prepared for
Division of Engineering Technology
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001
NRC Job Code B0845

ABSTRACT

The Structural Aging Program provides the United States Nuclear Regulatory Commission with potential structural safety issues and acceptance criteria for use in continued service assessments of nuclear power plant safety-related concrete structures. The program was organized under four task areas — Program Management, Materials Property Data Base, Structural Component Assessment/Repair Technology, and Quantitative Methodology for Continued Service Determinations. Under these tasks, over 90 papers and reports were prepared addressing pertinent aspects associated with aging management of nuclear power plant reinforced concrete structures. Contained in this report is a summary of program results in the form of information related to longevity of nuclear power plant reinforced concrete structures, a Structural Materials Information Center presenting data and information on the time variation of concrete materials under the influence of environmental stressors and aging factors, in-service inspection and condition assessments techniques, repair materials and methods, evaluation of nuclear power plant reinforced concrete structures, and a reliability-based methodology for current and future condition assessments. Recommendations for future activities are also provided.

TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd)

TABLE OF CONTENTS (cont'd)

TABLE OF CONTENTS (cont'd)

LIST OF FIGURES

LIST OF FIGURES (cont'd)

LIST OF FIGURES (cont'd)

LIST OF TABLES

LIST OF TABLES (cont'd)

LIST OF TABLES (cont'd)

ACKNOWLEDGEMENTS

The authors would like to acknowledge the continuing support and guidance throughout the program provided by the U.S. Nuclear Regulatory Commission Technical Monitors, Mr. Herman Graves, III and Mr. Gunter Arndt. Appreciation is also extended to the many organizations that contributed to the success of the program. Specific organizations that participated in the program through subcontracted activities included Construction Technologies Laboratories; Corrpro Companies; Howard Universities; Multiple Dynamics Corporation; National Institute of Standards and Technology; Sargent & Lundy Engineers; Taywood Engineering; The Johns Hopkins University; and Wiss, Janney, Elstner, Associates. Contributions were also made to the program by consultants that either served on a peer review board (T. Brown, J. Clifton, W. Gamble, C. Hookham, R. Ravindra, and J. Reed) or researched special topics (H. Hill and C. Hookham). Finally, special thanks are due Ms. Pam Abbott for typing the manuscript.

EXECUTIVE SUMMARY

The Structural Aging (SAG) Program addressed safety-related concrete structures in nuclear power plants and provided the United States Nuclear Regulatory Commission (USNRC) with the following: (1) identification and evaluation of the structural degradation processes; (2) issues to be addressed under nuclear power plant (NPP) 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 in use, or needed; and (4) quantitative methodologies for assessing current or future structural safety margins. In meeting these objectives activities were conducted under three technical task areas: (1) materials property data base, (2) structural component assessment and repair technologies, and (3) quantitative methodology for continued service determinations. Results obtained under these three tasks are contained in this report that consists of six chapters.

Chapter one provides background information pertaining to the importance of continuing the service of NPPs. License renewal activities sponsored by the U.S. utilities and the USNRC are summarized. Also included is the overall objective of the SAG Program, objectives of the three technical task areas, and information pertaining to subcontracted activities. A complete listing of reports and papers prepared under this program is contained in the Appendix.

Safety-related concrete structures and longevity considerations are addressed in the second chapter. Design criteria are summarized and typical safety-related concrete structures are identified as well as their materials of construction. Aging factors and environmental stressors that can impact the performance of these structures are described as well as service-life models for estimating future performance. An application is presented in which service life models are used to estimate onset of corrosion in NPP reinforced concrete structures. Details are provided on a structural materials information center consisting of a handbook and an electronic data base that contain data and information on the time variation of material properties under the influence of pertinent environmental stressors and aging factors. Information is provided on the performance of U.S. and United Kingdom reinforced concrete structures in nuclear power stations.

Chapter three addresses in-service inspection (ISI), condition assessment, and remedial measures. Current and emerging ISI techniques are identified, described, and recommendations provided relative to their use. Techniques for conducting condition assessments of general civil engineering structures and existing requirements for NPP reinforced concrete structures are summarized. Repair considerations, materials, and techniques are identified. Guidance is provided on repair options for various forms of concrete degradation (e.g., cracking and spalling). Remedial measures specifically addressing corrosion-damaged concrete are described.

Background for use in development of a methodology for inspection of NPP reinforced concrete structures is provided in the fourth chapter. Selection of structures for evaluation is described in terms of an aging assessment methodology, based in part on probabilistic risk assessments. Applications of commonly used inspection methods are noted (i.e., visual, nondestructive, destructive, and analytical). Considerations are presented for situations where accessibility is an issue. A visual-based inspection methodology and criteria for use in interpretation of results are developed. Inspection scheduling and personnel qualification requirements are provided. Development of a NPP inspection program for reinforced concrete structures is illustrated through an example.

A reliability-based methodology for current and future condition assessments is addressed in the fifth chapter. Modeling of degradation mechanisms, statistical data on loads and resistance for NPP concrete structures, and basic aspects of the time-dependent reliability analysis are described. Conceptual examples are presented for a reinforced concrete slab and low-rise shear wall to illustrate the time-dependent reliability analysis. The role of inspection and repair in maintaining reliability is discussed. Both single full inspection/repair and multiple full inspection/repair strategies are considered as well as partial inspection/repair strategies. Examples are used to illustrate the effect of an inspection/repair operation. Optimum inspection/repair strategies to minimize expected future costs of keeping the failure probability of the structure below an established target failure probability are developed and demonstrated through an example application.

Chapter 6 presents a summary of program results, general conclusions that can be derived from the program, and recommendations for additional activities.

1. INTRODUCTION

1.1 BACKGROUND

There are 109 nuclear power reactors presently licensed for commercial operation in the United States with 1 reactor still under construction and 5 reactors partially completed, but under a deferred construction schedule[1*] (see Appendix A). The Atomic Energy Act (AEA) of 1954 limits the duration of operating licenses for most of these reactors to a maximum of 40 years. Forty-nine of these reactors have been in commercial operation for 20 or more years. Expiration of operating licenses for these reactors will start to occur in the year 2000 when Big Rock Point's license expires. Through 2010 an additional 13 plants will also reach the end of their initial operating license period with a potential net loss of electrical generating capacity of approximately 9 GW. An additional 30 GW loss of net electrical generating capacity will occur between 2011 and 2015 when the initial licenses for an additional 37 plants are scheduled to expire. Under current economic, social, and political conditions in the United States, the prospects for early resumption of building of new nuclear plants to replace lost generating capacity are very limited.[2] In some areas of the country it may be too late because of the 10 to 15 years required to plan and build replacement power plants. Continuing the service of existing nuclear plants through a renewal of their initial operating licenses provides a timely and cost-effective solution to the problem of meeting future energy demand.

The 40 year term on the duration of an operating license that was provided in the AEA of 1954 apparently was based on various financial considerations (e.g., bond maturity) and was not based on safety or technical concerns. No technical information was presented to suggest that the nuclear power plants (NPPs) would become unsafe if they were to operate after 40 years.[3] In fact, the AEA permits the renewal of operating licenses. Paragraph 50.51 of Part 10 of the *Code of Federal Regulations*[4] implements this authority; however, no prior standards or procedures have been provided for preparing or evaluating license renewal applications.

1.2 U.S. UTILITY AND NUCLEAR REGULATORY COMMISSION (USNRC) AGING AND LICENSE RENEWAL ACTIVITIES

As a consequence of the significant economic and energy supply implications due to expiration of the operating licenses for existing NPPs, both the U.S. utilities and USNRC have conducted extensive research programs addressing aging management and its relation to license renewal.

1.2.1 U.S. Utilities

The nuclear utility industry has expressed considerable interest in operating nuclear power plants beyond their initial term of operation and has undertaken several initiatives in support of this. A Steering Committee on Nuclear Plant Life Extension (NUPLEX) was formed under the direction of the Nuclear Management and Resources Council (NUMARC) [now part of Nuclear Energy Institute (NEI)]. The Electric Power Research Institute (EPRI), in cooperation with the U.S. Department of Energy (DOE) and two utilities, sponsored research on life extension, including pilot studies on two nuclear plants, Surry-1[5] and Monticello.[6] This led to funding by DOE of two lead plants (Yankee Rowe and Monticello) to develop formal requests for renewal of their operating licenses. The technical aspect of the DOE program was to provide an initial

* References are collected and provided as Chap. 7 of the report

evaluation of the effects of aging on commercial nuclear power plants and establish the scope of the effort required to extend the operating lifetime of these plants beyond the initial 40 years of licensed operation. Since that time, Yankee Rowe has been permanently shut down, and although a license renewal application has been prepared for Monticello as a Boiling-Water Reactor Owner's Group document, submittal of the license renewal application has been delayed.

Ten industry reports sponsored by DOE and EPRI in support of the license renewal process have been developed under the auspices of NUMARC and submitted for USNRC staff review. The industry reports evaluate the age-related degradation effects on specific structures and components (e.g., Class 1 structures, pressurized-water reactor containments, and pressurized-water reactor internals), describe the bases for how existing programs required by various regulations address the aging concerns, and provide specific recommendations for corrective actions that should be implemented for specific components or structures not presently addressed by effective age-related management programs. These reports are intended to provide a partial demonstration of the viability and stability of the license renewal process by reaching agreement with the USNRC staff on technical issues related to light-water reactor systems, structures, and components for operation beyond the current licensing term. When approved by the USNRC, renewal applicants could reference the industry reports in the same manner that they can reference a topical report for other licensing actions. These reports were prepared prior to issuance of the revised License Renewal Rule discussed in the next section.

A number of activities addressing license renewal are being conducted by the various owners groups [e.g., Babcock & Wilcox (B&WOG), Boiling Water (BWROG), and Westinghouse (WOG)], and individual utilities [e.g., Baltimore Gas and Electric (BG&E), Virginia Power, and Duke Power].[7,8] The NEI recently has formed a license renewal working group (LRWG) consisting of three owner's groups (B&WOG, WOG, BWROG), two utilities (BG&E, Virginia Power), DOE, and EPRI to coordinate owner's group technical activities. The LRWG is conducting activities under two primary tasks: (1) new license renewal technical issues (e.g., generic technical issues, technical reports coordination, and definition of terms), and (2) new license renewal implementation guide (e.g., input coordination, lead industry effort, and technical direction). Also, a number of industry-sponsored workshops specifically addressing license renewal have taken place (e.g., Baltimore Gas and Electric Life-Cycle and EPRI/DOE License Renewal).

1.2.2 U.S. Nuclear Regulatory Commission (USNRC)

The USNRC program on the degradation of nuclear power plant systems, structures, and components (SSCs) due to aging began in the 1980's. This program has addressed the development of rules related to renewal of operating licenses and has conducted research in support of the various rulemaking activities.

1.2.2.1 Rulemaking activities

The USNRC staff in 1982, recognizing the potential impact of plant aging phenomena on the continued safe operation of NPPs, convened a workshop in Bethesda, Maryland. The objective of the workshop was to focus attention on how best to proceed to identify and resolve various technical issues relevant to plant aging. By 1986, age-related degradation had become a higher priority with the recognition that utilities were interested in continuing the service of their existing nuclear power plants past the initial licensing term. In May 1987, the Technical Review Group for Aging and Life Extension (TIRGALEX) produced a document defining technical safety and regulatory policy issues associated with plant aging. A primary conclusion of this group was that many aging phenomena are readily managed and do not pose major technical issues that would preclude continuing the service of NPPs past 40 years, provided that aging was properly managed through programs that maintained, surveyed, repaired, and replaced key SSCs.

In August 1988, the USNRC staff published an Advanced Notice of Rulemaking in the *Federal Register* (53 FR 32919) announcing the intention to prepare a proposed rule on license renewal. The draft proposed rule (10 CFR Part 54) was published for public review and comment in the *Federal Register* (Vol. 55, No. 137, July 17, 1990). Its intent was to provide the regulatory philosophy for license renewal through establishment of (1) the technical requirements that a license renewal applicant must satisfy, (2) the nature of information to be provided in a license renewal application, and (3) the application procedures. Two important principles underlying license renewal are that (1) the regulatory process is adequate to ensure that the licensing bases of all currently operating plants provide and maintain an acceptable level of safety so that operation will not be inimical to public health and safety or common defense and security, and (2) the plant-specific licensing basis must be maintained during the renewal term in the same manner and to the same extent as during the original licensing term. To satisfy these principles, the rule requires that (1) all license renewal applications compile the current licensing basis (CLB) for their plants, submit a summary list of documents that identify portions of the CLB relevant to the Integrated Plant Assessment (IPA), and maintain all CLB documents in an auditable and retrievable form; (2) age-related degradation of all SSCs important to license renewal be evaluated through IPA; and (3) existing or newly established effective programs be used to manage age-related degradation. The CLB is defined as all regulatory requirements and licensee commitments for the plant that are in effect at the time of the license renewal application and thus evolves with time and is plant specific. The IPA consists of a screening process to select SSCs important to license renewal based on their intended safety functions or contribution to challenging safety systems; an evaluation and demonstration of the effectiveness of the already ongoing licensee actions under existing regulatory requirements and plant-specific programs to address aging concerns; and implementation, as necessary, of supplemental programs to prevent or mitigate age-related degradation during the renewed license term. To amplify and support the proposed rule, written guidance was provided in the form of a draft regulatory guide *Standard Format and Content of Technical Information for Applications to Renew Nuclear Power Plant Operating Licenses* (DG 1009), and draft standard review plan *Standard Review Plan for the Review of License Renewal Applications for Nuclear Power Plants* (NUREG 1299), both published for public comment on December 4, 1990.[9]

In further action, the Commission amended its regulations on July 10, 1991, through addition of §50.65, "Requirements for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants (56 *Federal Register* 31324)," to be implemented by each licensee no later than July 10, 1996. The maintenance rule requires that power reactor licensees monitor the performance or condition of SSCs against licensee-established goals in a manner sufficient to provide reasonable assurance that these SSCs are capable of fulfilling their intended functions. It provides a basis for concluding that the effects of aging will be effectively managed during the license renewal period.

Subsequently, as a result of interactions with industry and a senior management review at the USNRC, the USNRC staff in 1993 provided its recommendations regarding key license renewal issues in two Commission policy papers – *Implementation of 10 CFR Part 54* (SECY-93-049), and *Requirements for Renewal of Operating Licenses for Nuclear Power Plants* (SECY-93-113). A staff requirements memorandum of June 28, 1993, noted that the Commission desired a predictable and stable regulatory process that defined the Commission's expectations for license renewal in a clear and unequivocal way. This would permit licensees to make decisions about license renewal without these decisions being influenced by the regulatory process that is perceived to be uncertain, unstable, or not clearly defined. The Commission directed the USNRC staff to convene a workshop to address alternative approaches for license renewal to take advantage of existing licensee activities and programs in demonstrating that aging will be addressed in an acceptable manner during the period of continued service. Also, it was directed that the USNRC staff should examine the extent to which greater reliance can be placed on the maintenance rule (10 CFR 50.65) as a basis for concluding that the effects of aging will be effectively managed during the continued service term. As a result, the USNRC staff on September 30, 1993, conducted a public workshop in Bethesda, Maryland, on license renewal. The results of this workshop and recommendations for revision of 10 CFR Part 54 were provided to the Commission in December 1993 in *License Renewal Workshop Results and Staff Proposals for Revision to 10 CFR Part 54* (SECY-93-331). The USNRC staff recommended that the Commission direct it to amend 10 CFR Part 54 to establish a more stable and predictable license renewal process. In their staff requirements memorandum of February 3, 1994, the Commission agreed with the USNRC staff's conceptual approach in SECY-93-331 and directed the staff to proceed with rulemaking to amend 10 CFR Part 54. The Commission noted that the amendment should establish a more stable and predictable license renewal process that identifies certain SSCs that require review to provide the necessary assurance that these SSCs will continue to perform their intended function for the period of continued service. The resulting proposed revisions developed by the USNRC staff were published for public review and comment in the *Federal Register* (Vol. 59; No. 174; September 9, 1994). The proposed rule, which has since been adopted, revises certain requirements contained in 10 CFR Part 54 and establishes a regulatory process that is simpler, more stable, and more predictable than the prior license renewal rule. Some of the changes implemented in the rule are that (1) the review focuses on the adverse effects of aging rather than identify all aging mechanisms; (2) the definitions of IPA and the IPA process are clarified to be consistent with the rule; (3) SSCs within the scope of license renewal are defined and important functions that must be maintained identified; (4) the IPA process is simplified through delineation of a methodology for determination of the need for an aging management review for structures and components, and only long-lived passive structures and components are subject to an aging management review for license renewal; (5) requirements for the CLB and conditions of renewed licenses would be changed to delete all references to age-related degradation unique to license renewal; and (6) the requirement for additional record and record-keeping requirements would be changed to be less prescriptive.

1.2.2.2 Research activities

In 1985, the Division of Engineering of the Office of Nuclear Regulatory Research issued the first comprehensive program plan for NPP aging research.[10] The Nuclear Plant Aging Research (NPAR) Program is aimed at developing the technical bases to ensure that the critical SSCs will provide adequate reliability as reactors age (i.e., understand and manage aging). The NPAR Program approach to understanding aging is to (1) define the component's boundary and all interfaces of interest, identify materials used in the design and fabrication of component parts; (2) identify applicable stressors and environments during the lifetime of the component, including those expected during and post design-basis events; (3) identify aging mechanisms and where they could be operative; and (4) determine age-related degradation effects and their significance on operability or performance (performing, when necessary, in situ testing and testing under controlled laboratory conditions; performing testing on naturally-aged components and samples of materials for correlation and validation). The NPAR Program approach to manage aging is to (1) identify detection and condition monitoring methods for evaluating age-related degradation effects, (2) identify and review ongoing programs with respect to their effectiveness in detecting and managing age-related degradation effects, and (3) develop appropriate recommendations to overcome deficiencies. This hardware-oriented program has studied 22 electrical and mechanical components, 13 safety-related systems, and 10 special topics (e.g., data needs and record keeping, risk evaluation of aging phenomena, and degradation modeling). Results obtained under this program have been summarized in approximately 160 technical reports and papers.[11] More detailed information on the overall program is provided elsewhere.[12]

One of the reports prepared under the NPAR Program was used as the basis to formulate the Structural Aging (SAG) Program.[13] The SAG Program was initiated in 1988 and had the overall objective of developing technical bases for addressing aging of the safety-related concrete structures and providing guidance for use in NPP evaluations for continued service. In meeting this objective, over 90 technical reports have been prepared and over 70 technical presentations have been made describing program results. These results can be used in the aging management of safety-related concrete structures to identify and evaluate (1) potential structural degradation processes; (2) issues to be addressed under NPP continued service reviews, as well as criteria, and their bases for resolution of these issues; (3) relevant in-service inspection (ISI) or structural assessment programs in use, or needed; and (4) quantitative methodologies for assessing current, or estimating future, structural safety margins.

1.3 OBJECTIVE AND APPLICATION OF RESULTS

The objective of this report is to provide a summary of results developed over the seven year duration of the SAG Program. These results provide background data and information that can be used by a reviewer or licensee to determine if the intent of the license renewal (10 CFR Part 54) and maintenance (10 CFR Part 50.65) rules are being met with respect to the safety-related concrete structures (i.e., necessary actions are being taken to ensure that these structures will continue to meet an acceptable level of safety, and age-related degradation is being adequately addressed to assure that there is not a loss of safety functions or an unacceptable reduction in safety margins). The results developed under this program have application to the required assessments related to selection of SCCs important to license renewal based on intended safety functions, evaluation and demonstration of the effectiveness of ongoing programs to address aging concerns, and implementation of programs to prevent or mitigate age-related degradation. Potential regulatory applications of this program 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 incorporation of results into national standards that could be referenced by Standard Review Plans.

1.4 APPROACH

The SAG Program consisted of a management task and three technical task areas (Fig. 1.1).

The objective of the program management task (Task S.1) was to effectively manage the technical tasks undertaken to address priority structural safety issues related to continuing the service of NPPs. A key function of the management task in addition to technology transfer was integration of the technical objectives and efforts of various program participants Individuals and organizations outside Oak Ridge National Laboratory (ORNL) that participated in the SAG Program are also presented in Fig. 1.1. The level 3 work breakdown structure for this task identifying subtasks and primary activities under each subtask is presented in Fig. 1.2.

The materials property data base task (Task S.2) developed a reference source containing data and information on the time variation of material properties under the influence of pertinent environmental stressors and aging factors. The data base has use in the prediction of potential long-term deterioration of critical structural components in NPPs and in establishing limits on hostile environmental exposure for these structures. The level 3 work breakdown structure for this task identifying subtasks and primary activities under each subtask is presented in Fig. 1.3.

The structural component assessment/repair technology task (Task S.3) identified a systematic methodology that can be used to (1) make quantitative assessments of the presence, magnitude, and significance of any environmental stressors or aging factors that could impact the durability of safety-related concrete structures in NPPs; and (2) provide recommended ISI or sampling procedures that can be utilized to develop the data required both for evaluating the current structural condition as well as trending the performance of these components for use in continued service assessments. Associated activities included identification and evaluation of techniques for mitigation of any environmental stressors or aging factors that may act on critical concrete components, and an assessment of techniques for repair, replacement, or retrofitting of concrete components that have experienced an unacceptable degree of deterioration. The level 3 work breakdown structure for this task identifying subtasks and primary activities under each subtask is presented in Fig. 1.4.

The quantitative methodology for continued service determinations task (Task S.4) developed a methodology to facilitate quantitative assessments of current and future structural reliability and performance of concrete structures in NPPs, taking into account those effects that might diminish the ability of the structures to withstand future operating, extreme environmental, or accidental conditions. Associated activities in meeting this objective included 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 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/repair strategies. The level 3 work breakdown structure for this task identifying subtasks and primary activities under each subtask is presented in Fig. 1.5.

Appendix B presents a listing of reports and papers that have been developed under the SAG Program. Contained in the balance of this report is a compendium of knowledge that is intended to provide the necessary data and information required to either develop an effective aging management program for the NPP reinforced concrete structures or to evaluate an existing program. Basic components of one approach for use in setting up a program for these structures is presented in Fig. 1.6. The relation of the various sections of the report to this approach is identified in the figure.

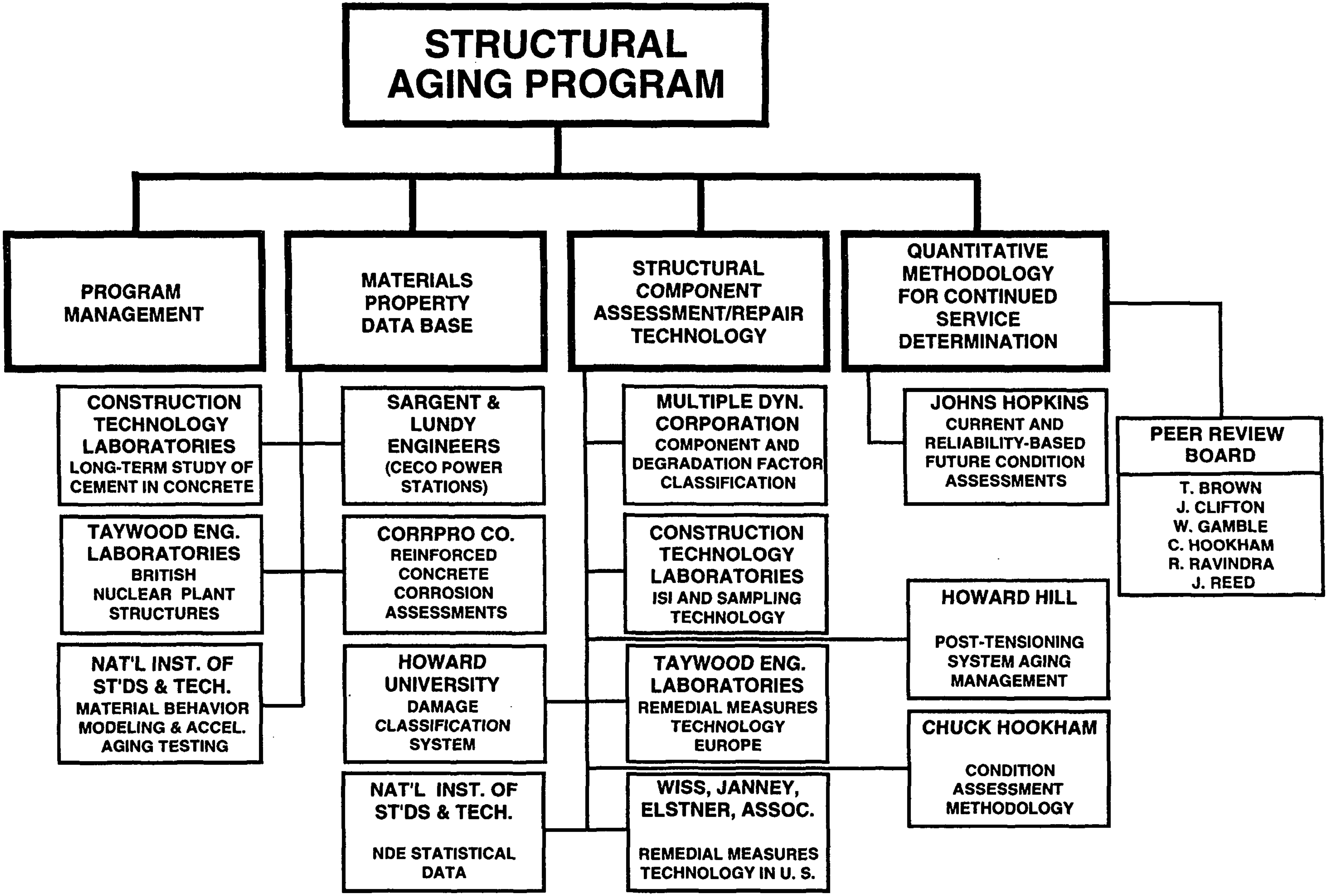


Fig. 1.1 Listing of subcontracted activities for each primary task area.

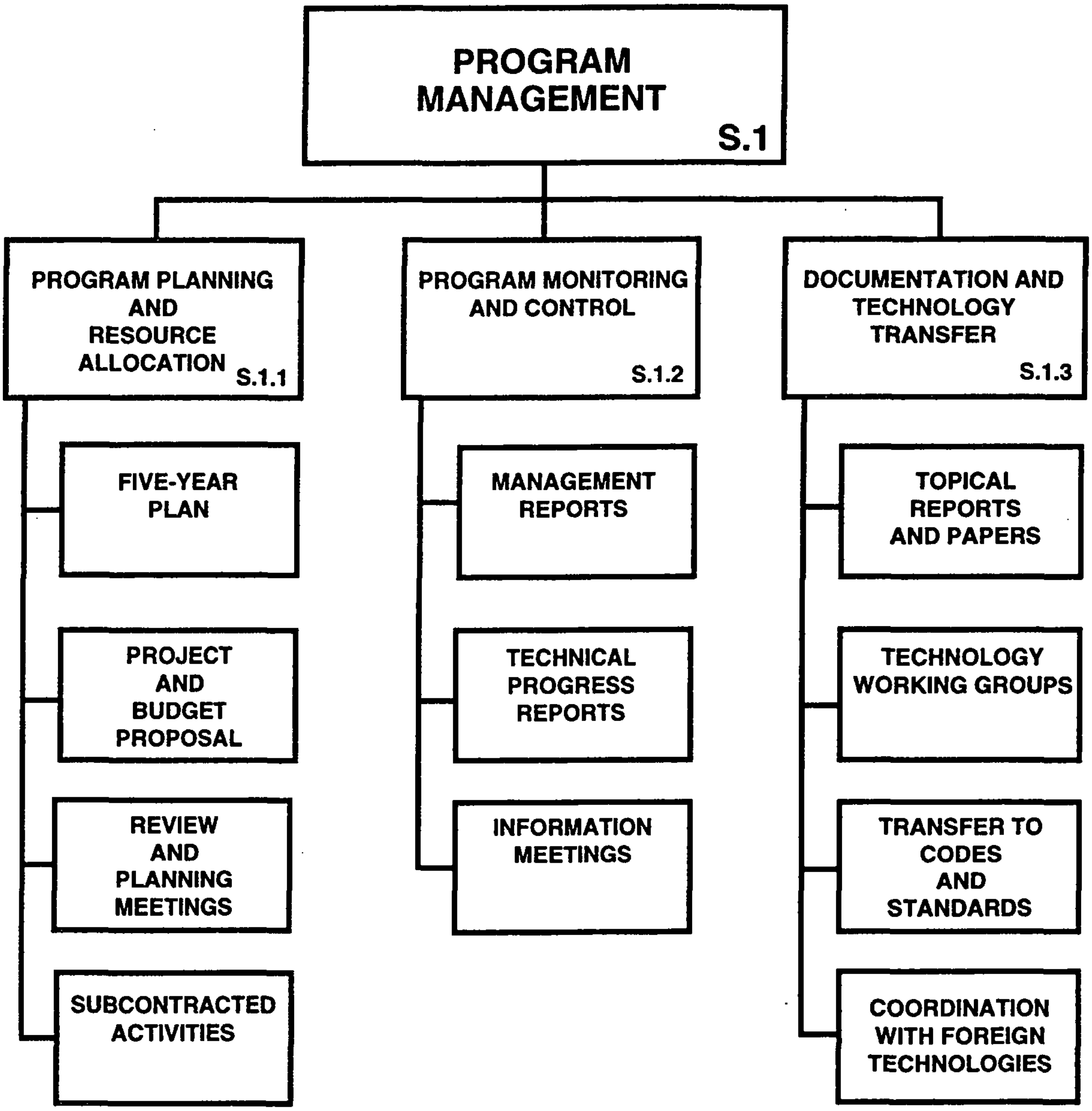


Fig. 1.2 Level 3 work breakdown structure for SAG Task S.1: Program Management.

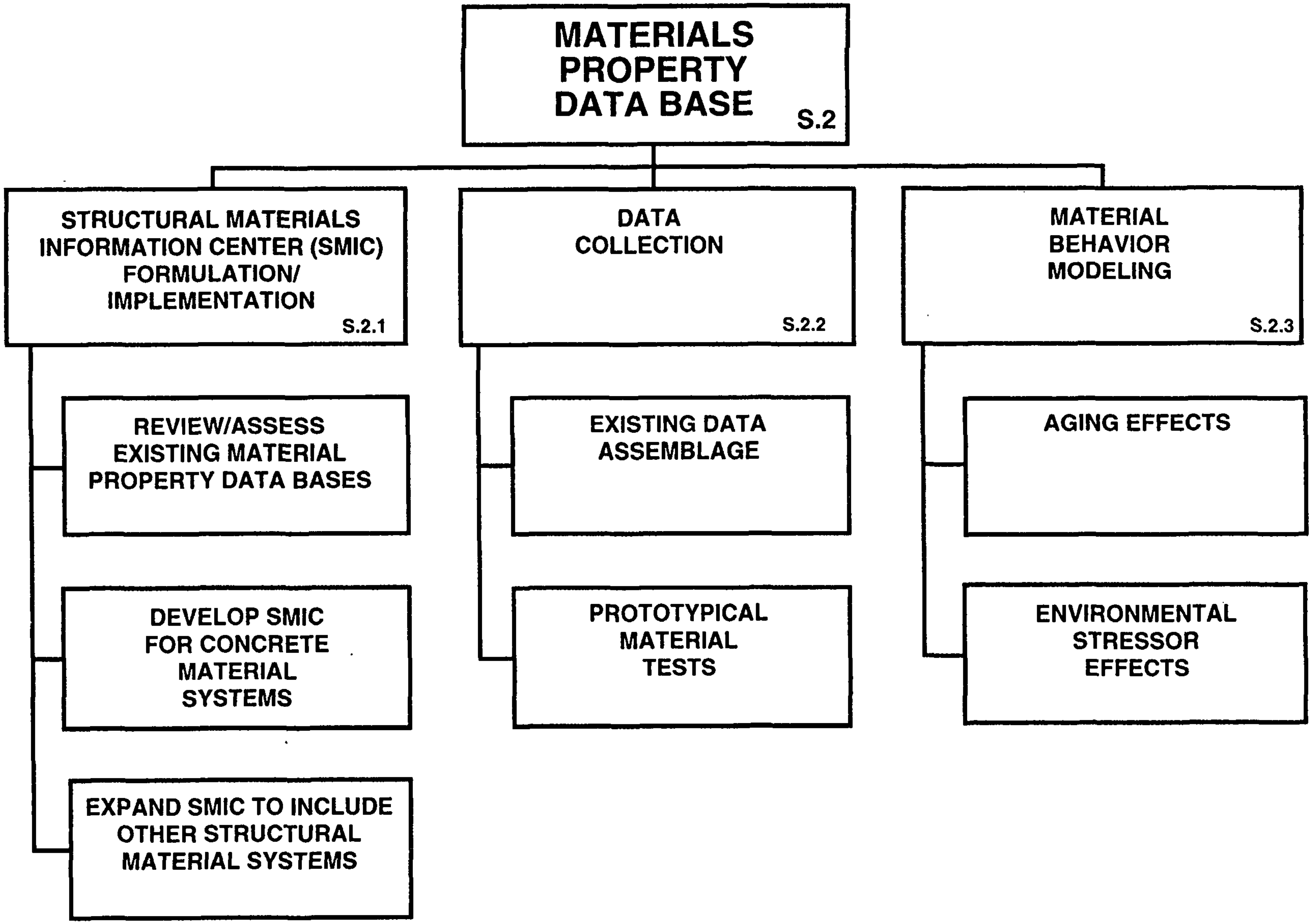


Fig. 1.3 Level 3 work breakdown structure for SAG Task S.2: Materials Property Data Base.

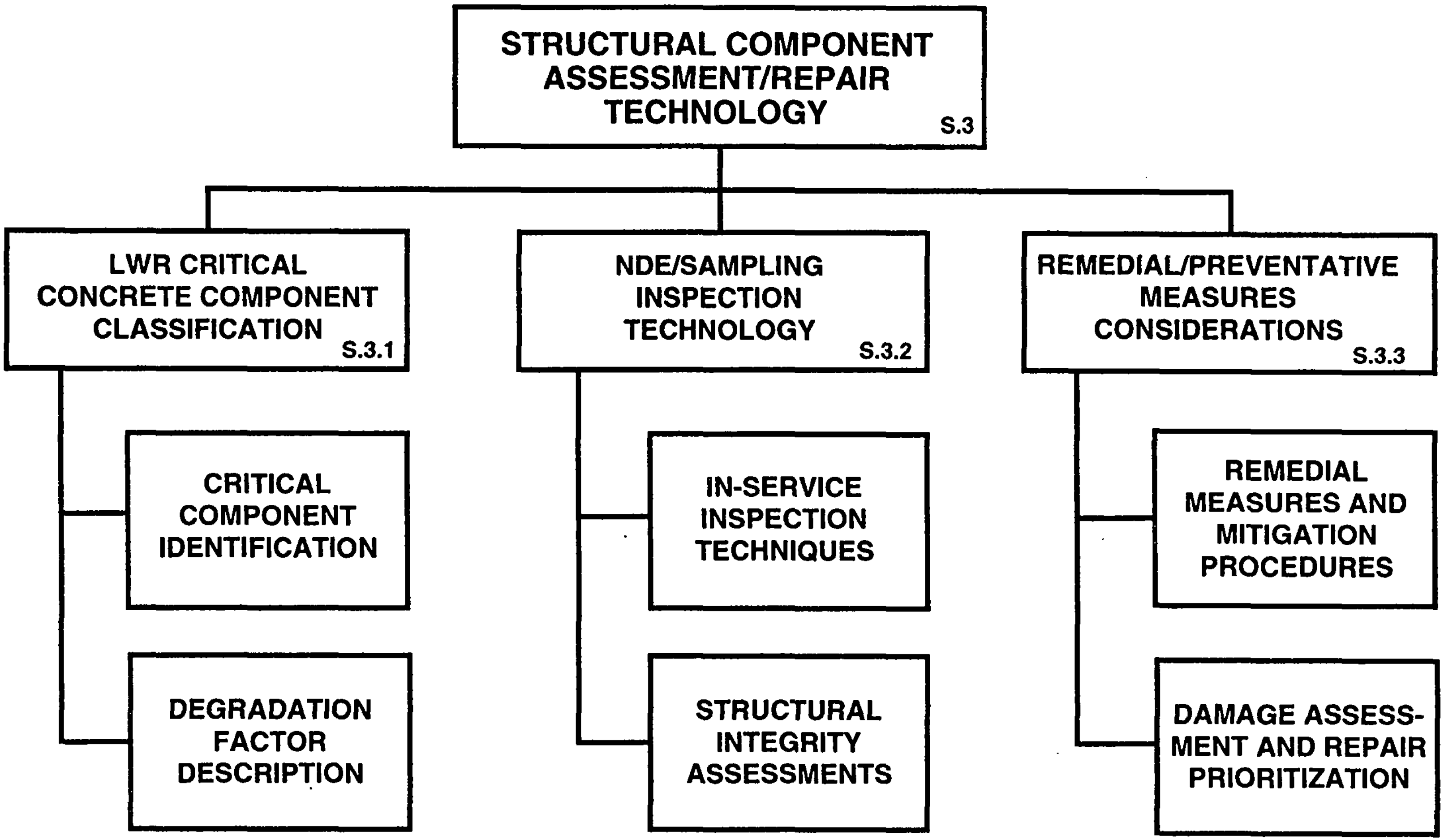


Fig. 1.4 Level 3 work breakdown structure for SAG Task S.3: Structural Component Assessment/Repair Technology.

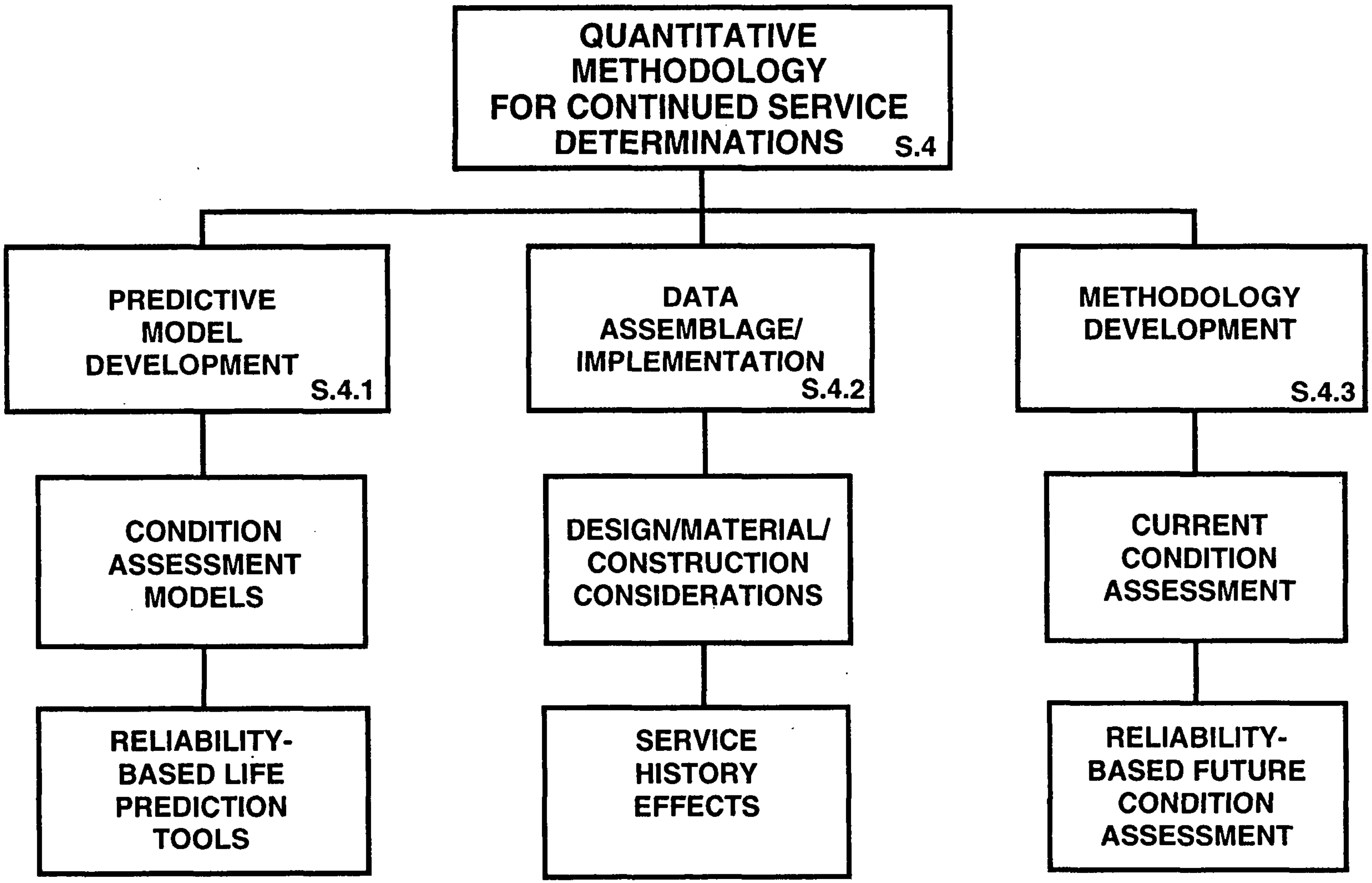


Fig. 1.5 Level 3 work breakdown structure for Task S.4: Quantitative Methodology for Continued Service Determinations.

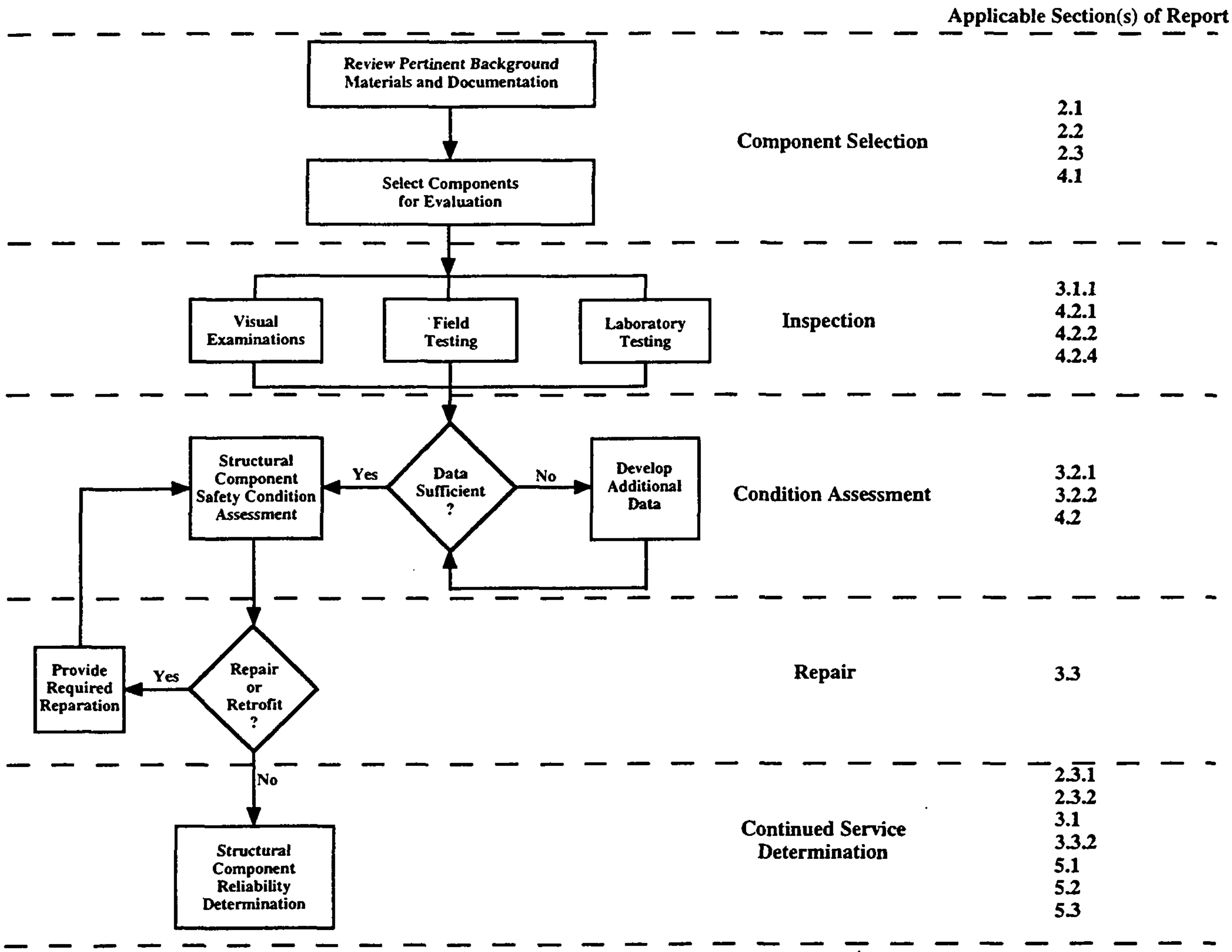


Fig. 1.6 Evaluation methodology for NPP reinforced concrete structures.

2. SAFETY-RELATED CONCRETE STRUCTURES DESCRIPTION AND LONGEVITY CONSIDERATIONS

2.1 DESIGN CRITERIA

All commercial nuclear power plants (NPPs) in the U.S. contain concrete structures whose performance and function are necessary for protection of the safety of plant operating personnel and the general public, as well as the environment. The basic laws that regulate the design (and construction) of NPPs are contained in Title 10 of the CFR[4] that is clarified by Regulatory Guides (e.g., R.G. 1.29),[14] NUREG reports, Standard Review Plans (e.g., Concrete and Steel Internal Structures of Steel or Concrete Containments),[15] etc. In addition, R.G. 1.29 and Part 100 to Title 10 of the CFR state that NPP structures important to safety must be designed to withstand the effects of earthquakes without the loss of function or threat to public safety. These "safety-related" structures are designated as seismic Category I. Seismic Category I structures typically include those classified by the American Society of Mechanical Engineers (ASME) and the American Nuclear Society (ANS) as Safety Classes 1, 2, and 3 (i.e., safety related).

Initially, existing building codes such as the American Concrete Institute (ACI) Standard 318, *Building Code Requirements for Reinforced Concrete*,[16] were used in the nuclear industry as the basis for the design and construction of concrete structural members. However, because the existing building codes did not cover the entire spectrum of design requirements and because they were not always considered adequate, the U.S. Nuclear Regulatory Commission (USNRC) developed its own criteria for the design of Category I structures (e.g., definitions of load combinations for both operating and accident conditions). Current requirements for nuclear safety-related concrete structures, other than concrete reactor vessels and concrete containments, are also based on ACI 318, but have incorporated modifications to accommodate the unique performance requirements of NPPs. These requirements were developed by ACI Committee 349 and first published in October 1976.[17] This Code has been endorsed by the USNRC as providing an adequate basis for complying with the general design criteria for structures other than reactor vessels and containments.[18] Reference 19 provides additional information on the design of seismic Category I structures that are required to remain functional if the Safe Shutdown Earthquake (SSE) occurs. Current requirements for concrete reactor vessels and concrete containments were developed by ACI Committee 359 and first published in 1977.[20] Supplemental load combination criteria are presented in Sect. 3.8.1 of the USNRC *Regulatory Standard Review Plan*.[21] However, since all but one of the construction permits for existing NPPs have been issued prior to 1978, it is unlikely that endorsed versions of either ACI 349 or ACI 359 were used in the design of many of the concrete structures at these plants. Older plants that used early ACI codes, however, have been reviewed by the USNRC through the Systematic Evaluation Program to determine if there were any safety concerns.[22]

2.2 SAFETY-RELATED CONCRETE STRUCTURES

A myriad of concrete-based structures are contained as a part of a light-water reactor (LWR) plant to provide foundation, support, shielding, and containment functions. Table 2.1* presents a listing of typical safety-related concrete structures that may be included as part of a LWR plant. Only a general description of these structures is provided in the following sections because detailed information of this type along with typical design parameters and operating conditions is

* Tables and figures appear at the end of each chapter.

provided elsewhere.[13,23–26] Information pertaining to a particular structure at a plant of interest can be obtained from sources such as the plant's safety analysis report or docket file. Concrete structures that are considered to be "plant specific" or unique have not been addressed in the discussion below, but some information provided for similar structures may be applicable. Additionally, the names of certain structures may vary from plant-to-plant depending on the nuclear steam supply system (NSSS) vendor, architect-engineering firm, and owner preference. The safety-related concrete structures, for purposes of discussion, have been separated into two categories — typical plant structures and auxiliary structures.

2.2.1 Typical Plant Structures

2.2.1.1 Boiling-water reactors

Typical safety-related concrete structures contained in boiling-water reactor (BWR) plants can be grouped into four general categories: primary containments, containment internal structures, secondary containments/reactor buildings, and other structures. Table 2.2 presents a summary of BWR structures that typically are included in the categories.

Primary Containment Of the 37 BWR plants that have been licensed for commercial operation in the U.S., 11 utilize either reinforced (2 Mark I, 5 Mark II, 2 Mark III) or prestressed (2 Mark II) concrete primary containments. Leak tightness of each of these containments is provided by a steel liner attached to the containment inside surface by studs (e.g., Nelson studs) or by structural steel members. Exposed surfaces of the carbon steel liner are typically painted to protect against corrosion and to facilitate decontamination should it be required. A portion of the liner toward the bottom of the containment and over the basemat is typically embedded in concrete to protect it from damage, abrasion, etc. due to corrosive fluids and impact. A seal to prevent the ingress of fluids is provided at the interface around the circumference of the containment where the vertical portion of the liner becomes embedded in the concrete.

BWR containments, because of provisions for pressure suppression, typically have "normally dry" sections (drywell) and "flooded" sections (wetwell) that are interconnected via piping or vents (see Figs. 2.1–2.3). Requirements for BWR containments include the following:

1. Provide an "essentially" leak-tight barrier against the uncontrolled release of radioactivity to the environment for all postulated design basis accident conditions;
2. Accommodate the calculated pressure and temperature conditions resulting from a loss-of-coolant accident;
3. Withstand periodic integrated leak-rate testing at the peak calculated accident pressure that may be at levels up to and including the containment design pressure; and
4. Permit appropriate periodic inspection of all important components and surfaces and the periodic testing of the leak tightness of containment penetrations.

In addition, the containment vessel can provide structural support for the NSSS and other internal equipment. The containment foundation, typically a basemat, provides the primary support and transfer of load to the earth below.

Containment Internal Structures Each of the three BWR plant types (Mark I, Mark II, and Mark III) incorporate a number of reinforced concrete containment internal structures. These structures may perform singular or several functions including the following:

1. Radiation shielding;
2. Human accessibility provisions;

3. NSSS and other equipment anchorage/support/protection;
4. Resistance to jet, pipe whip, and other loadings produced by emergency conditions;
5. Boundary of wetwells and pool structures, allow communication between drywell and wetwell (Mark II and III);
6. Lateral stability for containment;
7. Transfer of containment loads to underlying foundation; and
8. Transfer of fuel to reactor (Mark III).

As many of these functions are interrelated with the required containment functions, these structures are considered safety-related.

Secondary Containments/Reactor Buildings Of the 26 BWR plants that utilize steel primary containments (1 Pre-Mark, 22 Mark I, 1 Mark II, 2 Mark III), all but the pre-Mark plant have reinforced concrete structures that serve as secondary containments or reactor buildings and provide support and shielding functions for the primary containment. Although the design parameters for the secondary containments of the Mark I and Mark II plants vary somewhat, the secondary containments are typically composed of beam, floor, and wall structural elements. These structures typically are safety-related because they provide additional radiation shielding; provide resistance to environmental/operational loadings; and house safety-related mechanical equipment, spent fuel, and the primary metal containment. Although these structures may be massive in cross-section in order to meet shielding or load-bearing requirements, they generally have smaller elemental thicknesses than primary containments because of reduced exposure under postulated accident loadings. These structures may be maintained at a slight negative pressure for collection and treatment of any airborne radioactive material that might escape during operating conditions.

Other Structures Included in this category are such things as foundations, walls, slabs, and fuel/equipment storage pools. The spent- and new-fuel storage pools, and the pools for reactor internals storage, typically have a four wall-with-bottom slab configuration. The walls and slab are composed of reinforced concrete members lined on the interior surface with stainless steel. Cross-sections of these members are generally large because they must support a large pool of water and heavy fuel/component loads, produced by high-density fuel storage considerations. The fuel storage pool in Mark III plants is located within the primary containment.

2.2.1.2 Pressurized-water reactors

Typical safety-related concrete structures in pressurized-water reactor (PWR) plants also can be grouped into the four general categories noted above for the BWR plants. Table 2.3 presents a summary listing of PWR structures that typically are included in these categories.

Primary Containment Of the 72 PWR plants that have been licensed for commercial operation in the U.S., 59 utilize either reinforced (11 large dry, 8 subatmospheric, 2 ice condenser) or prestressed (38 large dry) concrete primary containments. In meeting the same basic functional and performance requirements as noted for BWR containments in Sect. 2.2.1.1, the concrete containments in PWR plants are of three different functional designs (Fig. 2.4–2.6): subatmospheric (reinforced concrete), ice condenser (reinforced concrete), and large/dry (reinforced and prestressed concrete). The primary differences between these containment designs relate to volume requirements, provisions for accident loadings/pressures, and containment internal structures layout.

The PWR containment structure generally consists of a concrete basemat foundation, vertical cylindrical walls, and dome. The basemat may consist of a simple mat foundation on fill, natural cut or bedrock, or may be a pile/pile cap arrangement. Most of the plants have utilized the simple mat on fill or bedrock design. Interior containment surfaces are lined with a thin carbon steel liner to prevent leakage. Two of the PWR plants (Bellefonte and Ginna) have rock anchor systems to which the post-tensioning tendons are attached.

<u>Containment Internal Structures</u> The containment internal structures in PWR plants are typically constructed of conventionally reinforced concrete and tend to be more massive in nature than the internal structures in BWR plants because they typically support the reactor pressure vessel, steam generators, and other large equipment and tanks. In addition, these structures provide shielding of radiation emitted by the NSSS. Some of the specific functions that these structures (typically floor slabs, walls, and columns) are required to perform include:

1. Provision of human accessibility;
2. Support and separation of various plant equipment;
3. Resistance to emergency loading conditions;
4. Transfer of containment loads to containment foundation;
5. Missile protection; and
6. Channeling/routing steam and air through ice condensers (PWR ice condenser containments).

<u>Secondary Containments/Reactor Buildings</u> PWR plants that utilize a metallic primary containment (large dry and ice condenser designs) are usually contained in reinforced concrete "enclosure" or "shield" buildings. This secondary containment consists of a vertical cylinder wall with shallow dome (Fig. 2.7) and is often supported by the containment basemat. In addition to withstanding environmental effects, the secondary containment provides radiation shielding and particulate collection and ensures that the free standing metallic primary containment is protected from the natural environment.

<u>Other Structures</u> Except for differences in the spent- and new-fuel storage pools, structures that fall into the other structures category are essentially the same at the PWR and BWR plants. The spent- and new-fuel storage pools for PWR plants are typically located in an auxiliary building proximate to the containment. These reinforced concrete wall and slab structures are generally massive in cross-section to support a large pool of water and the fuel elements, and are lined on the water side with stainless steel. The pools are connected to the reactor/refueling cavity (inside containment) via a transfer channel that is also a safety-related structure since it must provide radiation shielding and support for the fuel transport mechanism and fuel.

2.2.2 Auxiliary Structures

Auxiliary structures are considered to be those concrete structures in a NPP that may or may not perform safety-related functions, depending on the plant-unique or site-specific design and licensing or operating criteria. These structures typically house important plant equipment or control-room facilities or provide additional radiation shielding/containment to meet 10 CFR requirements. They may be located immediately adjacent to the secondary containment (e.g., auxiliary building, diesel generator building, etc.) or be separated on site (e.g., intake structures, offgas stacks, etc.). Although these reinforced concrete structures may take many different physical configurations in meeting their functional and performance requirements, they typically fall into two broad categories: (1) common structures, and (2) plant-unique structures.

2.2.2.1 Common structures

Common building structures are typically configured in a rectangular box shape, and consist of reinforced concrete floor slabs, walls, and mat foundation. These subelements are typically of lighter construction (thinner sections with reduced conventional reinforcing) than the plant containment structures. They may also be composite with structural steel framing and contain shear walls for vertical and horizontal load resistance. Primary functions of these structures are to provide an enclosure for equipment important to plant safety and to provide secondary radiation containment.

2.2.2.2 Plant-unique structures

Plant-unique concrete structures include components such as intake canal liners, offgas stacks, and emergency cooling pathways. Although these structures are typically constructed of conventional reinforced concrete, their configuration and methods of construction differ from that of general building construction because the structures must meet specific design loading conditions dictated by their function as well as that of potential extreme environmental conditions (e.g., earthquake, flood, tornado, etc.). In addition, these structures may be required to resist the effects of the natural environment, and may be exposed to cooling water (river, ocean, lake). Typically, the plant-unique structures contribute to plant safety by serving to dissipate heat and radiation, or to protect other safety-related components.

2.3 AGING AND LONG-TERM DURABILITY CONSIDERATIONS

As previously noted, the reinforced concrete structures at NPPs have been designed in accordance with national consensus codes and standards (e.g., ACI 318).[16] 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 procedures 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 the concrete structures in conformance with these calculations was a certain level of durability (e.g., minimum requirements for concrete cover to protect embedded steel reinforcement under different anticipated environmental conditions). Although the vast majority of reinforced concrete structures, and particularly those associated with NPPs, have and continue to meet their functional and performance requirements, examples can be cited where these structures have exhibited degradation primarily due to the effects of environmental stressors.[13,24,27] Estimations of the future performance of NPP concrete structures requires knowledge on their materials of construction, potential sources of degradation, and performance history or longevity.

2.3.1 Materials of Construction

Nuclear safety-related concrete structures are composed of several constituents that, in concert, perform multiple functions (e.g., load-carrying capacity, radiation shielding, and leak tightness). Primarily, these constituents include the following material systems: concrete, conventional steel reinforcement, prestressing steel, steel liner plate, and embedment steel. More detailed information on materials of construction is available elsewhere.[13,20,24]

2.3.1.1 Concrete

The concrete typically used in nuclear safety-related structures consists of Type II portland cement,[28] fine aggregates (e.g., sand), water, various admixtures for improving properties or performance of the concrete, and either normal-weight or heavyweight coarse aggregate. Type II portland cement was used because of its improved sulfate resistance and reduced heat of hydration relative to the general purpose Type I portland cement. Both the water and fine and coarse aggregates are normally acquired from local sources and subjected to material characterization testing prior to use. Various admixtures have been used to improve air entrainment (i.e., enhanced durability), improve workability (i.e., enhanced placement and compaction), modify hardening or setting characteristics, aid in curing, reduce evolution of heat, or provide other property improvements.[29] Coarse aggregate consists of gravel, crushed gravel, or crushed stone. For those concrete structures in nuclear power plants that provide primary (biological) radiation shielding, heavyweight, or dense aggregate materials, such as barites, limonites, magnetites, ilmenites, etc., may have been used to reduce the section thickness requirements needed for attenuation.

The constituents are then proportioned and mixed to develop portland cement concrete that has specific properties. Depending on the characteristics of the specific structure, the concrete mix may be adjusted to provide increased strength, higher durability, or better workability for placement. The hardened concrete typically provides the compressive load-carrying capacity for the structure. Reinforced concrete, in one form or another, has been used in the construction of all LWR plants. Specified concrete unconfined compressive strengths typically have ranged from 13 to 55 MPa, with 28 MPa being most common.

2.3.1.2 Conventional steel reinforcement

Most of the mild, or conventional, reinforcing steels[30] used in nuclear power plants to provide primary tensile and shear load resistance/transfer consists of plain carbon steel bar stock with deformations (lugs or protrusions) on the surface. These bars typically conform with ASTM A 615[31] or A 706[32] specifications (older vintage plants may contain bars conforming to ASTM A 432[33] or A 305[34] specifications that have been superseded by the above). The minimum yield strength of this material ranges from 270 MPa to 415 MPa, with the 415 MPa material being most common. Conventional reinforcing steel also encompasses welded wire fabric (ASTM A 185[35] and A 497[36]), deformed wire (ASTM A 496[37]), bar and rod mats (ASTM A 184[38]), and all accessory steel components used in positioning/placing the reinforcement (e.g., seats, ties, etc.).

2.3.1.3 Prestressing steel

A post-tensioned prestressing system consists of tendons that are installed, tensioned, and then anchored to the hardened concrete forming the structure. A number of concrete containment structures utilize post-tensioned steel tendons to provide primary resistance to tensile loadings. Three major categories of prestressing systems exist depending on the type of tendon utilized: wire, strand, or bar. These systems typically conform to ASTM specifications A 421,[39] A 416,[40] and A 722[41] and have minimum ultimate strengths ranging from 2000 kN to 10,000 kN. The tendons are installed within preplaced ducts in the containment structure and are post-tensioned from one or both ends after the concrete has achieved sufficient strength. After tensioning, the tendons are anchored by buttonheads, wedges, or nuts. Corrosion protection is provided by filling the ducts with wax or corrosion-inhibiting grease (unbonded), or portland cement grout (bonded). Supplemental conventional reinforcing is also used to minimize shrinkage or temperature effects and to provide local load-carrying capacity or load transfer. With the exception of Robinsion 2 (bar tendons) and Three Mile Island 2 (strand tendons), plants that have post-tensioned containments utilize unbonded tendons. Bellefonte and Ginna each have grouted tendons (rock anchors) to which tendons are attached.

2.3.1.4 Liner plate

Leak tightness of reinforced and post-tensioned concrete containment vessels is provided by steel liner plate. A typical liner is composed of steel plate stock less than 13 mm thick, joined by welding, and anchored to the concrete by studs (Nelson studs or similar conforming to ASTM A 108[42]), structural steel shapes, or other steel products. The drywell portions of BWR containments and PWR containments are typically lined with carbon steel (ASTM A 36[30] or A 516[43]). The liners of LWR fuel pool structures typically consist of stainless steel (ASTM A 276,[44] Type 304[45] is common). The liners of wetwells also have used carbon steel materials such as ASTM A 285,[46] A 516,[43] and A 537.[47] Certain LWR facilities also have used carbon steel clad with stainless steel weld metal for liner members. Although the liner's primary function is to provide a leaktight barrier for containment of radiation, it acts as part of the formwork during concrete placement and may be used in the support of internal piping/equipment.

2.3.1.5 Embedment steel

Anchorage to concrete is required for heavy machinery, structural members, piping, ductwork, cable trays, towers, and many other types of structures. Anchorage design had to meet certain requirements such as ease of installation, load capacity, susceptibility to vibration, preload retention, temperature range, corrosion resistance, postinstallation or preinstallation, and ease of inspection or stiffness. In meeting its function, loads that the anchor must transfer to the concrete vary over a wide combination of tension, bending, shear, and compression. Examples of types of anchors available include embedded bolts (ASTM A 307,[48] A 325,[49] or A 490[50]), grouted bolts, embedded studs, self-drilled expansion anchors, or wedge anchors. Embedded steel, typically ASTM A 36,[30] may also be constructed of structural plates or shapes installed during concrete placement

2.3.2 Aging Factors and Environmental Stressors

As concrete ages, changes in its properties will occur as a result of continuing microstructural changes (e.g., slow hydration, crystallization of amorphous constituents, and reactions between cement paste and aggregates), as well as environmental influences. These changes do not have to be detrimental to the point that reinforced concrete will not be able to meet its functional and performance requirements. When specifications covering concrete's production are correct and followed, reinforced concrete need not deteriorate.[51] Concrete, however, can suffer undesirable degrees of change with time because of improper specifications, a violation of specifications, or environmental effects. Basic factors related to the production of durable concrete include materials selection, design, execution, and curing, Fig. 2.8.[52] Quality control/quality assurance programs at NPPs generally have been very effective in ensuring that these factors are adequately addressed.[53]

Surveys of reported errors involving general civil engineering concrete structures in North American and Europe[54,55] concluded that when errors occurred they were almost always the result of faulty construction or design deficiencies.* Errors due to construction were generally discovered during construction. A similar conclusion was derived from a survey questionnaire

* A limitation of the North American study was that the information presented was strongly biased toward errors that escaped detection until revealed by the structure and thus did not present a true picture of the error-detection process of the review check system. Also, the survey favored those structures and serviceability characteristics that revealed themselves in a short period of time and thus does not represent the actual incidences of concrete degradation.[54]

sent to U.S. utilities to obtain information related to in-service inspection procedures, incidences of degradation, and repair procedures that have been utilized with respect to the concrete structures.[56] Responses to the survey questionnaire, provided by slightly less than half the commercial NPPs in the U.S., indicated that the majority of problems associated with the concrete structures initiated during construction and have been corrected.

Results of the surveys noted above indicate that as these structures age, deterioration due to the effects of environmental stressors is the primary threat to their durability. From an aging management perspective, it is important not only to understand the potential degradation factors and their manifestations, but to be able to estimate future performance through service life models.

2.3.2.1 Environmental stressor considerations

The longevity, or long-term performance of safety-related concrete structures is primarily a function of the durability or propensity of these structures to withstand the potential effects of degradation. Table 2.4 presents a summary of the degradation factors that potentially can impact the performance of the basic constituents that compose safety-related concrete structures in NPPs (i.e., concrete, mild steel reinforcement, post-tensioning system, and liner/structural steel members). Also contained in the table is a listing of primary manifestations of each degradation factor. A listing of several areas in NPPs that potentially may experience degradation is provided in Table 2.5. More detailed information to that summarized below is available elsewhere.[13,23–27]

<u>**Concrete Material Systems**</u> The durability of concrete materials can be limited as a result of adverse performance of its cement-paste matrix or aggregate constituents under either chemical or physical attack. In practice, these processes may occur concurrently to reinforce each other. In nearly all chemical and physical processes influencing the durability of concrete structures, dominant factors involved include transport mechanisms within the pores and cracks, and the presence of water.

Cracking occurs in virtually all concrete structures and, because of concrete's inherently low tensile strength and lack of ductility, can never be totally eliminated. Cracks are significant from the standpoint that they can indicate major structural problems (active cracks); provide an important avenue for the ingress of hostile environments (active or dormant cracks); and may inhibit a component from meeting its performance requirements (active or dormant cracks) (e.g., diminished shielding capacity). As shown in Fig. 2.9,[57] cracks in concrete can form either before or after the concrete has hardened. Examples of cracks that form in concrete due to movements generated within the concrete (e.g., shrinkage, and expansion and contraction due to temperature change) or expansion of material embedded within the concrete (e.g., corrosion of steel reinforcement) are presented in Fig. 2.10,[57] and an indication of the potential time occurrence of cracks in concrete due to several of these causes is noted in Fig. 2.11.[52] Information on cracking and its classification with respect to damage is provided in Ref. 58. From an aging management perspective for existing structures, causes of concrete degradation due to either chemical or physical causes are of primary interest.

Chemical Attack

Chemical attack involves the alteration of concrete through chemical reaction with either the cement paste or coarse aggregate, or embedded steel reinforcement.** Generally, the attack occurs on the exposed surface region of the concrete (cover concrete), but with the presence of cracks or

** Corrosion of embedded steel reinforcement due to carbonation of the concrete or the action of chloride ions is covered under the section addressing mild steel reinforcement.

prolonged exposure, chemical attack can affect entire structural cross sections. Chemical causes of deterioration can be grouped into three categories: (1) hydrolysis of cement paste components by soft water; (2) cation exchange reactions between aggressive fluids and the cement paste; and (3) reactions leading to formation of expansion products.[59] The rate of chemical attack on concrete is a function of the pH of the aggressive fluid and the concrete permeability, alkalinity, and reactivity. Figure 2.12 presents a summary of the types of chemical reactions responsible for concrete deterioration and the detrimental effects that can occur.[60] Chemical attack of concrete may occur in several different forms as highlighted in the following sections.

Leaching and Efflorescence Pure water that contains little or no calcium ions, or acidic groundwater present in the form of dissolved carbon dioxide gas, carbonic acid, or bicarbonate ion, tends to hydrolyze or dissolve the alkali oxides and calcium-containing products. The rate of leaching is dependent on the amount of dissolved salts contained in the percolating fluid, rate of permeation of the fluid through the cement paste matrix, and temperature. Extensive leaching causes an increase in porosity and permeability thus lowering the strength of the concrete and making it more vulnerable to hostile environments (e.g., water saturation and frost damage, or chloride penetration and corrosion of embedded steel). The rate of leaching can be controlled by minimizing the percolation of water through the concrete. Concretes produced using low water-cement ratios, adequate cement content, and proper compaction and curing are most resistant to leaching.

Efflorescence occurs on the surface of concrete following the percolation of a fluid (e.g., water) through the material, either intermittently or continuously, or when an exposed surface is alternately wetted and dried. It forms due to crystallization of the dissolved salts as a result of evaporation of the fluid or interaction with carbon dioxide in the atmosphere. As such, efflorescence is primarily an aesthetic problem rather than a durability problem, but may indicate that alterations to the cement paste are taking place in the concrete.

Sulfate Attack Magnesium and alkali sulfates present in soils, groundwater, and seawater react with the calcium hydroxide and alumina-bearing phases of portland cement to form gypsum and ettringite. These reactions, if enough water is present, result in expansion and irregular cracking of the concrete that can lead to progressive loss of strength and mass. Structures subjected to seawater are more resistant to sulfate attack because of the presence of chlorides that form chloro-aluminates to moderate the reaction. Concretes that use cements low in tricalcium aluminate (e.g., Type V sulfate resisting) and those that are dense and of low permeability are most resistant to sulfate attack. Guidelines on concrete exposed to sulfate attack are provided in Refs. 16 and 61.

Acids and Bases. Acids present in groundwater (e.g., sulfuric or carbonic) and certain plant internal fluids (e.g., boric and sulfuric acids) can combine with the calcium compounds in the hydrated cement paste (i.e., calcium hydroxide, calcium silicate hydrate, and calcium aluminate hydrate) to form soluble materials that are readily leached from the concrete to increase its porosity and permeability. The main factor determining the extent of attack is not so much the aggressiveness of the attacking acid, but more the solubility of the resulting calcium salt. The rate of deterioration is also accelerated if the aggressive chemical solution is flowing. Since under acid attack there is a conversion of the hardened cement, the concrete permeability is not as important as for other types of chemical attack (e.g., leaching and sulfate attack). Due to the large buffering capacity of concrete and the relatively small amount of acid contained in rain, acid rain will convert only an insignificant amount of the concrete.[52] Acid rain is even a smaller threat to NPP structures than general civil engineering concrete structures because of their massive cross sections.

As hydrated cement paste is an alkaline material, high quality concretes made with chemically stable aggregates normally are resistant to bases. However, sodium and potassium hydroxides in high concentrations (> 20%) can cause concrete to disintegrate.

Under mild chemical attack, a dense concrete with low water-cement ratio may provide suitable resistance. As corrosive chemicals can attack concrete only in the presence of water, designs to minimize attack by acids and bases generally involve the use of protective barrier systems. Table 2.6 presents a listing of the reactivity with concrete of various chemicals that may be found in NPPs or the surrounding environment. Reference 62 presents additional information on the effect of chemicals on concrete.

Alkali-Aggregate Reactions Chemical reactions involving alkali ions (portland cement), hydroxyl ions, and certain siliceous constituents that may be present in aggregate materials can form a gel. As the alkali-silica gel comes in contact with water, swelling (i.e., hydraulic pressure) occurs that can cause cracking that eventually could lead to complete destruction of the concrete.[63] Visible concrete damage starts with small surface cracks exhibiting an irregular pattern (or map cracking). The expansion will develop in the direction of least constraint (i.e., parallel surface patterns developing inward from surface for slabs and cracking parallel to compression forces in columns or prestressed members). Pop-outs and glassy appearing seepage of varying composition can appear as a result of alkali-silica reactions. Expansion reactions also can occur as a result of alkali-carbonate reactions (i.e., dedolomitization).

Primary factors influencing alkali-aggregate reactions include the aggregate reactivity (i.e., amount and grain size of reactive aggregate), alkali and calcium concentrations in concrete pore water, cement content (i.e., alkali content), and presence of water. Although alkali-aggregate reactions typically occur within 10 years of construction, deterioration has not occurred in some structures until 15 or even 25 or more years following construction. The delay in exhibiting deterioration indicates that there may be less reactive forms of silica that can eventually cause deterioration.[64] Control of the alkali-aggregate reactions is generally through elimination of deleteriously reactive aggregate materials from consideration through petrographic examinations, laboratory evaluations, and use of materials with proven service histories. An improved rapid test method for evaluating the alkali reactivity of siliceous aggregates has been proposed.[65] A method for evaluating potential alkali reactivity of carbonate aggregate is also available.[66] Known deleterious reactive aggregate materials are provided in ACI 201.2R.[67] Additional mitigating procedures include use of pozzolans, restricting the cement alkali contents to less than 0.6% by weight Na_20 equivalent, and application of barriers to restrict or eliminate moisture.

Physical Attack

Physical attack involves the degradation of concrete due to external influences and generally involves cracking due to exceeding the tensile strength of the concrete, or loss of surface material. Concrete attack due to overload conditions is not considered as an aging mechanism.

Salt Crystallization Salts can produce cracks in concrete through crystal growth pressures that arise through physical causes (e.g., repeated salt crystallization due to evaporation in the pores). Structures in contact with fluctuating water levels or in contact with groundwaters containing large quantities of dissolved salts (e.g., $NaC\ell$, $CaS0_4$, and $NaS0_4$) are susceptible to this type of deterioration. The problem of salt crystallization is minimized for low permeability concretes and where sealers or barriers have been effectively applied to prevent water ingress or subsequent evaporation.

Frost Attack Concrete, when in a saturated or near saturated condition, can be susceptible to damage during freezing and thawing cycles caused by hydraulic pressure generated in the capillary cavities of the cement paste as water freezes. Damage to concrete resulting from frost attack can take several forms: scaling, spalling, and pattern cracking (e.g., D-cracking).[64] The damage is incurred after an extended number of cycles and is observed on exposed surfaces of affected structures. Factors controlling the resistance of concrete to frost action include air entrainment (i.e., size and spacing of air bubbles as opposed to entrapped air), water-cement ratio, curing, strength, and degree of saturation. Selection of durable aggregate materials is also important. An estimate of the susceptibility of concrete aggregates for known or assumed field environmental conditions can be provided from ASTM C 682.[68] Guidelines for production of frost resistance concretes are provided in ACI 201.2R[67] and ACI 318[16] in terms of total air content as a function of maximum aggregate size and exposure condition. Areas in the U.S. subjected, on average, to the largest number of freeze-thaw cycles per year are provided in a weathering index chart contained in Ref. 69.

Abrasion/Erosion/Cavitation Progressive loss of material at the concrete surface can occur due to abrasion, erosion, or cavitation. Abrasion generally refers to dry attrition, while erosion is normally used to describe wear by the abrasive action of fluids containing solid particles in suspension. Cavitation relates to the loss of surface material by formation of vapor bubbles and their subsequent collapse, due to sudden change of direction or pressure in rapidly flowing water, on the surface of the structure. Resistance of concrete to abrasion and erosion is dependent on the quality of the concrete (low porosity, high strength) and in particular the aggregate particles used in the mix. While good quality concrete may show good resistance to abrasion and erosion, it may still suffer severe loss of surface material due to cavitation. The best way to guard against the effects of cavitation is to eliminate the cause(s) of cavitation. Reference 70 provides additional information on the effects of erosion on concrete structures.

Thermal Exposure/Thermal Cycling Elevated-temperature exposure and thermal gradients are important to concrete structures in that they affect the concrete's strength (i.e., ability to carry loads) and stiffness (i.e., structural deformations and loads that develop at constraints). The mechanical property variations result largely because of changes in the moisture content of the concrete constituents and progressive deterioration of the cement paste and aggregate (especially significant where thermal expansion values for cement paste and aggregate are markedly different). Significant deterioration of the concrete strength does not generally occur until the exposure temperature reaches ~400°C at which dehydration of calcium hydroxide occurs.[71] Reference 72 suggests that concrete exposed to temperatures of 90°C may lose only up to 10% of its room-temperature strength and modulus of elasticity values.* The response of concrete to elevated-temperature exposure depends on a number of factors (e.g., type and porosity of aggregate, rate of heating, permeability, moisture state, etc.). In addition to potential reductions in strength and modulus of elasticity, thermal exposure of concrete can result in cracking, or when the rate of heating is high and concrete permeability low, surface spalling can occur. Elevated temperatures also are important in that they affect the volume change and creep of concrete.[73] References 74–77 provide additional information on the effects of elevated temperature on concrete materials and structures.

Thermal cycling, even at relatively low temperatures (<65°C), can have some deleterious effects on concrete's mechanical properties (i.e., compressive, tensile and bond strengths, and modulus of elasticity are reduced). Most reinforced concrete structures are subjected to thermal cycling due to daily temperature fluctuations and are designed accordingly (i.e., inclusion of steel

---

* Except near certain penetrations carrying elevated temperature process fluids (e.g., steam lines), concrete temperatures are not expected to exceed 65°C.

reinforcement). At higher temperatures (200 to 300°C), the first thermal cycle causes the largest percentage of damage, with the extent of damage markedly dependent on aggregate type and is associated with loss of bond between the aggregate and matrix. Temperature variations, or thermal cycles, also can become important if the deformation of the structure resulting from the temperature variations is constrained.

A design-oriented approach for considering thermal loads on reinforced concrete structures is provided in ACI 349.1R.[78] Limited information on the design of temperature-resistant concrete structures is available through guides.[79,80] Currently available codes pertaining to NPP structures[17,20] generally handle elevated temperature applications by requiring special provisions (e.g., cooling) to limit the concrete temperature to ≤65°C, except for local areas where temperatures can increase to 93°C. These codes, however, do allow higher temperatures if tests have been performed to evaluate the reduction in strength and the reduction is applied to the design allowable.

Irradiation Irradiation in the form of either fast and thermal neutrons emitted by the reactor core or gamma rays produced as a result of capture of neutrons by members (particularly steel) in contact with concrete can affect the concrete. The fast neutrons are mainly responsible for the considerable growth, caused by atomic displacements, that has been measured in certain aggregate (e.g., flint). Nuclear heating occurs as a result of energy introduced into the concrete as the neutrons or gamma radiation interact with the molecules within the concrete material. Reference 81 indicates that nuclear heating is negligible for incident energy fluxes less than 10^{10} MeV/cm^2 per s. Gamma rays produce radiolysis of water in cement paste that can affect concrete's creep and shrinkage behavior to a limited extent and also result in evolution of gas. Prolonged exposure of concrete to irradiation can result in decreases in tensile and compressive strengths and modulus of elasticity. Irradiation has little effect on shielding properties of concrete beyond the effect of moisture loss due to temperature increase. Approximate threshold levels necessary to create measurable damage in concrete have been reported in limited research studies.[82] These levels are 1×10^{19} neutrons/square centimeter for neutron fluence and 10^{10} rads of dose for gamma radiation. Table 2.7 from Ref. 83 provides data for estimated radiation environments at the outside surface of LWR pressure vessels for a 1000 MW(e) plant operating at a capacity factor of 80%. Results indicate that radiation levels may approach the limits provided above in a concrete primary shield wall after 40 years of operation (32 equivalent full-power years). However, these values are upper limits and are probably higher than would be experienced because of the attenuating effects that would occur due to the presence of air gaps, insulation, etc., that could be positioned between the pressure vessel and concrete structures. Additional information on the interaction of radiation and concrete is available in Ref. 84.

Fatigue/Vibration Concrete structures subjected to fluctuations in loading, temperature, or moisture content (that are not large enough to cause failure in a single application) can be damaged by fatigue. Fatigue damage initiates as microcracks in the cement paste, proximate to the large aggregate particles, reinforcing steel, or stress risers (e.g., defects). Upon continued or reversed load application, these microcracks may propagate to form structurally significant cracks that can expose the concrete and reinforcing steel to hostile environments or produce increased deflections. Ultimate failure of a concrete structure in fatigue will occur as a result of excessive cracking, excessive deflections, or brittle fracture. Fatigue failure of concrete is unusual because of its good resistance to fatigue,[85,86] and concrete structures are designed using codes that limit design stress levels to values below concrete's endurance limit. However, as structures age, there may be instances of local fatigue damage at locations where reciprocating equipment is attached, or at supports for pipes that exhibit flow-induced vibrations.

Mild Steel Reinforcing Systems Mild steel reinforcing systems are provided in concrete structures to control the extent of cracking and the width of cracks at operating temperatures, resist tensile stresses and compressive stresses for elastic design, and provide structural reinforcement where required by limit condition design procedures.[20] Potential causes of degradation of the mild reinforcing steel are corrosion, elevated temperature, irradiation, and fatigue. Of these, corrosion is the factor of most concern for aging management of NPP structures. Information on the other potential degradation factors is provided for completeness and special situations that might occur.

Corrosion

Corrosion of steel in concrete is an electrochemical process that can assume the form of either general or pitting corrosion. Both water and oxygen must be present for corrosion to occur. The electrochemical potentials that form the corrosion cells may be generated in two ways: (1) composition cells formed when two dissimilar metals are embedded in concrete, such as steel rebars and aluminum conduit, or when significant variations exist in surface characteristics of the steel; and (2) concentration cells formed due to differences in concentration of dissolved ions in the vicinity of steel, such as alkalies, chlorides, and oxygen.[59] As a result, one of two metals (or different parts of the same metal when only one metal is present) becomes anodic and the other cathodic. Other potential causes of corrosion include the effects of stray electrical currents or galvanic action with an embedded steel of different metallurgy. The transformation of metallic iron to ferric oxide (rust) is accompanied by an increase in volume that can cause cracking and spalling of the concrete. In addition, corrosion will result in a reduction in effective steel cross-section and capacity. Depending on the source, local embrittlement may also be produced.

In good-quality, well-compacted concretes, reinforcing steel with adequate cover should not be susceptible to corrosion because the highly alkaline conditions present within the concrete ($pH > 12$) causes a passive iron oxide film to form on the iron surface (i.e., metallic iron will not be available for anodic activity). However, when the concrete pH falls below 11, a porous oxide layer (rust) can form on the reinforcing steel due to corrosion. Carbonation and the presence of chloride ions can destroy the passive iron oxide film.

Reduction of the concrete pH can occur as a result of leaching of alkaline substances by water or carbonation [i.e., calcium hydroxide is converted to calcium carbonate (calcite)]. The penetration of carbon dioxide from the environment is generally a slow process dependent on the concrete permeability, the concrete moisture content, and the carbon dioxide content and relative humidity of the ambient medium. Carbonation may be accelerated due to the concrete being porous (i.e., poor quality) or the presence of microcracks.

The passive iron oxide film on the steel reinforcement can also be destroyed by the penetration of chloride ions, even at high alkalinites ($pH > 11.5$). Maximum permissible chloride contents, as well as minimum recommended cover requirements, are provided in codes and guides.[16,67] For typical concrete mixtures, the threshold chloride content to initiate steel corrosion is about 0.6 to 0.9 kg $C\ell^{-}/m^3$ (Ref. 59). Chlorides may be present in concrete due to external sources (seawater effects, deicing salts, etc.) or may be introduced naturally into the concrete via aggregate or mix water transport. Furthermore, when large amounts of chloride are present, concrete tends to hold more moisture, which also increases the risk of steel corrosion by lowering concrete's electrical resistivity. Once the passivity of the steel is destroyed, the electrical resistivity of concrete and availability of oxygen control the rate of corrosion. Methods of excluding external sources of chloride ions from concrete are provided in Refs. 62 and 87. Detailed information on corrosion of steel embedded in concrete, its detection, and its repair, as well as the potential for stray electrical current corrosion to occur is available elsewhere.[88]

Elevated Temperature

The property of mild steel reinforcement of most importance to design is the yield strength. The reinforcing steel yield strength can be affected by elevated temperature exposure, but the temperatures to cause a strength decrease are significantly higher (except under unusual conditions) than would be experienced by a NPP concrete structure. Data for German reinforcing steels[89] indicate that for temperatures up to ~200°C, the yield strength is reduced by 10% or less, and at 500°C it falls to about 50% its reference room temperature value. Hot-rolled steels tend to resist the effects of temperature better than cold drawn or twisted steel. The steel modulus of elasticity exhibits similar reductions with increasing temperature. Other data[90,91] confirm the effects of temperatures above 200°C on the mild steel reinforcing as well as providing a threshold temperature of about 300°C for loss of bond properties with the concrete.

Irradiation

Neutron irradiation produces changes in the mechanical properties of carbon steels (e.g., increased yield strength and rise in the ductile-to-brittle transition temperature). The changes result from the displacement of atoms from their normal sites by high-energy neutrons, causing the formation of interstitials and vacancies. A threshold level of neutron fluence of 1×10^{18} neutrons per square centimeter has been cited for alteration of reinforcing steel mechanical properties.[92] As noted previously,[83] fluence levels of this magnitude are not likely to be experienced by the safety-related concrete structures in NPPs, except possibly in the concrete primary biological shield wall over an extended operating period.

Fatigue

Fatigue of the mild reinforcing system would be coupled with that of the surrounding concrete. The result of applied repeated loadings, or vibrations, is generally a loss of bond between the steel reinforcement and concrete. For extreme conditions, the strength of the mild steel reinforcing system may be reduced or failures may occur at applied stress levels less than yield. However, there have been few documented cases of fatigue failures of reinforcing steel in concrete structures and those published occurred at relatively high stress/cycle combinations.[85] Because of the typically low normal stress levels in reinforcing steel elements in NPP safety-related concrete structures, fatigue failure is not likely to occur.

Post-Tensioning Systems The post-tensioning systems used in NPPs are designed to have (1) consistently high strength and strain at failure, (2) serviceability throughout their lifetime, (3) reliable and safe prestressing procedures, and (4) ability to be retensioned and replaced (nongrouted systems). Potential causes of degradation of the post-tensioning systems include corrosion, elevated temperature, irradiation, fatigue, and loss of prestressing force. Of these, corrosion and loss of prestressing force are most pertinent from a NPP aging management perspective.

Corrosion

Corrosion of prestressing systems can be highly localized or uniform. Most prestressing corrosion-related failures involving general civil engineering structures have been the result of localized attack produced by pitting, stress corrosion, hydrogen embrittlement, or a combination of these. Pitting is the electrochemical process that results in locally intensified material loss at the tendon surface, potentially reducing the cross section to the point where it is incapable of supporting load. Stress corrosion cracking results in the fracture of a normally ductile metal or alloy under stress (tensile or residual) while in specific corrosive environments. Hydrogen embrittlement, frequently associated with hydrogen sulfide exposure, occurs when hydrogen atoms enter the metal lattice and significantly reduce its ductility. Hydrogen embrittlement also may occur as a result of improper application of cathodic protection to post-tensioning systems.[93-95] Failure of post-tensioning systems can also occur as a result of microbiologically-induced corrosion. Due to the stress state in the post-tensioning systems, the tolerance for corrosion attack is much less than for the mild steel reinforcement.

Elevated Temperature

The effect of elevated temperature on all heat-treated and drawn wires can be significant, and on cooling the wires may not regain their initial strength because the heating destroys the crystal transformations achieved by the heat-treating process. Short-term heating, on the order of 3 to 5 min., even to temperatures as high as 400°C, however, may not harm the prestressing wire's mechanical properties.[96] Results of a Belgian study[89] involving 30 types of prestressing steels indicate that thermal exposures up to ~200°C do not significantly reduce (< 10%) the tensile strength of prestressing wires or strands. References 91 and 97 support results of the Belgian study.

Elevated-temperature exposures also affect the relaxation and creep properties of prestressing tendons. Reference 98 indicates that losses in a 15.2-mm-diameter strand initially stressed to 75% of its guaranteed ultimate tensile strength at 40°C will be 5 to 6.4% after 30 years. Relaxation losses of tendons composed of stress-relieved wires are of about the same magnitude as stress-relieved strand, but relaxation of a strand is greater than that of its straight constituent wire because of the combined stress relaxation in the helical wires.[99] Creep (length change under constant stress) of stress-relieved wire is negligible up to 50% its tensile strength. Also, the creep effect in steel varies with its chemical composition as well as with mechanical and thermal treatment applied during the manufacturing process. As temperature levels experienced by the prestressing tendons in LWR facilities are below 200°C, the possibility for thermal damage to the prestressing steels under normal operating conditions is low.

Irradiation

Irradiation of post-tensioning system steel affects its mechanical properties because atoms are displaced from their normal sites by high-energy neutrons to form interstitials and vacancies. These defects can propagate or combine and effectively both strengthen the steel and reduce its ductility; or, at higher temperatures, they can recombine and annihilate each other and, for a given neutron dose, reduce the irradiation damage.[92] Results obtained from studies[92] in which 2.5-mm-diam prestressing wires were stressed to 70% of their tensile strength and irradiated to a total dose of 4×10^{16} neutrons per square centimeter (flux of 2×10^{10} neutrons$\cdot$cm$^2\cdot$per s) showed that for exposures up to this level, the relaxation behavior of irradiated and unirradiated materials was similar. These flux levels are higher than the level likely to be experienced in a LWR containment vessel.

Fatigue

Repeated reversals of stress, or variations in stress, applied to concrete structural elements (beams in particular) can result in fatigue failure in any of the following modes: (1) failure of the concrete due to flexural compression; (2) failure of the concrete due to diagonal tension or shear; (3) failure of the prestressing steel due to flexural, tensile-stress variations; (4) failure of pre-tensioned beams (grouted tendons) due to loss of bond stress; and (5) failure of the end anchorages of post-tensioned structures.[100] The majority of fatigue failures that occurred while testing prestressed concrete beams have resulted from fatigue of the tendons due to stress concentrations that occur in the tendon at a location where a crack occurs. In unbonded post-tensioned construction, the end anchorages could be subjected to some variation in stress under the action of changing external load, but unbonded tendons are not generally used in members subjected to frequent variations in stress. Reference 20 presents high-cycle and low-cycle dynamic tensile test requirements for prestressing tendon systems used in concrete containments.

Loss of Prestressing Force

Maintaining an adequate level of prestressing force in post-tensioned concrete containments is important to the overall safety of the NPP, especially during postulated accident conditions. Primary contributors to the loss of initial force level that was applied by the prestressing tendons include (1) friction, (2) end anchorage deflection (take up end slip), (3) elastic shortening, (4) tendon relaxation, and (5) concrete creep and shrinkage.[101–104] Of these factors, tendon relaxation and concrete creep and shrinkage are time-dependent factors and thus aging related.

Stress relaxation, defined as loss of stress (force) in the steel when the strain (elongation) does not vary, is related to tendon material properties, initial stress level, exposure temperature, and time. Creep and shrinkage of concrete represent volume changes of the concrete that occur over the life of the structure that can significantly affect the force levels in the tendons. Guidelines for developing surveillance programs acceptable to the USNRC and for providing reasonable assurance (when properly implemented) that the structural integrity of the containment is being maintained are provided in Regulatory Guides.[105,106] Reference 106 is a companion to Ref. 105 and provides clarification with respect to determination of prestressing forces and prediction of prestressing force losses over the service life of the structure.

Liner and Structural Steel Liner and structural steel members are subject to the same general degradation mechanisms as the steel reinforcement. Of these, corrosion and fatigue are of most importance to aging management. Except for structural steel members that assist in providing support for the reactor pressure vessels in certain plants (e.g., Trojan and Turkey Point), these members are generally not subjected to the effects of elevated temperature or irradiation.

Corrosion

The primary degradation factor for the liner plate and structural steel (both embedded sections and those within containment) is corrosion. Typically the liner plate and any installed steel are coated, either with a primer or a primer-finish coat system to prevent corrosion (e.g., zinc-rich primer with polyamide epoxy or modified phenolic coatings). Depending on the component, a corrosion allowance may also have been provided during the design stage. However, little allowance will have been provided for the relatively thin liner plate (i.e., ~6.3-mm thick).

The corrosion process that affects these components is similar to that for conventional reinforcing steel. Figure 2.13 presents a schematic representation of forms of corrosion that may be found on metals.[88] For liner plates, the influence of local attack that can lead to loss of leak tightness is of most concern. Local attack may result due to accumulation of moisture in areas experiencing loss of coating integrity, or failure of adjoining floor-liner sealant. The rate of attack may be rapid, depending on the aggressiveness of the environment. Reference 107 contains corrosion data for structural steel in numerous environments. For an industrial environment, the atmospheric (general) corrosion rate was found to be 0.02 to 0.04 mm/yr. This same reference reported pitting rates of 0.056 mm/yr for low carbon steels placed in polluted seawater. In general, depending on the environmental parameters, surface corrosion rates were noted to range from 0.001 mm/yr to 0.03 mm/yr.

Corrosion of structural steel piles, used in certain containment configurations for transferring foundation loadings to greater depths below grade, is also a possible degradation mechanism. Similar to other containment steel, the concern for piles is from localized corrosion resulting in significant loss of cross-sectional area. One study[108] examined corrosion data from 43 piling installations of varying depths (up to 41.5 m) with times of exposure ranging from 7 to 50 years in a wide variety of conditions. The conclusion of this study was that the type and amount of corrosion observed in steel pilings driven in undisturbed soil, regardless of soil characteristics and properties, was not sufficient to significantly affect the piling's performance as load-bearing structures. However, pilings placed in oxygen-enhanced fills, those exposed above grade, or those exposed to seawater or salt spray may be somewhat affected.[109]

Fatigue

The effects of repeated loads such as from polar crane operations or flow-induced vibrations may possibly detract from the function and performance of liner plate and structural steel members. The influence of repeated loads generally has been addressed at the design stage per national design codes. However, the effects of conditions outside of design predictions and local stress intensification points (material flaws, etc.) may result in fatigue-related problems. With respect to the liner plate, possible fatigue sites include base metal delaminations, weld defects, arc strikes, shape changes near penetrations, structural attachments, and concrete floor interfaces. For structural steel members (liner attachments and anchorages), the locations most susceptible to fatigue include large containment penetration framing (hatches, etc.) and liner anchorages near vibrating load conditions (such as those generated in structural attachments).

Anchorage Embedments Anchorage to concrete is required for heavy machinery, structural members, piping, ductwork, cable trays, towers, and many other types of structures. An anchorage might have to meet certain requirements for ease of installation, load capacity, susceptibility to vibration, preload retention, temperature range, corrosion resistance, post-installation or pre-installation, and ease of inspection and stiffness.[110] In meeting its function, loads that the anchor must transfer to the concrete vary over a wide combination of tension, bending, shear, and compression. Several potential factors related to failure or degradation of the anchorage systems include design detail errors, installation errors (improper embedment depth or insufficient lateral cover, improper torque), material defects (low anchor or concrete strengths), shear or shear-tension interaction, slip, and preload relaxation.[110] Aging effects that could impair the ability of an anchorage to meet its performance requirements would be primarily those that result in deterioration of concrete properties, because if a failure did occur, it would most likely initiate in the concrete.

2.3.2.2 Service life models

The actual safety and functional response of a structure in service depends on parameters chosen prior to construction (e.g., structural dimensioning and detailing, and choice of materials), and presumed parameters (e.g., environmental and loading conditions) that depend on subsequent service conditions that may be somewhat unpredictable. Prediction of the remaining service life* of concrete requires information on the condition of the concrete, major environmental stressors and aging factors, processes causing the deterioration, and rates of deterioration.[112] An integral part of structural safety and serviceability assessments is inspection that provides a link between the

* Service life is the period of time after construction during which all properties exceed the minimum acceptable values when routinely maintained; whereas, durability is the capability of maintaining the capacity of the structure to perform the function(s) for which it was designed and constructed.[111] Durability incorporates the concept of design requirements being met for a specific time and service life incorporates the concept of predicting the time that the design requirements will be met.

environmental conditions to which the structure is subjected and the manner in which it performs with time. Maintenance and repair in conjunction with a systematic inspection program can effectively extend the service life of structures. In-service inspection, repair, and the role of ISI and repair in extending the usable life of a structure are addressed in Chapter 5 of this report. Contained in the balance of this section is information primarily related to estimating the service life of in-service reinforced concrete structures.

Approaches for Estimating Service Life Approaches that have been 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) application of stochastic and reliability concepts, and (5) mathematical modeling based on the chemistry and physics of degradation processes. Often two or more of these processes may be used in combination. A description and critical review of these approaches has been completed.[113] The most promising approaches were considered to be accelerated testing, applications of stochastic and reliability concepts, and use of mathematical models.

Accelerated testing programs for determining concrete durability involve the use of elevated stressors (e.g., concentration of reactants, temperature, humidity, etc.) to expedite degradation. If the testing programs are properly designed, performed, and interpreted, they should provide a sound basis for estimating the concrete performance and service life. An important requirement for accelerated testing is that the degradation mechanism in the accelerated test be the same as responsible for in-service deterioration. Mathematical modeling is used to relate the accelerated degradation rate to the in-service rate. The performance of concrete exposed to frost (i.e., freeze-thaw cycles) or sulfate attack is frequently estimated using accelerated testing.[114,115]

Service life models using stochastic methods are based on the premise that service lives can not be precisely predicted because of the large number of factors involved, as well as their interaction.[116] Stochastic approaches include the reliability method, and the combination of statistical and deterministic models. The reliability method combines the principles of accelerated degradation testing and probabilistic concepts. The reliability method takes into account the broad distribution in times to failure exhibited by supposedly identical specimens tested under identical conditions. By changing a parameter such as applied stress, families of curves relating probability of failure to failure time can be developed. These curves can then be used to develop stressor (e.g., degradation factor) vs time-to-failure diagrams for different probabilities of failure.[117] Often the statistical models are combined with deterministic models. Carbonation of concrete and corrosion of embedded steel reinforcement have been addressed using the stochastic method.[116,118]

Most of the degradation processes for concrete are associated with concrete intrusion by water, ions, or gases. Mathematical models for estimating service life under the influence of these processes can be developed by considering (1) the rate of penetration of aggressive media into the concrete, and (2) the rate of chemical reactions and physical processes. References 119–122 describe service life models for degradation processes that are considered most likely to be applicable to NPP safety-related concrete structures (e.g., corrosion, sulfate attack, frost attack, and leaching). Service life models for degradation caused by radiation, salt crystallization, microbiological attack, and alkali-aggregate reactions do not appear to have been developed.

Service Life Methods Most Applicable to NPP Concrete Structures Estimation of the remaining service life of reinforced concrete structures in nuclear power facilities essentially involves determining the present condition of the concrete and then estimating the time required for concrete to degrade to the failure state. In estimating the time-to-failure, the specific process(es) responsible for the degradation needs to be identified and the degradation rates for the process(es) determined.

The amount of degradation of reinforced concrete is dependent on the environment, geometry of the structure, properties of the concrete, the specific degradation processes, and the concentration of the aggressive specie(s). Assume that these factors are invariant and thus can be represented by a constant, k_d, that does not change from year-to-year. Also assume that climatic variations from season-to-season smooth out over several decades so that only the number of years is represented by a time function, t_y. The cumulative amount of deterioration, A_d (e.g., loss of steel reinforcement section or depth of chloride ion penetration), at t_y can be represented by

$$A_d = k_d t_y^n \quad (2.1)$$

where n is the time order.[122] The overall rate of deterioration, R_d, is given by

$$R_d = n k_d t_y^{n-1}. \quad (2.2)$$

Equation (2.2) indicates that when $n < 1$, the rate of degradation decreases with time; when $n = 1$, the rate is constant; and when $n > 1$, the rate increases with time. Values of n that have been established for models of importance to potential degradation of NPP concrete structures are provided below:

Degradation Factor	**Time Order, n**
Sulfate Attack	
Wetting and Drying	1
Immersed	0.5
Corrosion	
Chloride Ion Diffusion	0.5
Active Corrosion	1
Acid Attack (Siliceous Aggregate)	0.5
Frost Attack	1

In general, $n = 0.5$ for diffusion-controlled processes, $n = 1$ for a reaction-controlled process, and n is between 0.5 and 1 if both convection and diffusion are involved in the transport of a reactive substance. Parameters for input into the particular degradation model ideally would come from inspections and testing of samples removed from the structure of interest. If the structure of interest is located in an area that can not be inspected or where samples cannot be removed for evaluation (e.g., environmental or accessibility constraints), values for material properties could be obtained from structures located elsewhere in the facility that were cast using the same materials and mix design. Other less desirable approaches would be to obtain material properties from specimens fabricated using the same materials, mix proportions, and curing conditions, or to obtain samples from facilities at other plants that have been shut down or decommissioned so that the areas of interest are accessible for inspection and sample removal.

Defining A_{df} as the amount of damage-at-failure, and rearranging Eq. (2.1) yields

$$t_{yf} = (A_{df}/k_d)^{1/n} \quad (2.3)$$

where t_{yf} is the time-to-failure. The estimated remaining service life is obtained by subtracting the age of the concrete when the inspection was made from t_{yf}.

Example Application of Service Life Models Survey data were obtained from reinforced concrete structures at several nuclear power stations in the United Kingdom ranging in age from 17 to 24 years after first commissioning — Windscale Advanced Gas-Cooled Reactor, Hinkley Point "A," Bradwell, and Wylfa.[123] One of the primary applications of this data was to provide input for the development of models to predict the onset of corrosion of the steel reinforcement at various locations in these structures. Current corrosion activity was assessed through half-cell potential and resistivity measurements. Core samples were removed from each of the structures and tested to determine the depth of carbonation and chloride ion content profiles — primary potential causes of corrosion of embedded steel at these facilities.

Corrosion of steel is recognized to be a two sage process (i.e., activation and propagation). The activation period is the time required for the depth of carbonation or the threshold level of chloride ions to reach the level of the steel and initiate the corrosion process. Once the corrosion process is activated, corrosion will propagate at a rate defined by the nature of the concrete and the environment, eventually resulting in cracking, staining, and spalling of the cover concrete. Most research has concentrated on the initiation stage of the corrosion process.

Carbonation results from the reaction of atmospheric carbon dioxide with the calcium hydroxide produced by cement hydration. It occurs most rapidly in relatively dry conditions (i.e., about 65% relative humidity) and results in the formation of calcium carbonate with a pH = 9 at which steel will corrode. Traditionally, the depth of carbonation, x, after an exposure time, t, is described by

$$x = kt^{1/2}. \tag{2.4}$$

The constant k is determined by the concentration of carbon dioxide in the atmosphere, its effective diffusivity through the concrete, and the amount of carbon dioxide consumed in the reaction with hydrated cement. As a result of a related research activity,[124] Eq. (2.4) was modified to accommodate the rapid early rate of carbonation in the near surface concrete layer as follows

$$x = A + kt^{1/2} \tag{2.5}$$

where A is primarily a function of the concrete mix proportions (i.e., compressive strength), but also includes exposure conditions [i.e., interior ($0 \leq A \leq 3$), sheltered exterior ($0 \leq A \leq 2$), and exposed exterior ($0 \leq A \leq 1$). Nomograms based on an extensive review of literature have been developed that correlate the k factor with concrete compressive strength and exposure condition (e.g., see Fig. 2.14).

Chlorides are generally assumed to migrate into concrete by a diffusion process and Fick's Second Law is assumed to apply as follows

$$\frac{\partial c}{\partial t} = D_c \cdot \frac{\partial^2 c}{\partial x^2}, \tag{2.6}$$

where C = chloride content by weight of cement,
t = time of exposure to chloride,
x = distance from concrete surface, and
D_c = diffusion coefficient of chloride.

If the surface chloride level is C_s and the level of chloride at depth x is C_x, then Eq. (2.6) can be solved to yield

$$C_x = C_s \left(1 - \operatorname{erf} \frac{x}{2\sqrt{D_{ce} t}} \right), \tag{2.7}$$

where D_{ce} is the effective diffusion coefficient of chloride in cover concrete and erf is the error function. Research noted previously[124] in conjunction with a study of old structures indicates that D_{ce} tends to decrease with age and varies through the depth of the cover concrete. In order to predict the time to activation of steel corrosion using Eq. (2.7), several factors must be defined [i.e., surface chloride content (C_s), effective diffusion coefficient of the cover zone (D_{ce}), and threshold level of chloride at which steel corrosion activates (C_a)]. Research[124] indicates that C_s reaches its ultimate value very quickly (i.e., within weeks), remains relatively constant thereafter if the exposure does not change drastically, and is determined in part by the particular concrete, Fig. 2.15. The effective diffusion coefficient, D_{ce}, at any point in time is determined by analysis of the chloride ingress profile. Figure 2.16 presents effective diffusion coefficients vs age for various concrete mixes based on results presented in Ref. 125. These results can be used to predict future values of D_{ce} based on a value determined at a known concrete age. The level of chloride at which steel corrosion activates, C_a, is generally assumed to be 0.4% by weight of cement. A lower value of C_a (e.g., 0.2%) can be used if more conservative estimates of service life are desired.

Results of the condition surveys at the United Kingdom nuclear power stations indicated that (1) depths of carbonation of the internal concrete were 50 mm or less, with coated concrete being much less susceptible to carbonation; and (2) levels of internal and external chloride ingress were generally low, being less than 0.05% by weight of cement in the surface 10 mm of concrete. Except for Wylfa, corrosion potential measurements generally indicated that there was no active corrosion. At Wylfa, the cooling water plant exhibited severe cracking, rust staining, spalling, and substantial loss of steel reinforcement section due to the harsh seawater environment. The carbonation depth and chloride profile data were input into Eqs (2.5) and (2.7), respectively, to estimate onset of steel corrosion at various locations in the stations. With the exception of Wylfa, estimates of minimum number of years to corrosion activation due to carbonation ranged from about 31 for an internal area near a heat exchanger at Windscale to over 140 in various areas of all other stations. Estimates of minimum number of years to corrosion activation due to chlorides were in excess of 100 for all locations examined in the stations except for Wylfa.

Needed Developments Quantitative design of reinforced concrete structures for durability requires an improved understanding of the degradation mechanisms, improved characterization of service environments, the development of advanced models, and the development of standards and guidelines of acceptance for durability estimations.[126,127] It has been suggested that a new set of codes and standards be required to include the interaction between the environment and applied loads in the estimation of service life.[128]

Extensive research and studies have been carried out to determine the durability of concrete under various service conditions (e.g., Refs. 129–132) and, thus, information on the progressive changes in the physical and chemical nature of concrete under such conditions is available. However, using this information to develop criteria and models for service life prediction is far from complete. Each concrete structure is somewhat unique due to variability in materials, geometry, construction practices, and environments experienced. Also, properties of the concrete have changed over the years (cement characteristics in particular). Furthermore, applications of the service life models to estimating the remaining service life of in-service concrete structures are few, especially when compared to applications for new concretes. If the value of n is known, the time order method [e.g., Eqs. (2.1) and (2.2)] is the most straight forward approach for estimating service life of in-service concrete when one process is rate controlling. Application of the time order method to specific degradation processes should be evaluated through additional modeling using field and experimental data. Also, the method requires additional study where more than one degradation process may be occurring simultaneously, with different time orders, but with no predominant rate-controlling process. Severe climatic changes, either transient or permanent, have not been addressed. Also, a technical basis does not exist for quantitatively estimating the effects of some degradation mechanisms (e.g., alkali-aggregate reactions) on the service life of concrete.

2.3.3 Long-Term Performance of Concrete Materials and NPP Structures

Data on the long-term performance of reinforced concrete materials and structures is of importance for demonstrating the durability of reinforced concrete structures in NPPs, and in predicting their performance under the influence of pertinent aging factors and environmental stressors. This information also has application to establishing limits on hostile environmental exposure for these structures and to development of inspection and maintenance programs that will prolong component service life and improve the probability of the component surviving an extreme event such as a loss-of-coolant accident.

Concrete, originally based on lime that hardened by atmospheric carbonation, has been utilized as a construction material for several thousand years. The oldest known concrete is from Yugoslavia and is about 7600 years old.[133] Gypsum mortars were used by the Egyptians to fabricate the Great Pyramid at Giza about 2500 BC. The Romans were the first to use hydraulic limes and discovered the benefits of pozzolans. The survival of several ancient concrete structures (e.g., Colosseum in Rome and Pont du Guard at Nîmes) attests to the durability that concrete can attain. A detailed study involving an examination of samples obtained from several ancient concrete structures utilizing physical and chemical techniques concluded that these structures survived primarily because of careful materials selection and construction, mild climatic conditions, and the lack of steel reinforcement.[133] These structures, however, were not fabricated using current hydraulic portland "cement" that was not in existence until about 1824. However, in Ref. 133 it was noted that samples were obtained for testing from several structures fabricated in the mid- to late 1800's. It was concluded that the durability of these structures was primarily due to high cement contents, but also the relatively slow cement-setting times and high construction quality. These portland cements differ somewhat from the portland cements used to fabricate NPP concrete structures in that the formulations have changed significantly as well as the fineness of the cement. Also, modern concretes have incorporated admixtures to improve workability, modify hardening or setting characteristics, aid in curing, and enhance the performance or durability. Results from the ancient and old portland cement-based concretes, however, do point out the importance to durability of material selection, good quality construction, and having adequate cementitious materials to produce dense concretes resistant to penetration by deleterious agents.

Prior reviews[13] of research conducted on concrete materials and structures indicated that only limited data are available on the long-term (40 to 80 years) properties of portland cement concretes, or on the examination of reinforced concrete structures that have an extended service time. Where concrete properties have been reported for conditions that have been well-documented, the results were generally for concretes having ages < 5 years, or for specimens that had been subjected to extreme, nonrepresentative environmental conditions such as seawater exposure or accelerated aging. Few investigations were reported providing results on examinations of structures that had been in service for the time period of interest, 20 to 100 years, and they did not generally provide the "high quality" information (e.g., baseline material characteristics and changes in material properties with time) that is desired for meaningful assessments to indicate how the structures have changed under the influence of aging factors and environmental stressors.

Over the last few years, however, data of use in continued service assessments and well-documented results from concrete specimens fabricated about 50 years ago and maintained under controlled conditions have become available (e.g., Ref. 134). Also, the testing of prototypical concrete specimens associated with the NPP structures (e.g., Ref. 123) and surveys to provide information on the condition of these structures has increased significantly as utilities have become interested in continuing the service of these facilities (e.g., Refs. 135 and 136). As a result, the Structural Aging (SAG) Program developed a Structural Materials Information Center containing information on the time variation of material properties under the influence of pertinent environmental stressors and aging factors. Also, information was assembled on the performance and condition assessments of the NPP safety-related concrete structures.

2.3.3.1 Structural materials information center (SMIC)

Results of a comprehensive review and assessment of existing material properties data bases indicated that a system of the type desired for use by the SAG Program did not exist (e.g., personal computer-based system that can easily access and retrieve material property data and information).[137] As a result, a plan was prepared for development of a data and information management system.[138] The plan was then implemented through development of the Structural Materials Information Center (SMIC).[139]

Description The SMIC has been developed in two formats — a *Structural Materials Handbook* and a *Structural Materials Electronic Data Base.* Since initial development, two updates have been prepared.[140,141] Provided below are general descriptions of the handbook and electronic data base.

Structural Materials Handbook

The *Structural Materials Handbook* has been developed as an expandable, hard-copy reference document that contains complete sets of data and information for each material in the SMIC. The handbook consists of four volumes that are provided in loose-leaf binders for ease of revision and updating. Volume 1 contains design and analysis information useful for structural assessments and safety margins evaluations (e.g., performance curves for mechanical, thermal, physical, and other properties presented as tables, graphs, and mathematical equations). Test results and data used to develop the performance curves in Vol. 1 are provided in Vol. 2. Volume 3 contains material data sheets that provide general information, as well as material composition and constituent material properties, for each material system contained in the handbook. Volume 4 contains appendices describing the handbook organization, as well as updating and revision procedures. Examples of pages that are contained in Vols. 1–3 that have been prepared for a long-term study on concrete properties (Ref. 134) are presented in Figs. 2.17–2.19, respectively.

Volumes 1, 2, and 3 of the handbook each contain four chapters of materials property data and information, with the chapters consistent between the volumes. Each material in the data base is assigned a unique seven-character material code that is used in the handbook and the electronic data base to organize materials with common characteristics. This code consists of a chapter index, a group index, a class index, and an identifier. The chapter index is used to represent the various material systems in the data base. The group index is used to arrange materials in each chapter into subsets of materials having distinguishing qualities such as common compositional traits. The class index is used to organize groups of materials with common compositional traits into subsets having a similar compositional makeup or chemistry. The identifier is used to differentiate structural materials having the same chapter, group, and class indices according to a specific concrete mix, ASTM standard specification for metallic reinforcement, etc. The arrangement of the material code parameters is shown in Fig. 2.20. Table 2.8 lists material code parameters that have been developed for the concrete and concrete-related materials presently in the data base.

A wide variety of information and materials property data has been collected and assembled for each material system included in the data base (e.g., general description, composition, mechanical property data, etc.). In setting up the data base, each material property has been identified by a unique four-digit property code selected from an established set of material properties categories. Table 2.9 lists the property code ranges and corresponding material property categories. Reference 139 presents a breakdown of the individual material property code values in each of these ranges.

Associated with each entry of data (numerical results of tests) or values (results of evaluation of data) into the data base is an assessment of the quality of the entries presented in the form of a letter grade. Although the criteria for assessing the quality of data and values are somewhat subjective, five quality levels have been developed. These levels are represented, in order of descending quality, by the letters A through E, Table 2.10. The 11 requirements utilized in the evaluation of the quality of data and values are listed in Ref. 139 along with specific criteria for each of the quality levels.

Each reference document that is used as an information source is assigned a unique integer identifier. In Vols. 1 and 2, reference numbers are listed to identify each information source, and all references that are used to develop a reported property for a particular material are provided in Vol. 3. Since each reference may be used for more than one property or structural material, a complete listing of references appears in Appendix E of Vol. 4. The integer identifier assigned to each reference source is consistent in both the handbook and the electronic data base.

Structural Materials Electronic Data Base

The *Structural Materials Electronic Data Base* is an electronically accessible version of the *Structural Materials Handbook.* It has been developed on an IBM-compatible personal computer using a commercially-available data base management system designed specifically for maintaining and displaying properties of engineering materials. To ensure that the handbook and electronic data base are compatible, each material included in the electronic data base is identified by the same common name and material code that has been used to represent the material in the handbook. Also, each electronic data base material record contains data and information taken directly from the handbook. Due to software limitations, the electronic data base is not as comprehensive as the handbook, but it does provide an efficient means for searching the various data base files to locate materials with similar characteristics or properties.

The electronic data base management system includes two software programs: Mat.DB (Ref. 142) and EnPlot (Ref. 143). Mat.DB is a menu-driven software program that employs window overlays to access data searching and editing features. This software is capable of maintaining, searching, and displaying textual, tabular, and graphical information and data contained in electronic data base files. Although Mat.DB has been developed for metallic materials, its formatting can be modified to accommodate nonmetallic and composite materials such as portland cement concretes. EnPlot is a software 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 generated with EnPlot can be entered directly into the Mat.DB data base files. These graphs are compatible with Microsoft Word, the work processing software used to prepare the handbook. The software was developed to run under Microsoft Windows Version 3.0 on IBM personal computers, or compatibles, using an Intel 80286 Processor, or higher, and DOS 3.1, or higher.

Material Property Data Bases Contained in the SMIC Two primary approaches have been utilized to obtain data and information for input into SMIC — open-literature references and testing of prototypical samples. A total of 144 data bases for concrete and concrete-related materials have been developed (i.e., 129 concrete, 12 metallic reinforcement, 1 prestressing steel, and 2 structural steel). Reference 141 provides a complete listing of the material property data bases contained in SMIC.

Examples of concrete material property data and information files currently available include compressive strength, modulus of elasticity and flexural strength vs time for several concrete materials cured under a variety of conditions (i.e., air drying, moist, or outdoor exposure) for periods up to 50 years; ultimate compressive strength and modulus of elasticity vs temperature at exposures up to 600°C for durations up to four months; weight loss vs time for specimens subjected to sulfuric acid concentrations (by weight) of 0.0016 to 0.02%; length change vs time for specimens subjected to wet (2.1% Na_2SO_4 solution) — dry cycling; and bond stress vs slip for reinforced concrete bond test specimens exposed for 14 days to either direct or alternating current (potential up to 20 volts). Metallic reinforcement (ASTM A 615 and A 15) performance curves are available for fatigue, and ambient and temperature-dependent (A 615 material only) engineering stress vs strain. Temperature-dependent engineering stress vs strain, and tensile yield strength, ultimate tensile strength, and ultimate elongation vs temperature performance curves are available for both prestressing tendon (ASTM A 421, Type BA) and structural steel (ASTM A 36) materials. Figure 2.21 presents information on the relative compressive strength of concrete specimens vs age for selected specimens tested under the University of Wisconsin[134] and Portland Cement Association[144,145] long-term concrete properties programs. The results presented have been normalized in terms of the 28-day reference concrete compressive strength and indicate the effect of mix design (i.e., water-cement ratio) and curing conditions (i.e., continuous moist, continuous air, inside laboratory environment, and outside environment). In general, the compressive strengths of the concretes tended to increase with age (e.g., the University of Wisconsin specimens having a water-cement ratio of 0.67 and continuously stored outside exhibited about a 180% increase in compressive strength at an age of about 25 years). The results presented in this figure indicate that two factors influenced the magnitude of increase in concrete compressive strength with age — relative value of 28-day reference strength and curing conditions. Specimens in each series that had the highest water-cement ratio generally exhibited the largest increases in compressive strength with age because of their lower 28-day reference strength values. Also, specimens for which moisture was available for continued cement hydration tended to exhibit greater increases in compressive strength with age. These results indicate that in estimating the increase in compressive strength with age of NPP concretes, the selection of a factor to apply to the reference 28- or 60-day reference strength should not be done "a priori." Although the strength increases experienced by the concretes in different locations in a NPP will vary somewhat due to different environments, the magnitude of increase in concrete compressive strength would not be expected to be nearly as great as that exhibited by some of the data in Fig. 2.21. The primary reason for this is that the reference strengths of NPP concretes would be relatively high (e.g., 27 to 40 MPa vs 12.3 MPa for the Wisconsin specimens exhibiting the 180% increase noted above). Also, the same factor should not be applied to all concrete structures in a NPP because of different environmental exposure conditions that would affect the strength gain (e.g., elevated temperatures and relative humidities).

Over the course of the SAG Program, a number of concrete samples were obtained from U.S. and U.K. nuclear power stations and tested. In the U.S., compressive strength tests were conducted on concretes from the Shippingport, EBR-II, Vallecitos, Palisades, Midland, Braidwood, Byron, Dresden, La Salle, Quad Cities, and Zion stations. In the U.K., results were obtained from specimens cast in conjunction with fabrication of the Wylfa, Heysham I, Heysham II, Hartlepool, Torness, and Sizewell "B" stations.[146] With the exception of the Wylfa station specimens, all the U.K. specimens had been initially cured under heat-cycled conditions to simulate the early in situ temperature rise due to cement hydration. The specimens were then continuously stored in a sealed, stable moisture state at temperatures from 10° to 95°C, with some under sustained loading (13.8 MPa) until tested. Figure 2.22 presents relative concrete compressive strength data obtained from the U.S. and U.K. data for which baseline data could be obtained. Results obtained from these tests indicate that although the concrete compressive strengths generally increased with age, the magnitude of increase for specimens tested at an age of about 25 years was 60% or less. Specimens obtained and tested from U.S. stations for which results were not presented in Fig 2.22 all exhibited concrete compressive strength values in excess of design requirements. These results could not be presented in the figure, however, because records providing the reference 28-day concrete compressive strength could not be located. This is likely to be a recurring problem at several of the older NPPs. Samples removed from these plants and tested will provide valuable information on the current concrete strength, but are of somewhat limited use for estimating future performance. Under ideal conditions it is desirable to have concrete compressive strength results obtained at several ages so that trending can be used to estimate future performance. One solution might be to periodically conduct tests on concrete materials that are removed as a result of plant modifications.

<u>Potential Applications of the SMIC</u> The structural integrity of a concrete structure can be evaluated using either a static load test or an analytical approach as outlined in Fig. 2.23.[147] For NPP concrete structures, the analytical approach is preferred due to such things as their massive size and accessibility restrictions.* Application of the analytical approach in the form of a theoretical stress analysis must take into account the physical characteristics of the structural members and their connection details, material properties, quality of construction, and the structure's current condition. Typically, two methods are available to obtain properties of the structural materials — nondestructive or destructive testing methods. When test methods are calibrated and certified and standardized procedures used, nondestructive testing can provide the required information. Destructive testing requires that samples be removed from the structure. Results from these tests in conjunction with visual examinations can be used to characterize the overall condition of a structure that is accessible. However, inspections, sampling, and nondestructive testing of all reinforced concrete structures in a NPP may not always be possible (e.g., primary biological shield or basemat). One potential approach for this situation is to use a comparative approach.

The comparative approach is based on the concept that the performance characteristics for materials in one structure can be estimated based on the results that are available on the performance of similar materials that have been exposed to similar service or environmental conditions (i.e., removal of samples for testing or application of nondestructive testing techniques is not required). Establishing material properties using the comparative approach requires a large knowledge base on material behavior under the influence of aging factors or environmental stressors such as provided by SMIC.

The comparative approach in conjunction with SMIC can be used to estimate current as well as future material properties.

---

* A structural integrity test is performed on the containment vessel prior to placing the plant in service and integrated leak-rate tests are periodically conducted that require pressurization of the containment. Ultimate strength evaluations would use the analytical approach.

Estimating Current Material Property Values A step-by-step procedure to estimate current material property values for use in a theoretical stress analysis of an existing concrete structure is provided below. This procedure should only be used as a guide, however, because each concrete structure has unique characteristics and features, and each theoretical stress analysis should be performed on a case-by-case basis.

1. Collect background information so that the structural materials used to construct the structure can be uniquely identified (Table 2.11).
2. Establish the service history (Table 2.11).
3. Identify material properties needed for the analysis (e.g., concrete strength and modulus of elasticity, and yield strength of metallic reinforcement).
4. Search SMIC to identify materials having similar compositions, characteristics, and exposure conditions (i.e., the *Electronic Data Base* can be searched based on either the seven-character material code or the four-digit property code).
5. From candidate materials provided, select properties for similar materials that reflect the service history of the structure being evaluated.
6. Estimate numerical values for each needed property by using the performance curves provided in Vol. 1 and the supporting documentation contained in Volume 2 of the *Structural Materials Handbook.*

Application of the above procedure can be illustrated through a hypothetical example. In this example, a utility is considering increasing the capacity of an overhead crane that is supported by reinforced concrete beams and columns that are part of a NPP safety-related concrete structure. In order to assess the feasibility of increasing the crane capacity, the current structural capacity of the reinforced concrete beams and columns is required. This entails an evaluation of the compressive strength of the concrete and the tensile yield strength of the reinforcing steel. One method would be to obtain material samples for destructive testing, but this is not feasible because the beams and columns are heavily reinforced and sample removal would degrade the structure. An alternate approach is to apply the six-step procedure noted above.

1. Collection and review of background information shows that the structures were fabricated 30-years ago using a normal-weight concrete (Table 2.12) and ASTM A15 No. 10 deformed (flexure and axial reinforcement) bars having a minimum yield strength of 276 MPa.
2. Review of plant records and operating logs indicates that the reinforced concrete has not been exposed to harsh environmental conditions. A visual inspection indicated no problems.
3. Compressive strength of the in situ concrete and modulus of elasticity and tensile yield strength of the steel reinforcement are required for the stress analyses.
4. Using information provided in Table 2.12, a search of the *Electronic Data Base* revealed three concretes having similar compositions and service histories to the concrete in question. Information on plain and deformed ASTM A 15 steel reinforcement was also located through a search of the *Electronic Data Base.*
5. Table 2.13 presents pertinent compressive strength information for the three concrete materials contained in SMIC.
6. The 30-year compressive strength value for the in situ concrete (41.7 MPa) is estimated by multiplying the 28-day value (24.4 MPa) by the ratio (1.71) of the average 30-year concrete strength to the average 28-day concrete strength for the similar concretes contained in SMIC. Since the steel properties do not vary significantly (if at all) with age, the original design properties were used.

Estimating Future Material Property Value A step-by-step procedure for estimating future material property values for use in theoretical stress analyses of an existing structure is provided below. Again, this information is provided only as a guide. This procedure has primary application to existing concrete structures that are required to remain in service for a specified time.

1. Establish current properties for structural materials using procedures described above, or destructive and nondestructive testing techniques can be used if feasible.
2. Establish the required continued service period, and estimate the service conditions for pertinent structural materials during this period.
3. Search SMIC to identify materials having similar compositions that were subjected to service conditions representative of that anticipated during the desired continued service period.
4. Establish numerical values for properties of interest for the pertinent structural materials using performance curves and supporting documentation obtained from the *Structural Materials Handbook.*

Application of the above procedure can be illustrated through another hypothetical example. In this example, a utility is investigating the possibility of annealing the reactor pressure vessel (RPV) at its NPP. In order to anneal the RPV steel, the temperature of affected parts of the RPV must be maintained at 445°C for approximately 168 hours. Results of a thermal analysis indicate that during the annealing operation, some concrete components located adjacent to the RPV could be exposed to temperatures up to 177°C. Prior to the annealing operation, an evaluation of the effects of the elevated temperature exposure on the performance of the reinforced concrete structures is required. The required analyses must reflect the current properties of the concrete materials as well as any changes in properties resulting from the elevated temperature exposure. A comparative approach is required to provide the required properties because the concrete is not accessible for visual examination, nondestructive examination, or removal of samples for testing. Application of the procedure described above can be used to obtain the required information for the concrete, metallic reinforcement, and structural steel materials that were used to fabricate the walls and floors adjacent to the RPV.

1. Collection and review of background information shows that the structures were fabricated 20-years ago using a normal-weight concrete (Table 2.12), various sizes of plain and deformed ASTM A 615 (Gr 60) steel reinforcement, and ASTM A 36 structural steel.
2. Review of plant records and operating logs indicates that the structural materials have not been exposed to harsh environmental conditions.
3. Using information provided in Table 2.12, a search of the *Electronic Data Base*, revealed three concretes having similar compositions and service histories (Table 2.13), and two similar concretes that had been exposed to elevated temperatures covering the magnitude and length of exposure of interest (Table 2.14). Ambient and elevated temperature results for the ASTM A 615 steel reinforcement and ASTM A 36 structural steel were also obtained from a search of the *Electronic Data Base* and are presented in Table 2.15.
4. Material property values just prior to the start of the annealing process are estimated. Procedures described above are used to estimate the current concrete compressive strength (i.e., 40.7 MPa). In this case, however, the multiplication factor (1.67) is determined using the 20-year old compressive strength data in Table 2.13. Steel material properties at this time are assumed to be time invariant because of the relatively benign service history of the structures.
5. Compressive strength of the concrete after the annealing operation is estimated by multiplying the estimated strength just prior to annealing (i.e., 40.7 MPa) by the ratio

(i.e., 0.59) of concrete strength after 177°C exposure for 7-days to the strength before exposure. Values for computing this ratio are provided in Table 2.14. The resulting estimated concrete compressive strength (i.e., 24 MPa) still exceeds the design compressive strength (i.e., 20.7 MPa) provided in Table 2.12. Pertinent material property values for the steel materials after the annealing process are provided in Table 2.15.

Recommendations for Continued Development of SMIC As noted previously, the SMIC contains 144 material property data bases. Reviews and assessments of data bases have indicated that SMIC is the only comprehensive information and data management system that has been developed specifically for the storage and retrieval of concrete and concrete-related materials, and in particular those materials that are representative of NPP concretes. As more and more information and data become available as a result of activities associated with continued service assessments of NPPs (e.g., testing of concrete core samples removed from the plants), it is recommended that this data and information continue to be incorporated into SMIC. Also, as SMIC has the capability to add other structural materials through additional files in the *Electronic Data Base* and chapters in the *Structural Materials Handbook*, it is recommended that SMIC be expanded to include information and data on other materials of importance to continuing the service of NPPs (e.g., reactor pressure vessel steels, structural steels, and sealants and coatings).

Also, as noted previously, Mat.DB, the data base management system for SMIC, was developed primarily for metallic materials. This has required the adaption of some of the information fields so that they could be used to present compositional information and time-dependent properties of composite materials such as portland cement concretes. Restrictions in the type of data that could be entered into a specific field were also encountered (e.g., concrete mixture proportions are reported in a field that was setup for percentage values ranging from 99.999 to 0.001 rather than units of mass per unit volume). This "force fitting" of information and data has resulted in a data base that new or occasional users may find objectionable or confusing. Due to software constraints and data-field limitations, the current version of Mat.DB (Version 1.22) is considered adequate only for examination of individual *Structural Materials Electronic Data Base* files and is not well suited for engineering evaluations in which properties for similar materials are combined and compared (e.g., data for only one material can be displayed on the computer screen at any given time). Table 2.16 presents a list of data base management system requirements considered necessary for storing and accessing materials property data and information at the SMIC.[148] Also identified in the table are some of the perceived weaknesses and limitations of Mat.DB (Version 1.22). A new version of Mat.DB (Version 2.0) based on Microsoft Windows (Ref. 149) is being developed that contains features making the software easier to use, but it is not anticipated to address all the requirements provided in Table 2.16.

Based on experience gained during development of the 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.[148] Data base management system software, computer hardware, and networking vs local operation were considered in the overall evaluation. Three classifications of data base management system software were considered: (1) standard — currently available software that can be used as-is to store and access properties of materials, (2) adaptable — currently available software that can be modified to accommodate the specific needs of the user, and (3) custom — software developed from "scratch" to user specifications using either commercially available data base management system development tools or a lower level computer language. Computer hardware requirements depend to some degree on the data base management system software selected and whether the software is operated locally or accessible using a wide-area network. Communication with a wide-area network requires a personal computer with a terminal emulator that is compatible with the operating system of the server that supports the data base management system software. Terminal emulators operate on almost any type of computer platform. When data base management system software is operated from individual systems that may or may not be connected to a local-area network, hardware such as IBM-compatible personal computers can be used. The greatest advantage of accessing a data base using a wide-area network is the ease with which updates and revisions can be distributed. Generally, only one copy of the data is produced and transmitted thus eliminating the need for mailing electronic media to individual users. Disadvantages of a wide-area network include relatively slow transmission rates over telephone lines and relatively high development and maintenance costs. The advantages of accessing a data base using a personal computer include faster data processing, that is especially valuable for graphical representations, and relatively low development and maintenance costs. The main disadvantage of using a personal computer is the increased distribution effort needed to mail updated electronic media to individual users. A feasible alternative to these two approaches is a combined approach in which distribution of updates is handled electronically and actual use of the system is performed using a personal computer. It was concluded from the assessment that custom software provides enough flexibility to satisfy the requirements presented in Table 2.16 and also permits entry of existing data and information files. Object-oriented relational data base software, such as Microsoft Access (Ref. 150), would provide the foundation for a new data base management system, and the data base could be completely designed and built locally. It is, therefore, recommended that in conjunction with the expansion and continuation of SMIC as a living document, a new data base management system be developed that incorporates the requirements in Table 2.16.

2.3.3.2 Longevity of NPP reinforced concrete structures

In general, the performance of NPP safety-related concrete structures has been very good. However, there have been several isolated incidences that if not remedied could challenge the capacity of the containment and other safety-related structures to meet future functional and performance requirements. Table 2.17 presents a summary of local degradation mechanisms that have been observed by one organization during condition surveys of various concrete structures at both U.S. and foreign NPPs located in areas having several different climatic conditions.[27] Some general observations derived from these results were that virtually all NPPs have experienced cracking of the concrete structures that exceeds typical acceptance criteria for width and length, numerous NPPs had groundwater intrusion occurring through the power block or other subsurface structures, few NPPs currently have a program for conducting periodic inspections of the concrete structures, and aging concerns exist for subsurface concrete structures as their physical condition cannot easily be verified. Collectively, it was concluded in this study that the general performance of the NPP concrete structures has been quite favorable, and proper evaluation and treatment of observed degradation at an early stage is both a cost effective and necessary approach to long-term plant operations. More specific results on the performance of U.S. and U.K. NPP concrete structures is provided below.

U.S. Experience Most of the instances related to degradation of NPP concrete structures in the U.S. occurred early in their life and have been corrected.[13,56] Causes were primarily related either to improper material selection and construction/design deficiencies, or environmental effects. Examples of some of the problems attributed to these deficiencies include low 28-d compressive strengths, voids under the post-tensioning tendon bearing plates resulting from improper concrete placement (Calvert Cliffs); cracking of post-tensioning tendon anchor heads due to stress corrosion or embrittlement (Bellefonte, Byron, and Farley); and containment dome delaminations due to low quality aggregate materials and absence of radial steel reinforcement (Crystal River) or unbalanced prestressing forces (Turkey Point 3).[151–153] Other construction-related problems have included occurrence of excessive voids or honeycomb in the concrete, contaminated concrete, cold joints, cadweld (steel reinforcement connector) deficiencies, materials out of specification, higher than code-allowable concrete temperatures, misplaced steel reinforcement, post-tensioning system buttonhead deficiencies, and water-contaminated corrosion inhibitors.[13]

Although continuing the service of a NPP past the initial operating license period is not expected to be limited by the concrete structures, several incidences of age-related degradation have been reported.[151–154] Examples of some of these problems include corrosion of steel reinforcement in water intake structures (San Onofre), corrosion of post-tensioning tendon wires (Fort St. Vrain), leaching of tendon gallery concrete (Three Mile Island), low prestressing forces (Ginna, Turkey Point 3, Zion, and Summer), and leakage of corrosion inhibitors from tendon sheaths (Palisades, Trojan, and Fort Calhoun). Other related problems include cracking and spalling of containment dome concrete due to freeze-thaw damage, low strengths of tendon wires, contamination of corrosion inhibitors by chlorides, and corrosion of concrete containment liners.

As the containment provides the final barrier between the NSSS and the outside environment, it contains radioactive material at elevated temperature in the unlikely event of an accident. Continuing the integrity of a containment that potentially can deteriorate under the action of various aging factors and environmental stressors, therefore is an essential aspect of NPP safety. Since post-tensioned concrete containments constitute the single largest type of containment structure and an in-depth evaluation of potential aging concerns of a post-tensioning system had not been performed, a study was conducted.[155] Primary aging mechanisms considered were corrosion, loss of prestressing force, and (potentially) loss of strength and ductility of the post-tensioning elements. Results provided by the study indicate that deterioration of the system hardware (including bearing zone concrete and sheathing filler) has not been significant. Water has occasionally been found in end caps that cover the anchor heads, but generally has been of no consequence or in a few instances has produced only minor surface staining of load-bearing components. Leakage of tendon sheathing filler (i.e., petroleum petrolatum wax type base material used to inhibit corrosion) has been observed at the end cap regions and on exterior concrete surfaces of several containments. The cause in the end cap region primarily has been defective gaskets/fittings, whereas filler on the concrete surface is believed to result from its migration under high head from the tendon duct seams through concrete cracks. This leakage generally has been accepted by the utilities as messy, but minor. Corrective actions have involved cleaning up the excess filler, replacement of the end cap gasket/fitting seals, and injection of additional filler into the ducts, if necessary. According to Ref. 156, this occurrence has been more prevalent in older plants that used an earlier version of sheathing filler (i.e., pre-1974) that had a lower melting point than the current product. Wire (or strand) strength and ductility do not appear to decrease with age under load. Tendon end anchorage forces* determined as part of examinations of the post-tensioning tendons conducted at regular intervals generally were above the time-dependent confidence limits; however, a few of the older containments have been found at or below these limits. Limited results obtained under this study tend to indicate that the minimum tendon forces obtained from end anchorage lift-off force measurements may exceed the actual prestressing force. Where end anchorage force measurements have been sufficiently low that tendon retensioning was required, it is recommended that the force in these tendons be monitored on a regular basis to determine how relaxation of retensioned tendons proceeds. The study concluded that there was essentially no evidence of physical deterioration of system hardware and that current examination programs[105,106,157] appear adequate to ensure the continuing physical integrity of post-tensioning systems. The one aging issue noted was the apparent discrepancy between the actual prestressing force and the force indicated by tendon lift-off load measurements. Potential contributors to this apparent discrepancy could be friction and

---

* Containment design criteria specify that the minimum force along the length of a tendon be sufficient to provide the required compressive force in the concrete. Since it is not feasible to measure this force, end anchorage force is used as a substitute for this value.

nonuniform prestressing forces along the tendon length. As the containments age, the significance of overestimation of the prestressing force takes on added importance because the losses may already be approaching the lower limit of acceptability (i.e., structural margins may be lower than indicated and unexpected cracking of the concrete can occur). Also, the impact of leakage of sheathing filler that has been observed at several plants should be evaluated to determine if there is an effect on mechanical properties of the concrete or structural performance (i.e., degradation of the bond that develops between the steel reinforcement and concrete).

U.K. Experience Condition surveys have been periodically conducted of U.K. nuclear power station concrete structures.[146,158,159] These surveys were part of a program to develop information on the likely durability of reinforced concrete components of buildings and structures at nuclear power stations over the currently-envisaged period of up to 100 years that includes completion of decommissioning. The studies included one Advanced Gas-Cooled Reactor and six Magnox plants that ranged in age from 13 to 26 years. Results from French nuclear stations at Chinon and Marcoule were also factored into the study. The condition surveys consisted of three primary elements: (1) visual inspection, (2) nondestructive testing (NDT) in situ, and (3) laboratory testing of retrieved samples. Pertinent results obtained from NDT in situ and laboratory testing of retrieved samples were described previously in this chapter. Visual inspections indicated that in general the components were in good condition, with the internal steel reinforcement in good condition and visible degradation of external concrete limited to a few localized areas of cracking and spalling due to corrosion of steel reinforcement or to exposed bolting. Little significant deterioration was expected throughout the 100 year period of interest for these structures provided adequate heating and ventilation was maintained in the buildings (e.g., $T \geq 15°C$ and relative humidity $\leq 50\%$). The most significant environmental stressor was determined to be corrosion of steel reinforcement from chlorides due to the close proximity of many of the U.K. stations to the coastline. Corrosion of the water intake structures due to chlorides was observed at one station (i.e. Wylfa). Carbonation depths and chloride ion profiles were obtained at selected locations and the results input into models to estimate the onset of corrosion (see previous discussion). A system for regular planned inspection and maintenance was proposed to monitor and protect against the processes causing corrosion. Inspections of the French stations indicated that the concrete structures, some dating from 1959, were generally considered to be in good condition. Some cracking and spalling was apparent in the externally-exposed concrete, however. Recently, through-wall corrosion of the liner of several of the French pressurized-water reactor plants has been observed due to failure of a seal at the lower floor-liner interface where the liner becomes embedded in the concrete.[160]

An investigation was conducted to develop additional data on longevity of NPP concrete structures.[161] Surveillance results for two prestressed concrete pressure vessels (PCPVs) each at the Wylfa, Hartlepool, and Heysham I stations were examined. Records reviewed included prestressing tendon anchorage lift-off load measurements, results of corrosion examinations, and visual examination results. Surveillance data for the Wylfa, Hartlepool, and Heysham I stations covered time periods since prestressing of 23, 14.5, and 13.4 years, respectively. It was concluded that performance of the PCPVs generally had been good. As expected, the tendon anchorage lift-off load measurements showed a trend for the loads to decrease with time due to a combination of prestressing steel relaxation and concrete creep. Examination of prestressing strands removed for inspection and testing revealed only a few minor, structurally insignificant pits indicating that the combination of waxes and greases used to inhibit corrosion of the ungrouted prestressing systems has been effective. Tensile test results for the prestressing strands exceeded design requirements. Visual examination of the concrete surfaces revealed a few surface cracks, with the cracks <0.30-mm-wide, and when active their growth rate was <20 microns per year. A comparison of concrete crack widths measured while the PCPVs were pressurized and unpressurized indicated that the changes in crack widths with pressure were insignificant. The cracks were associated with drying shrinkage.

2.3.3.3 Commentary on longevity

As concrete ages, changes in its properties will occur as a result of continuing microstructural changes (i.e., slow hydration, crystallization of amorphous constituents, and reactions between cement paste and aggregates), as well as environmental influences. These changes do not have to be detrimental to the point that concrete will not be able to meet its performance requirements. When specifications covering concretes production are correct and are followed, concrete will not deteriorate.[51] Concrete, however, can suffer undesirable changes with time because of improper specifications, a violation of specifications, or adverse performance of its cement paste matrix or aggregate constituents under either physical or chemical attack. Guidelines for production of durable concrete are available in national consensus codes and standards such as ACI 318[16] that have been developed over the years through knowledge acquired in testing laboratories and supplemented by field experience. Serviceability of concrete has been incorporated into the codes through strength requirements and limitations on service load conditions in the structure (e.g., allowable crack widths, limitations on midspan deflections of beams, and maximum service level stresses in prestressed members). Durability generally has been included through specifications for maximum water-cement ratios, requirements for entrained air, minimum concrete cover over reinforcement, etc.

Water is the single most important factor controlling the degradation processes of concrete, apart from mechanical deterioration (i.e., the process of deterioration of concrete with time is generally dependent on the transport of a fluid through concrete). The relationship between the concepts of concrete durability and performance is illustrated in Fig. 2.8 that was obtained from Ref. 52. The rate, extent, and effect of fluid transport are largely dependent on the concrete pore structure (i.e., size and distribution), presence of cracks, and micro climate at the concrete surface. The primary mode of transport in uncracked concrete is through the cement paste pore structure (i.e., its permeability). Although the coefficient of permeability for concrete depends primarily on the water-cement ratio and maximum aggregate size, it is influenced by the curing temperature, drying, and addition of chemical or mineral admixtures as well as the tortuosity of the path of flow. Concrete strength, although a reasonable indicator of potential durability under most scenarios, may not be sufficient. It is important also that adequate cementitious materials be included in the concrete mix to reduce its permeability.

In general, the performance of reinforced concrete structures in NPPs has been very good. Incidents of degradation reported generally occurred early in the life of the structures and primarily have been attributed to construction/design deficiencies, improper material selection, or environmental effects. Although the vast majority of these structures will continue to meet their functional and performance requirements during the current licensing period (i.e., nominally 40 years) as well as the continued service period being considered (i.e., 20 years), it is reasonable to assume that there will be isolated examples where the structures may not exhibit the desired durability (e.g., water intake structures at San Onofre) without some form of intervention. Aging concerns of most interest are primarily related to corrosion of steel reinforcement and liner materials, loss of prestressing force, and possibly the effects of tendon sheathing filler leakage. Since these structures have already been designed and constructed, outside of possibly the addition of barrier coatings and sealants to accessible structures to prevent ingress of hostile environments, the most prudent approach for maintaining adequate structural margins as well as extending usable life is through an aging management program that involves application of ISI and maintenance strategies. Figure 2.24 illustrates the relationship between structural performance, service life, and time, and the impact of ISI/repair activity.

Table 2.1. Typical safety-related concrete structures in LWR plants and their accessibility for visual examination.

Concrete Structure	Accessibility
Primary containment	
Containment dome/roof	Internal liner/complete external
Containment foundation/basemat	Internal liner (not embedded) or top surface
Slabs and walls	Internal liner/external above grade
Containment internal structures	
Slabs and walls	Generally accessible
Reactor vessel support structure (or pedestal)	Typically lined or hard to access
Crane support structures	Generally accessible
Reactor shield wall (biological)	Typically lined
Ice condenser dividing wall (ice condenser plants)	Lined or hard to access
NSSS equipment supports/vault structures	Generally accessible
Weir and vent walls (Mark III)	Lined with limited access
Pool structures (Mark III)	Lined
Diaphragm floor (Mark II)	Lined with limited access
Drywell/wetwell slabs and walls (Mark III)	Internal liner/partial external access
Secondary Containment/Reactor Buildings	
Slabs, columns, and walls	Accessible on multiple surfaces
Foundation	Top surface
Sacrificial shield wall (metallic containments)	Internal lined/external accessible
Fuel/Equipment Storage Pools	
Walls, slabs, and canals	Internal lined/partial external
Auxiliary building	Generally accessible
Fuel storage building	Generally accessible
Control room (or building)	Generally accessible
Diesel generator building	Generally accessible
Piping or electrical cable ducts or tunnels	Limited accessibility
Radioactive waste storage building	Generally accessible
Stacks	Partial internal/external above grade
Intake structures (inc. concrete water intake piping and canal embankments)	Internal accessible/external above grade and waterline
Pumping stations	Partially accessible
Cooling towers	Accessible above grade
Plant discharge structures	Internal accessible/external above grade and waterline
Emergency cooling water structures	Limited accessibility
Dams	External surfaces above waterline
Water wells	Limited accessibility
Turbine building	Generally accessible

Table 2.2. Typical safety-related concrete structures at BWR plants.

		Importance Factor*
A.	**Primary Containment**	
	Concrete Containment	
	1. Basemat Foundation	10
	2. Drywell Pedestal	10
	3. Vertical Walls (Mark I)	8
	4. Steel Liner	4
	5. Suppression Chamber (Mark I)	8
	6. Chamber Steel Liner (Mark I)	4
	7. Vertical Walls (Mark II)	8
	8. Vertical Walls (Truncated Cone – Mark II)	8
	9. Concrete Dome (Mark III)	8
	10. Polar Crane Support (Mark III)	10
	Steel Containment	
	1. Basemat Foundation	10
B.	**Containment Internal Structures**	
	1. Bottom Slab (Steel Mark I and Pre-Mark Containments)	10
	2. Reactor Pedestal/Support Structure	9
	3. Biological (Reactor) Shield Wall	6
	4. Floor Slabs	6
	5. Walls	7
	6. Columns	7
	7. Diaphragm Floor (Mark II)	7
	8. NSSS Equipment Pedestals/Supports	5
	9. Upper and Fuel Pool Slabs (Mark III)	5
	10. Drywell Wall (Mark III)	6
	11. Weir/Vent Wall (Mark III)	7
	12. Crane Support Structure (Mark III)	5
C.	**Secondary Containments/Reactor Buildings**	
	1. Basemat Foundation (if isolated from containment basemat)	10
	2. Walls	8
	3. Slabs	6
	4. Columns	7
	5. Equipment Supports/Pedestals	4
	6. Sacrificial Shield Wall (Metal Containments)	6
	7. Spent/New Fuel Pool Walls/Slabs	6
	8. Drywell Foundation (Mark I)	9
D.	**Other Structures (Category I)**	
	1. Foundations[a]	10
	2. Walls[a]	8
	3. Slabs[a]	6
	4. Cable Ducts	5
	5. Pipe Tunnels	5
	6. Stacks	5
	7. Concrete Intake Piping	6
	8. Cooling Tower Basins	5
	9. Dams	6
	10. Intake Crib Structures	6
	11. Embankments	4
	12. Tanks	5
	13. Water Wells	4

* See Chapt. 4 for description and use of importance factors.

Table 2.3. **Typical safety-related concrete structures at PWR plants.**

		Importance Factor*
A.	**Primary Containment**	
	Concrete Containment	
	1. Basemat Foundation	10
	2. Tendon Access Galleries	3
	3. Vertical Walls (and Buttresses)	8
	4. Ring Girder (Prestressed Concrete Containment Vessel)	9
	5. Dome	8
	Steel Containment	
	1. Basemat Foundation	10
B.	**Containment Internal Structures**	
	1. Bottom Floor (Metal Containments)	10
	2. Floor Slabs	6
	3. Walls	7
	4. Columns	6
	5. NSSS Equipment Pedestals/Supports	5
	6. Primary Shield Wall (Reactor Cavity)	8
	7. Reactor Coolant Vault Walls	7
	8. Beams	5
	9. Crane Support Structures	4
	10. Ice Condenser Divider Wall and Slab	5
	11. Refueling Pool and Canal Walls	6
C.	**Secondary Containment Buildings (metal containments)**	
	1. Foundation	10
	2. Walls	8
	3. Slabs	6
D.	**Other Structures (Category I)**	
	1. Foundations[a]	10
	2. Walls[a]	8
	3. Slabs[a]	6
	4. Cable Ducts	5
	5. Pipe Tunnels	5
	6. Stacks	5
	7. Concrete Intake Piping	6
	8. Hyperbolic Cooling Towers	5
	9. Dams	6
	10. Intake Crib Structures	6
	11. Embankments	4
	12. Tanks	5
	13. Water Wells	4

* See Chapter 4 for description and use of importance factors.

[a] Components of other site buildings such as Auxiliary, Turbine, Control, and Diesel Generator Buildings.

Table 2.4. **Degradation factors that can impact the performance of safety-related concrete structures.**

Material System	Degradation Factor	Primary Manifestation
Concrete	Chemical Attack	
	Leaching and efflorescence	Increase porosity
	Sulfate attack	Volume change/cracking
	Acids and bases	Increased porosity/erosion
	Alkali-aggregate reactions	Volume change/cracking
	Physical Attack	
	Salt crystallization	Cracking
	Frost attack	Cracking/spalling
	Abrasion/erosion/cavitation	Section loss
	Thermal exposure/thermal cycling	Cracking/spalling/strength loss
	Irradiation	Volume change/cracking
	Fatigue/vibration	Cracking
Mild Steel Reinforcement	Corrosion	Concrete cracking/spalling
	Elevated temperature	Decreased strength
	Irradiation	Reduced ductility
	Fatigue	Bond loss
Post-Tensioning	Corrosion	Section loss/capacity loss
	Elevated temperature	Reduced strength
	Irradiation	Reduced ductility
	Fatigue	Concrete cracking
	Stress relaxation/end effects	Prestress force loss
Liner/Structural Steel	Corrosion	Section loss
	Fatigue	Cracking
	Elevated temperature	Reduced strength
	Irradiation	Reduced durability

Table 2.5. Locations in NPPs where concrete-related materials may exhibit degradation.

Material System	Degradation Factor	Potential Areas of Deterioration
Concrete	Chemical Attack	• Subterranean areas • Surfaces exposed to cooling water or decontamination fluids • In-containment floors and slabs subject to chemical spills • Containment/shield, auxiliary building structures (ocean atmosphere, alkali-aggregate reaction)
	Freeze/Thaw Cycling	• External structures where water may collect • Intake/discharge structures, particularly at water line of cooling water source
	Thermal Exposure/Thermal Cycling	• Containment/shield structures (diurnal and seasonal effects) • Biological shield • Areas located near reactor coolant pressure boundary or hot penetrations
	Irradiation	• In-containment structures proximate to reactor coolant pressure boundary (e.g., biological • Localized areas of specific containment designs
	Abrasion/Erosion/Cavitation	• Floor and slab elements • Cooling water intake or discharge structures
	Fatigue/Vibration	• Local areas in containment (e.g., near liner anchors) • Local areas at equipment supports or piping vibrations
Mild Steel Reinforcement	Corrosion	• Outer layer of conventional steel reinforcing in all structures
	Irradiation	• In-containment structures proximate to reactor coolant pressure boundary (e.g., biological shield)
Post-tensioning Systems	Corrosion	• Containment buildings
	Relaxation/concrete creep and shrinkage	• Containment buildings
Liner/Structural Steel	Corrosion	• Areas of moisture storage or accumulation • Areas exposed to chemical or borated water spills
	Thermal exposure/thermal cycling	• Hot penetrations
	Fatigue	• Discontinuities and equipment supports • Penetrations subject to cyclic thermal and mechanical loads.

Table 2.6. **Reactivity of various chemicals with portland cement concrete and reinforcing steel/liner plates.**

Material	Reactivity Effect on Concrete	Reactivity Effect on Reinforcing Steel/Liner Plate
Acetone	Liquid loss by penetration (may cause slow disintegration)	None
Acidic Water (less than 6.5 pH)	Disintegrates concrete slowly	May attack rebar and embedments
Boric Acid	Negligible effect unless immersed	Severely corrosive to liner and reinforcing steel
Borated Water (and boron)	Negligible effect unless immersed	Very corrosive at high concentration
Chlorine Gas	Concrete (moist) slowly disintegrates	Highly corrosive
Deicing Salts	Scaling of non-air entrained concrete	Highly corrosive
Diesel Exhaust Gases	May disintegrate moist concrete by action of carbonic, nitric, or sulphurous acid; minimal effect on hardened dry concrete	Minimal
Formaldehyde (formic acid)	Disintegrates slowly	Minimal
Hydrochloric Acid	Disintegrates concrete rapidly	Highly corrosive
Hydroxides	At low concentrations, slow disintegration; at high concentrations, greater disintegration	Unknown
Lubricating Oil	Fatty oils, if present, slowly disintegrate concrete	Minimal
Seawater	Disintegrates concrete with inadequate sulfate resistance	Highly corrosive
Sodium Hydroxide	Not harmful below 20% concentration, disintegrates at concentrations above 20%	Minimal
Sodium Pentaborate	Disintegrates at varying rates depending on concentration	Dependent on concentration
Sulfates	Disintegrates at varying rates with concentration (concretes with low sulfate resistance such as Type I Portland cement concrete)	Harmful at certain concentrations
Sulphuric Acid (sulphurous)	Disintegrates rapidly in concentration between 10 and 80%	Harmful

Table 2.7. Estimated irradiation levels at the outside boundary of a RPV for several operating periods.

	BWR*			PWR*		
	40 Year (32 EFPY)	60 Year (48 EFPY)	80 Year (64 EFPY)	40 Year (32 EFPY)	60 Year (48 EFPY)	80 Year (64 EFPY)
Neutron Fluence (n/cm^2)						
Slow ($E < 1.0$ MeV)	3.7×10^{18}	5.6×10^{18}	7.5×10^{18}	2.0×10^{19}	3.0×10^{19}	4.0×10^{19}
Fast ($E > 1.0$ MeV)	5.1×10^{17}	7.7×10^{17}	1.0×10^{18}	1.0×10^{18}	1.5×10^{18}	2.0×10^{18}
Gamma Total Integrated Dose (rads)	1.6×10^{10}	2.4×10^{10}	3.2×10^{10}	4.7×10^{9}	7.0×10^{9}	9.3×10^{9}

* 1000 MW(e) plant with 80% capacity factor.
EFPY = effective full-power years.
Source: Copyright © 1977. Electric Power Research Institute. EPRI NP-152. *PWR and BWR Radiation Environment for Radiation Damage Studies*. Reprinted with Permission.

Table 2.8. Material code identification and description.

Portland Cement Concretes Handbook Chapter Index 01 – Electronic Data Base File CONCRETE.DB			
Group Index	Group Index Description Type of Concrete	Class Index	Class Index Description Type of Aggregate
A	Insulating	A	Stone
B	Structural Lightweight	B	Gravel
C	Normal-Weight	C	Manufactured or By-produce
D	Heavyweight		
Metallic Reinforcements Handbook Chapter Index 02 – Electronic Data Base File REBAR.DB			
Group Index	Group Index Description Type of Reinforcement	Class Index	Class Index Description Characteristic Feature
A	Carbon Steel Bars	A	Uncoated without Deformations
B	Stainless Steel Bars	B	Coated without Deformations
C	Steel Wires	C	Uncoated with Deformations
D	Bar Mats/Wire Fabric	D	Coated with Deformations
Prestressing Tendons Handbook Chapter Index 03 – Electronic Data Base File TENDON.DB			
Group Index	Group Index Description Type of Tendon	Class Index	Class Index Description Characteristic Feature
A	Carbon Steel Bars	A	Materials without Deformations
B	Carbon Steel Wires	B	Materials with Deformations
C	Strand		
D	Nonmetallic Materials		
Structural Steels Handbook Chapter Index 04 – Electronic Data Base File STEEL.DB			
Group Index	Group Index Description Type of Reinforcement	Class Index	Class Index Description Characteristic Failure
A	Carbon Steels	A	Hot- or Cold-Rolled Steels
B	Stainless Steels	B	Bolting Materials
		C	Special Materials

Table 2.9. **Property code range descriptions.**

Property Code Ranges	Property Code Range Description
1000-1999	General Information
2000-2999	Constituent Material and Plastic Concrete Properties
3000-3999	Mechanical Properties
4000-4999	Thermal, Physical, and Other Properties
5000-9999	Available for Data Base Expansion

Table 2.10. Quality level definitions.

Quality Levels and Corresponding Term Definitions		
Quality Level	Quality Level Term Description	Relative Quality Level Rating
A	Recommended Property	Highest
B	Selected Property	
C	Typical Property	
D	Provisional Property	
E	Interim Property	Lowest

Table 2.11. Background information and service history requirements for portland cement concretes and embedded metallic materials.

Background Information		
	Portland Cement Concrete	Embedded Metallic Materials
Material Description	Lightweight Structural Lightweight Normal-Weight Heavy-Weight	Metallic Reinforcement Prestressing Tendon Structural Steel
Material Identification	Common Name or Designation	Material Specification (including Type, Class, Grade, etc.)
Baseline Properties	28-Day Compressive Strength (or other reference properties) Specified Design Strength	Yield Strength or Ultimate Tensile Strength
Material Composition	Constituent Materials Mixture Proportions Petrographic Reports	Mill Certificate Chemical Analysis Inspection Reports
Plastic Concrete Properties	Water-Cementitious Materials Ratio Cement Content Air Content Unit Weight	Not Applicable
Service History		
	Portland Cement Concrete	Embedded Metallic Materials
Physical Location	Inside/Outside Above/Below Grade Submerged Soil and Groundwater Conditions	Inside/Outside Above/Below Grade Submerged Soil and Groundwater Conditions
Natural Environment	Temperature Extremes Freeze-Thaw Cycles Weather Conditions	Temperature Extremes Weather Conditions
Normal Operating Environment	Temperature Range Humidity Range Radiation Exposure Chemical Exposure Electrical Currents	Temperature Range Radiation Exposure Chemical Exposure Electrical Currents
Abnormal Exposure Conditions	Fire Chemical Spills Abnormal Temperature Exposure	Fire Chemical Spills Abnormal Temperature Exposure

Table 2.12. Mix proportions, baseline information, and plastic concrete properties for the portland cement concrete used in the examples.

Mix Proportions			
Property Code 1210	Material Composition		
Constituent Material	Mix Proportions per Unit Volume		Property Code
	kg/m³	lb/yd³	
Portland Cement ASTM C 150, Type I	279	470	2001
Fine Aggregate	653	1100	2211
Coarse Aggregate			
Maximum Size 38 mm (1-1/2 in.)	1246	2100	2221
Water	139	235	2421
Total	2317	3905	
Baseline Information			
Property Code 1310	Baseline Information		
Property Description	Data or Value		Property Code
	MPa	psi	
Design Compressive Strength	20.7	3000	3021
Average Compressive Strength at 28 Days	24.4	3540	3022
Plastic Concrete Properties			
Property Code 2600	Plastic Concrete Properties		
Property Description or Designation	Property Value		Property Code
Cement Content	279 kg/m³	470 lb/yd³	2601
Water-Cement Ratio	0.50	0.50	2603
Air Content	1.0%	1.0%	2613
Unit Weight	2317 kg/m³	144.6 lb/ft³	2621
Slump	76 mm	3.0 in.	2631

Table 2.13. Time-dependent compressive strength values for portland cement concretes obtained from SMIC.

Material Code	Compressive Strength, MPa 28-day	20-year	30-year
01CB002	19.2	35.9	37.8
01CB003	12.3	25.5	26.1
01CB007	33.6	47.3	47.4
Average =	21.7	36.2	37.1

Table 2.14. Temperature-dependent compressive strength values for portland cement concretes obtained from SMIC.

Material Code	Compressive Strength, MPa Before Exposure	After Exposure to 177°C for 7 days
01CA004	42.8	26.6
01CA005	63.3	35.7
Average =	53.1	31.2

Table 2.15. Ambient and temperature-dependent properties for ASTM A 615 (Gr 60) and A 36 steels.

	Property Value at			
	ASTM A 615 Steel		ASTM A 36 Steel	
Material Property	Ambient	177°C	Ambient	177°C
Modulus of Elasticity, GPa	200	191	200	191
Tensile Yield Strength, MPa	414	414	248	220
Ultimate Tensile Strength, MPa	621	621	621	420
Elongation, %				
Plates and Bars	—	—	20	15
Shapes	—	—	23	17
#3, #4, #5 Bars	9	9	—	—
#7, #8 Bars	8	8	—	—
#9, #10, #11, #14, #18 Bars	7	7	—	—
Thermal Coefficient of Expansion, mm/mm/°C	11.7×10^{-6}	11.3×10^{-6}	11.7×10^{-6}	11.3×10^{-6}

Table 2.16. Requirements for a data base management system for the SMIC.

	Data Base Management System Requirement	Mat.DB, Version 1.22 Perceived Weaknesses and Limitations
a.	Each material and its associated properties should be represented in the handbook and the electronic data base using the same format. In addition, handbook pages should be printed automatically using information and data stored in the electronic data base. These two capabilities would eliminate the need for double entry of data and information and for maintaining the data base in two separate formats. This would ensure that both presentation formats were compatible and would greatly reduce the effort required to reproduce and distribute hard copies of the data base.	Does not exist in Mat.DB
b.	The data base management system needs to accommodate variable-length field names so that complete material names, property designations, and other types of descriptive information can be recorded. "Material Codes" and "Property Codes" are used in both the handbook and electronic data base to identify the various materials and to distinguish one property from another (Refs. 139 and 140). These codes are used as an indexing system for organizing the handbook and provide a convenient way to subdivide materials and properties into common groups. However, there is no inherent reason for displaying code abbreviations on the computer screen when corresponding text could be inserted as appropriate. The ability for the end user to edit these names could potentially be desirable.	Codes and other cryptic abbreviations are used in the *Structural Materials Electronic Data Base* because Mat.DB has limited space for representing this type of information. This limitation makes the data base confusing to use and somewhat difficult to learn because the codes are only identified and defined in the *Structural Materials Handbook*. The field names used in Mat.DB are not always appropriate for all types of materials and in particular concrete. Adapting to this limitation occasionally introduces interpretation problems for the end user.
c.	The ability to simultaneously display multiple windows of tabular or graphical data is a desirable data base management system characteristic. This feature would greatly enhance the usefulness of the data base because it would provide a way for the end user to compare the same types of properties and information for materials with similar compositions or characteristics. This capability would also allow the end user to superimpose curves from different materials onto the same plot making comparison of time- and environment-dependent performance relatively easy and accurate.	A new version of Mat.DB is currently being developed using Microsoft Windows (Ref. 149). This version is expected to include the ability to overlay multiple windows of data, but superimposing graphs may not be possible.
d.	Material composition is an important data base parameter that needs to be represented accurately and precisely. While dimensionless units such as percent may be suitable for reporting the composition of metallic materials, units of mass per unit volume may be required for composite materials such as concrete. The data base management system must be able to accommodate a broad range of units for reporting material composition. Representing these units in both the International System of Units (SI) and customary units is also desirable.	Units of percent are the only ones available in Mat.DB for reporting the composition of materials. This limitation is particularly inconvenient for concrete in which mixture proportions are typically reported as mass per unit volume.

Table 2.16. (Cont'd)

	Data Base Management System Requirement	Mat.DB, Version 1.22 Perceived Weaknesses and Limitations
e.	The data base management system needs to be capable of storing and displaying mathematical equations in such a way that they can be used to construct tables and generate graphical representations of performance curves. The performance curves reported in the *Structural Materials Handbook* were developed from test results or synthesized from minimum property values. These curves are one of the most important features in the data base because they provide the basis for comparing time- and environment-dependent properties for different materials. In order for this feature to be interactive, the equations must be solved in real time and the results displayed upon demand.	This capability does not currently exist within Mat.DB. The graphs included in the *Structural Materials Electronic Data Base* are simply pictorial representations that were developed using EnPlot, and the property values presented in the spreadsheets were entered as numerical values. Mat.DB was not designed to accommodate mathematical equations.
f.	The data stored in a data base management system need to be used to construct tables and create graphs. This feature is essential for limiting files to a manageable size and economizing data input efforts	The same data and values that are used by EnPlot to prepare engineering graphs must also be entered into Mat.DB. These two programs do not share a common data file.
g.	Customized help features are needed to assist the end user by answering fundamental questions associated with terminology such as differences between various data categories and material designations. Suggested guidelines for using material properties data, and information are also considered necessary so that the end user can take full advantage of the data base and its features. Help files are often necessary to clarify terminology, prevent misuse, and enhance the significance of the reported data and information.	Customized prompt text files can be displayed using Mat.DB utility features, but these files only provide information that enhances the identify of reported data and values. These fields do not contain information that provides guidance to the end user.
h.	Notes are a very important part of any materials property data base. The ability to search the notes for keywords and phrases is considered necessary.	Recent advances in data base technology have made this feature possible, but the current version of Mat.DB does not have this capability.

Table 2.17. Condition survey results for various NPP concrete structures.

Local Degradation Mechanisms	Plant									
	A	B	C	D	E	F	G	H	I	J
<u>Concrete</u>										
Chemical Attack	b,c	c	b	c	c	c	c	c		
Efflorescence and Leaching		b,c,d	b,c		b,d	b,d	d	b,d,f	a,b, c,d	b,f
Alkali-Aggregate Reaction										a
Freeze/Thaw Cycling	d			a,d			d	f		
Thermal Exposure			c	c	c			c		
Abrasion/Erosion			c				c,d			
Fatigue/Vibration		c								
Cracking	c,d, f,g	a,b, c,d	c,d,g	c,d	a,b, c,d,g	b,e, d,f,g	b,f	b,c, d,f	b,c, d,f	b,f,g
<u>Conventional Reinforcing</u>										
Corrosion	b,d	b,d	b		b,d	b		b,d	b	b,f
<u>Prestressing System</u>										
Corrosion	n/a	e[1]	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
<u>Block Walls</u>										
Excessive Cracking		c			d		c	a		
<u>Structural Steel and Liners</u>										
Corrosion	d	e	c,d				c,e		e	g
<u>Soil/Structure Issues</u>										
Differential Settlement	c									
Soil Erosion (Scour)	d									

Key:

a – External Structure (Power Block)
b – Subgrade Structure (Power Block)
c – Internal Structure (Power Block)
d – Water Control Structure (Intake, Discharge, Etc.)
e – Containment Vessel
f – Other Site Structure
g – Equipment Supports

Notes:

1. Corrosion limited to exposed grease can and bearing plate surfaces (no tendon corrosion noted).

Source: F. E. Gregor and C. J. Hookham, "Remnant Life Preservation of LWR Plant Structures," *Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology held August 15–20, 1993*, in Stuttgart, Germany, Elsevier Science Publishers, Amsterdam, The Netherlands, 1993; reprinted with permission from author.

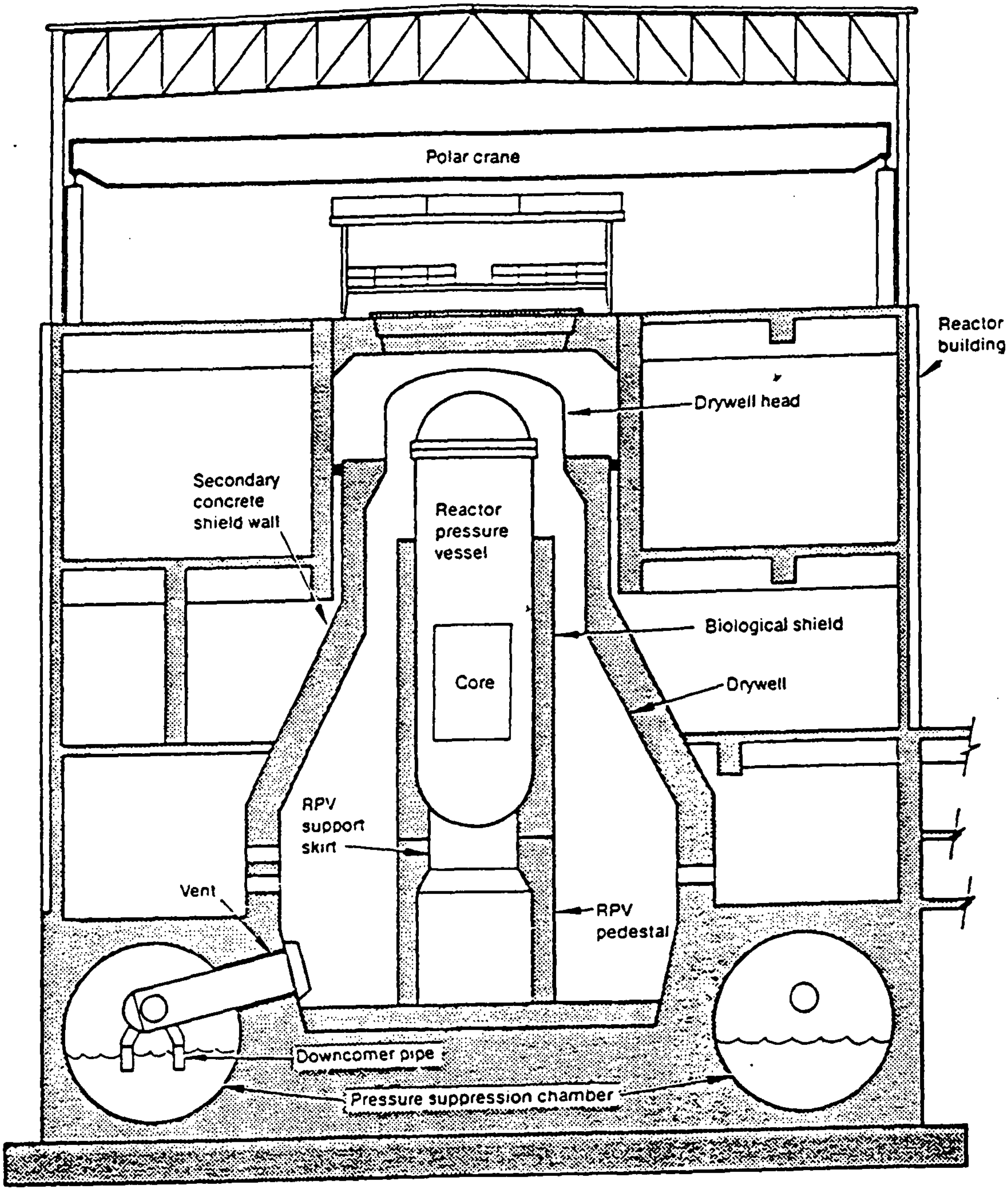


Fig. 2.1 BWR Mark I type reinforced concrete containment. Source: Copyright © 1994. Electric Power Research Institute. EPRI TR-103840-R1. *BWR Containments License Renewal Industry Report; Revision 1.* Reprinted with Permission.

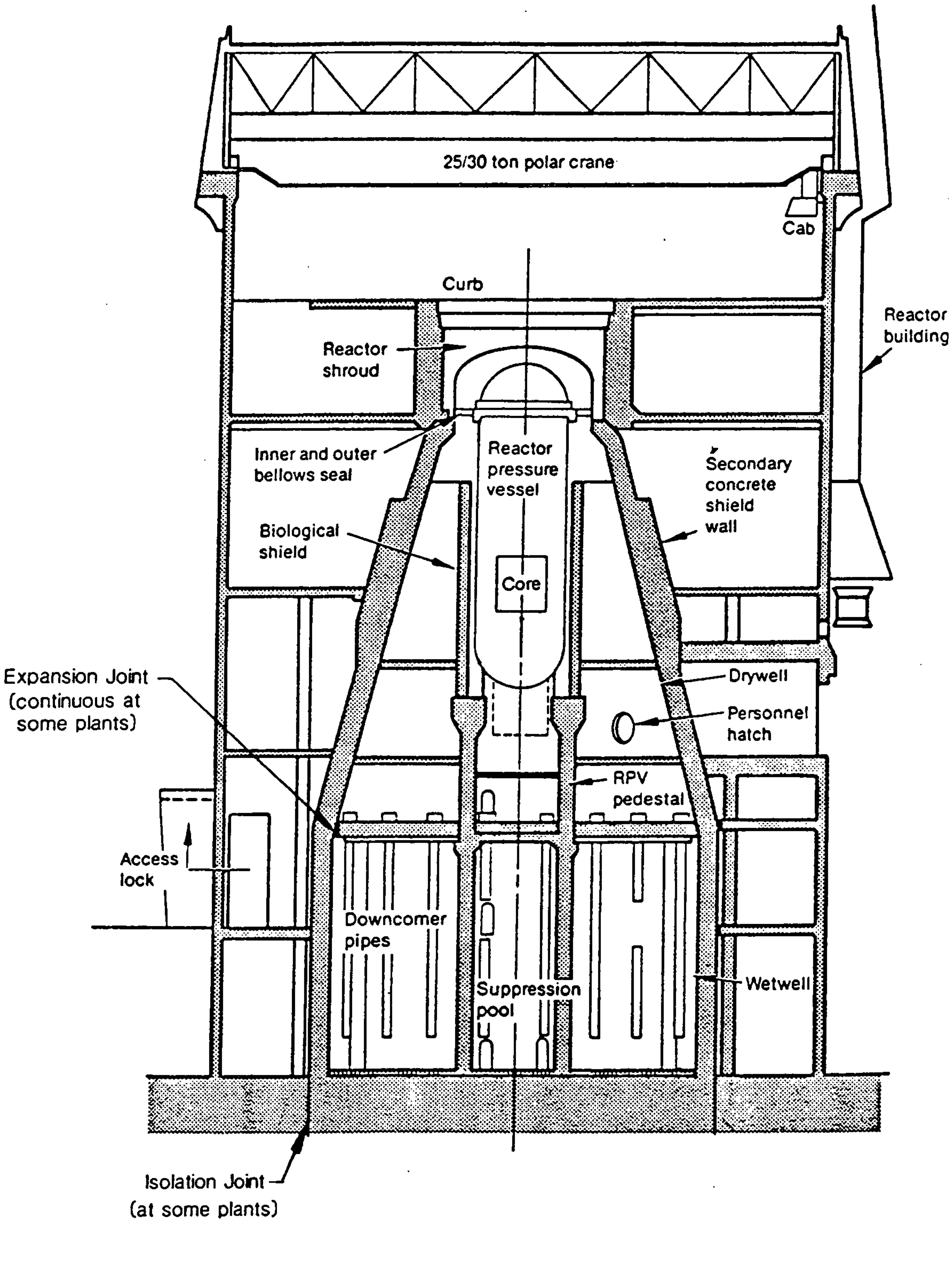


Fig. 2.2 BWR Mark II type reinforced concrete containment. Source: Copyright © 1994. Electric Power Research Institute. EPRI TR-103840-R1. *BWR Containments License Renewal Industry Report: Revision 1*. Reprinted with Permission.

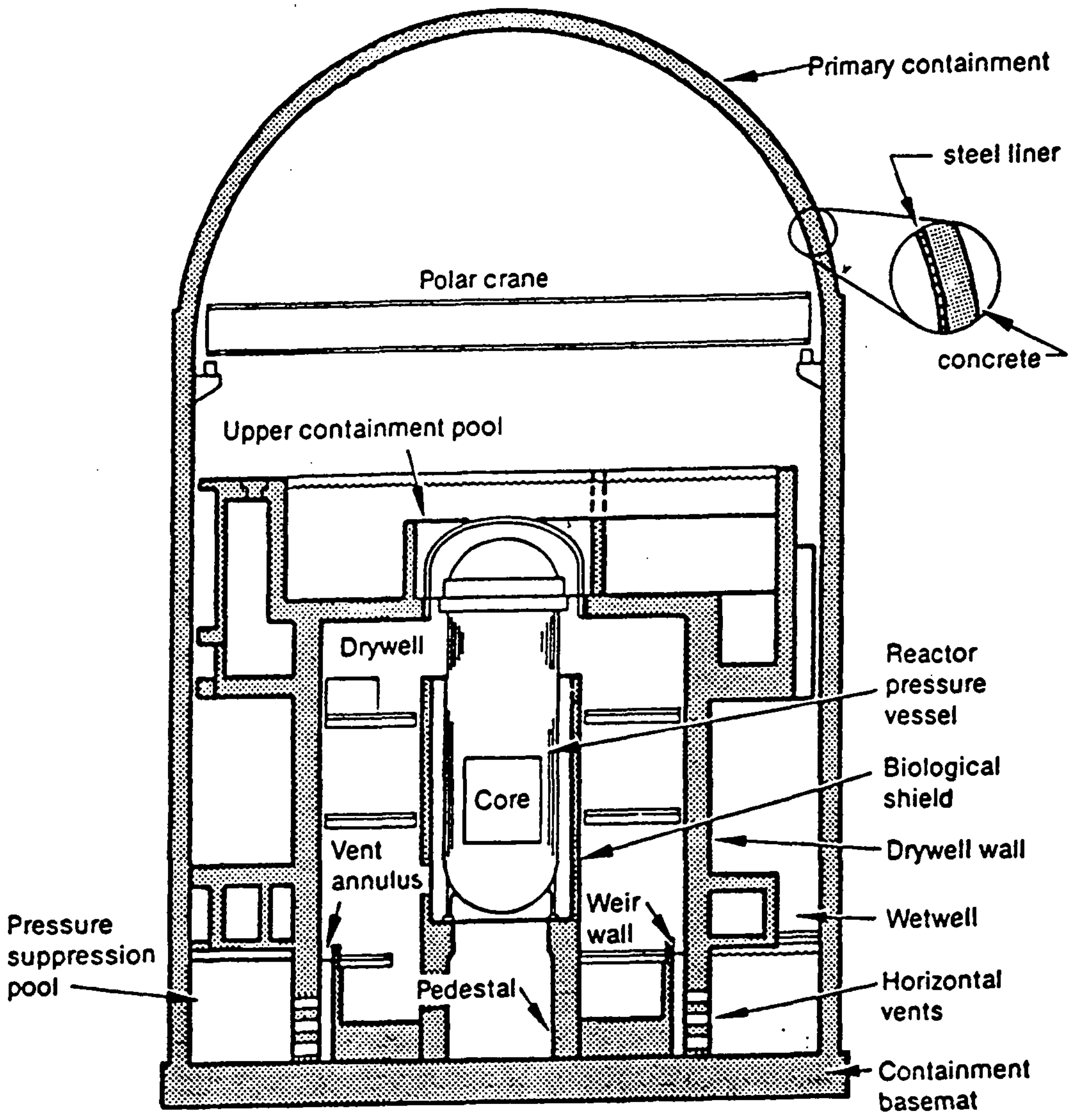


Fig. 2.3 BWR Mark III type reinforced concrete containment. Source: Copyright © 1994. Electric Power Research Institute. EPRI TR-103840-R1. *BWR Containments License Renewal Industry Report: Revision 1.* Reprinted with Permission.

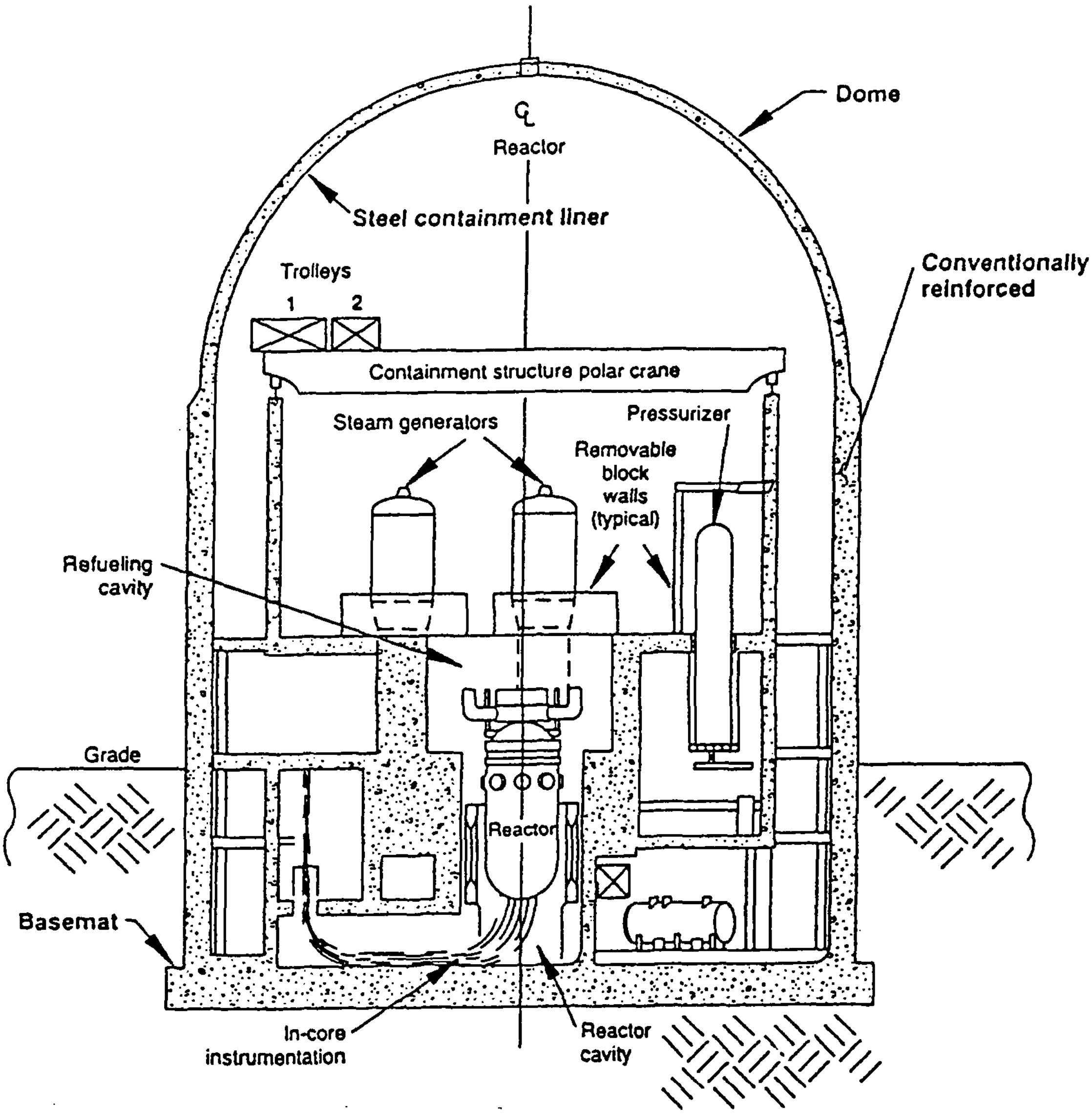


Fig. 2.4 PWR subatmospheric type reinforced concrete containment.

Source: *Insights for Aging Management of Major Light Water Reactor Components, Volume 2 – Reinforced and Prestressed Concrete Containments,* NUREG/CR-5314 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, July 1994 (Draft).

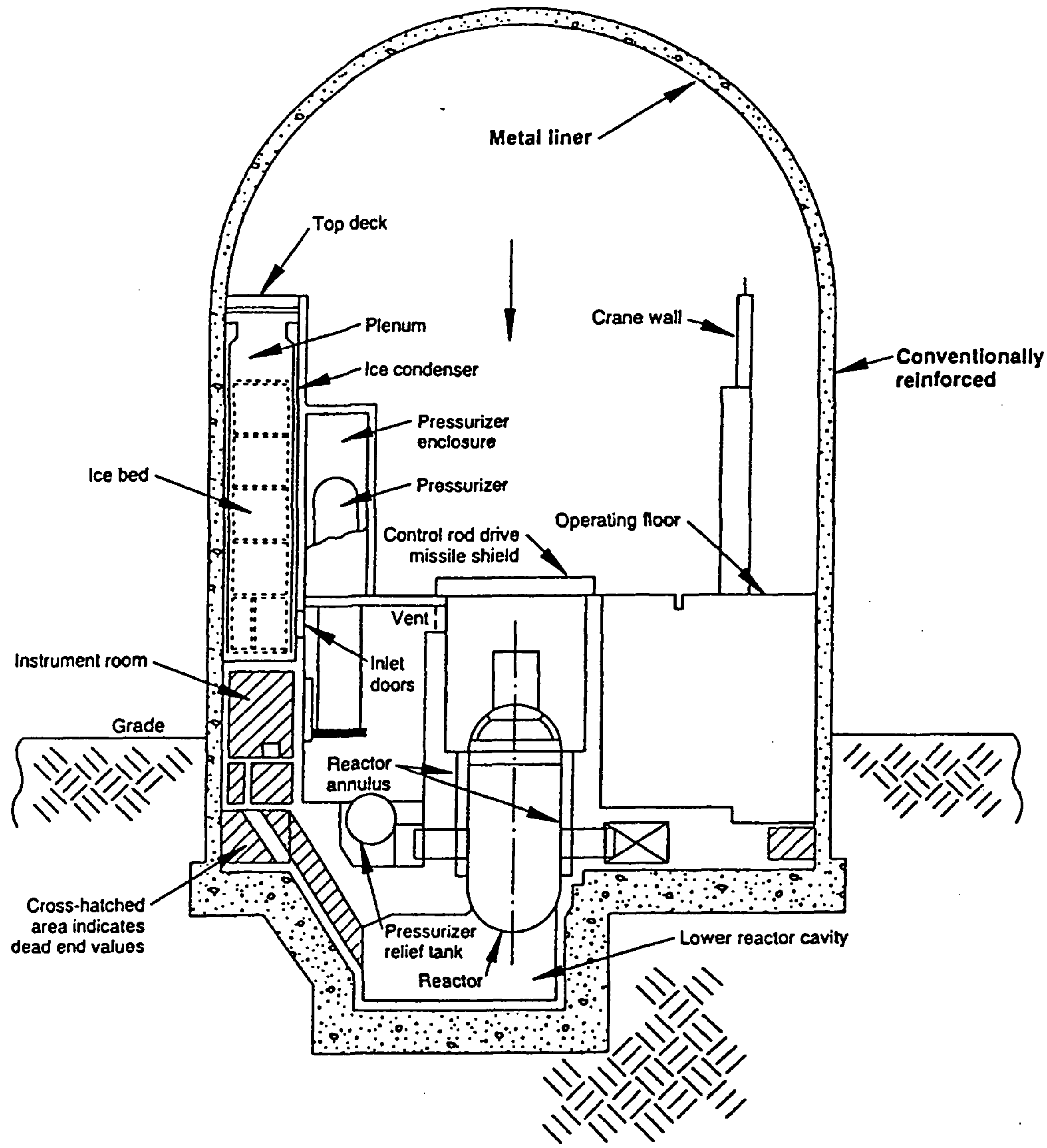


Fig. 2.5 PWR reinforced concrete containment with ice condenser.

Source: *Insights for Aging Management of Major Light Water Reactor Components, Volume 2 – Reinforced and Prestressed Concrete Containments,* NUREG/CR-5314 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, July 1994 (Draft).

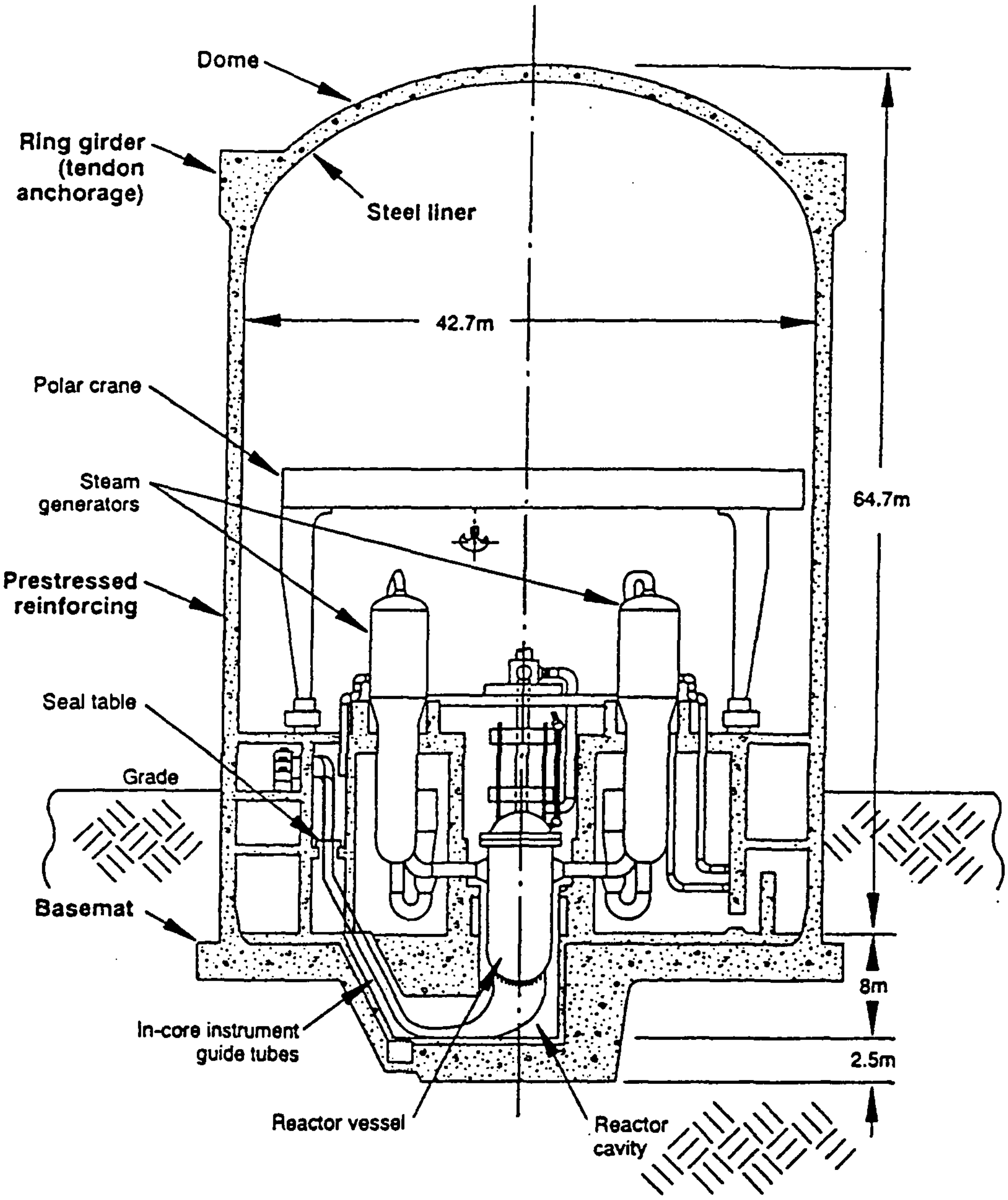


Fig. 2.6 PWR large dry prestressed concrete containment.

Source: *Insights for Aging Management of Major Light Water Reactor Components, Volume 2 – Reinforced and Prestressed Concrete Containments,* NUREG/CR-5314 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, July 1994 (Draft).

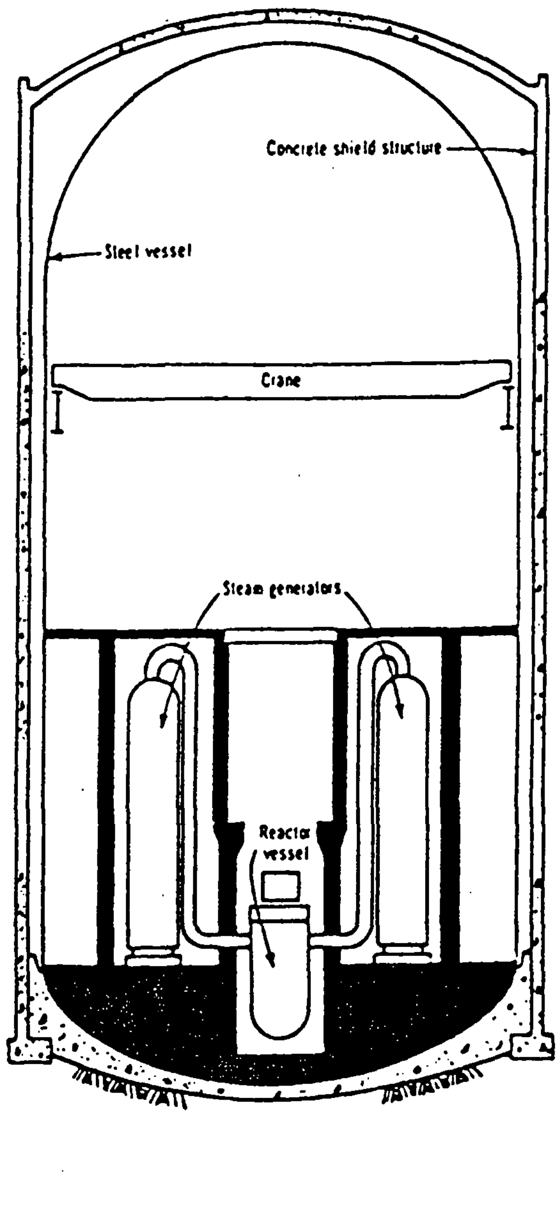


Fig. 2.7 PWR free-standing steel containment with elliptical bottom. Source: Copyright © 1994. Electric Power Research Institute. EPRI TR-103835. *PWR Containment Structures License Renewal Industry Report: Revision 1*. Reprinted with Permission.

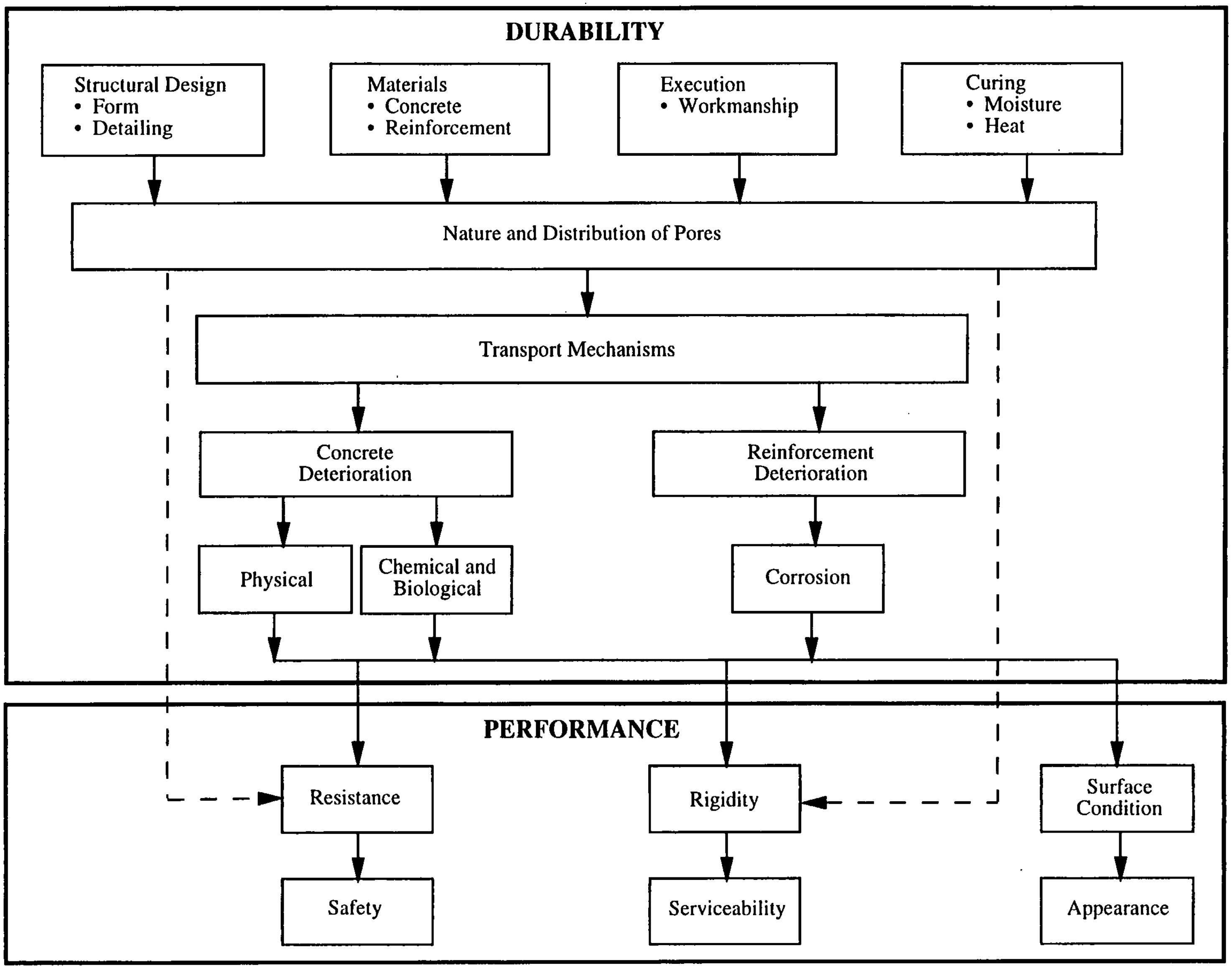


Fig. 2.8 Relationship between the concepts of concrete durability and performance. Source: Comité Euro-International du Béton (CEB), *Durable Concrete Structures–Design Guide*, published by Thomas Telford Services Ltd., London, England, 1989; reprinted with permission from CEB and Thomas Telford Services Ltd.

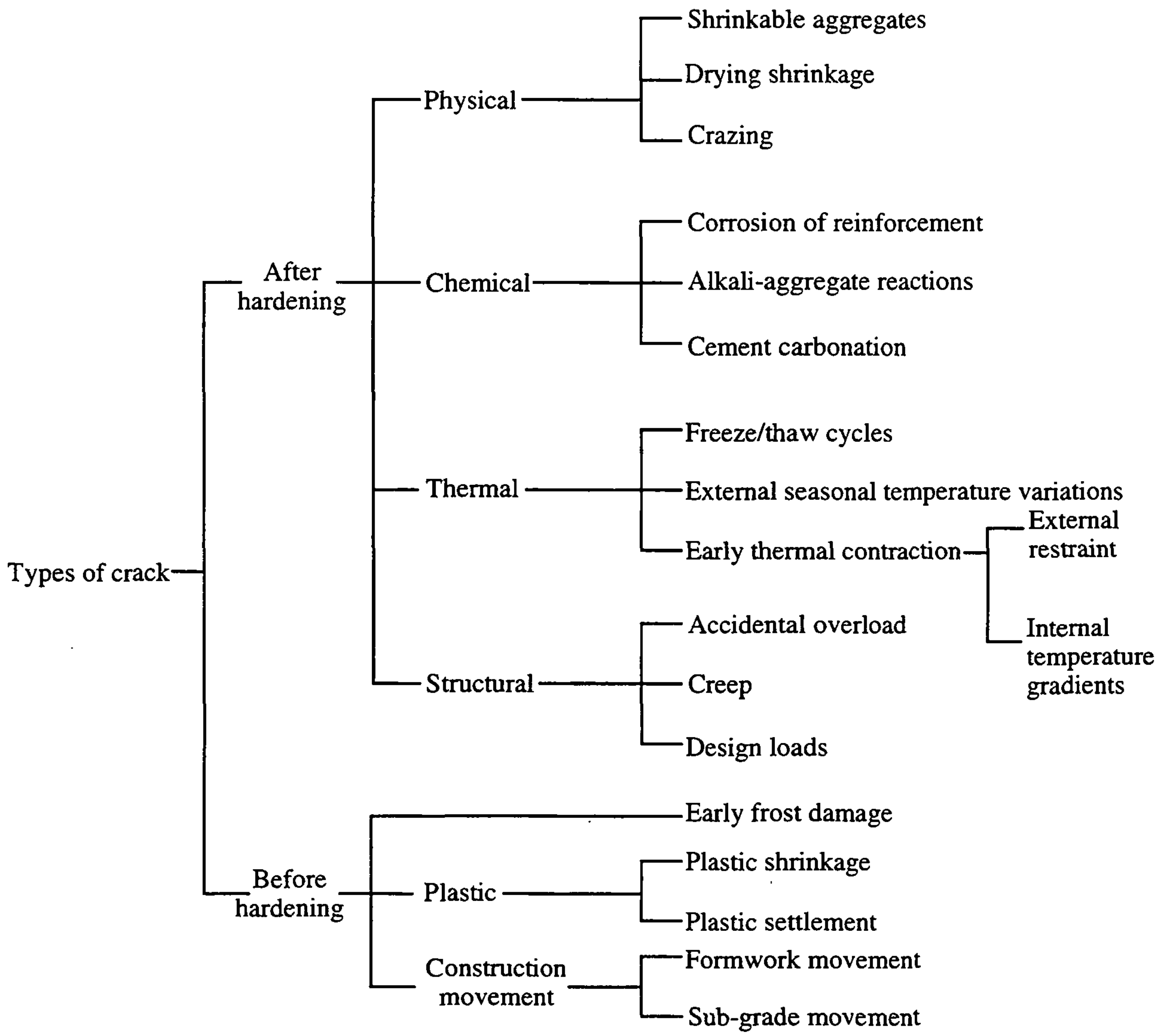
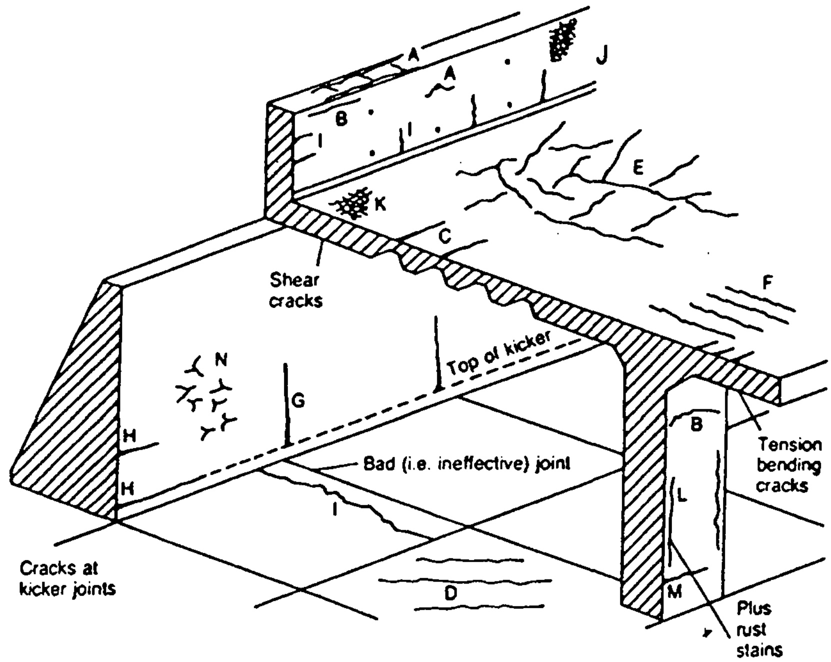


Fig. 2.9 Relationship between primary causes and types of cracks that can occur in concrete. Source: Comité Euro-International du Béton (CEB), *Durable Concrete Structures–Design Guide*, published by Thomas Telford Services Ltd., London, England, 1989; reprinted with permission from CEB and Thomas Telford Services Ltd.



A = plastic settlement, over reinforcement
B = plastic settlement, arching
C = plastic settlement, change of depth
D = plastic shrinkage, diagonal
E = plastic shrinkage, random
F = plastic shrinkage, over reinforcement

G = early thermal contraction, external restraint
H = early thermal contraction, internal restraint
I = long-term drying shrinkage
J = crazing, against formwork
K = crazing, floated concrete
L = corrosion of steel reinforcement

Fig. 2.10 Examples of intrinsic cracks in hypothetical concrete. Source: Comité Euro-International du Béton (CEB), *Durable Concrete Structures-Design Guide*, published by Thomas Telford Services Ltd., London, England, 1989; reprinted with permission from CEB and Thomas Telford Services Ltd.

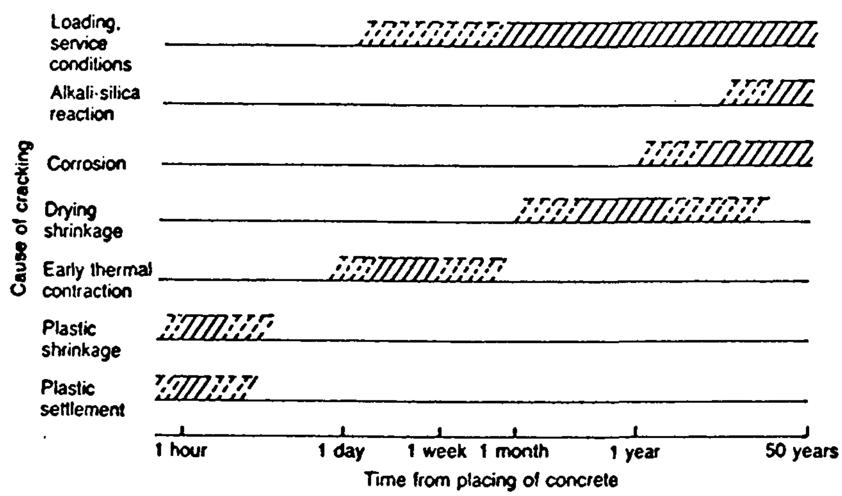
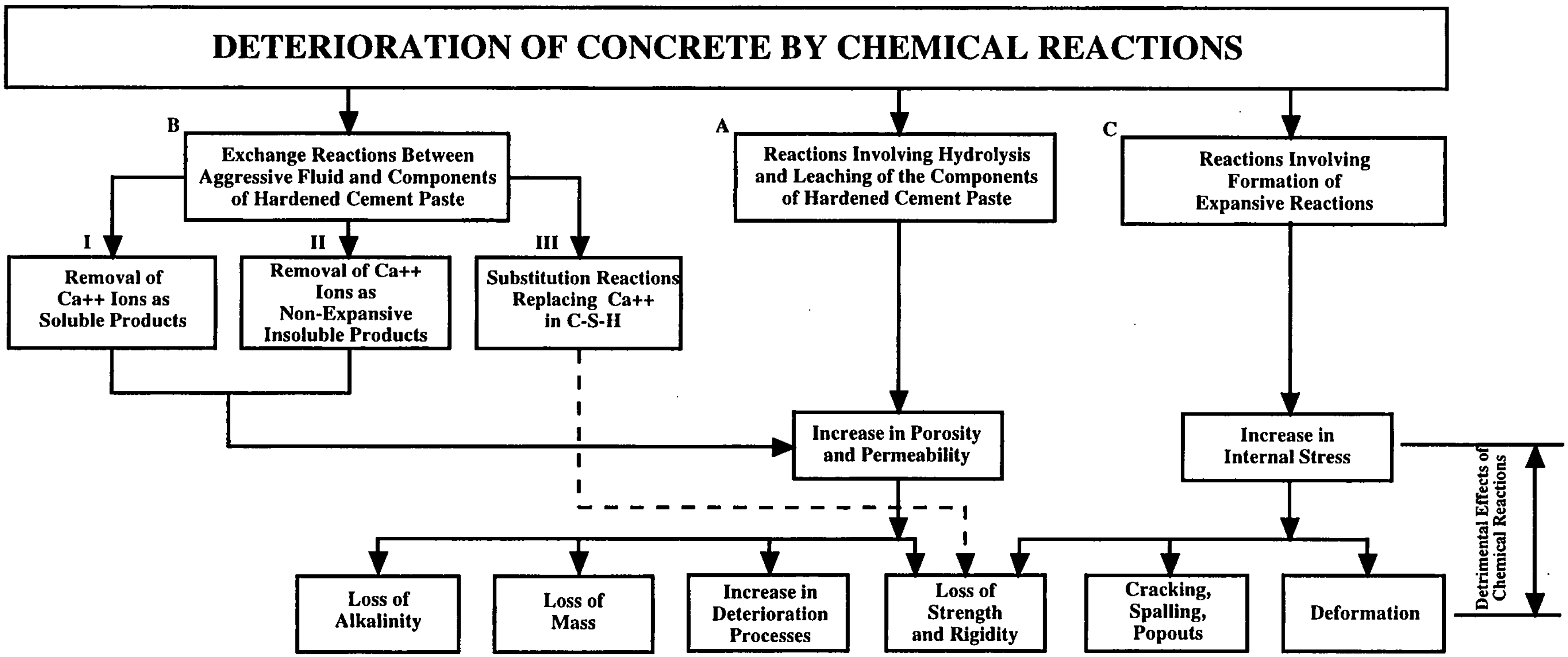


Fig. 2.11 Time of appearance of cracks from placing of concrete. Source: Comité Euro-International du Béton (CEB), *Durable Concrete Structures-Design Guide*, published by Thomas Telford Services Ltd., London, England, 1989; reprinted with permission from CEB and Thomas Telford Services Ltd.



A: Softwater attack on calcium hydroxide and C-S-H present in hydrated portland cements;
B(I): acidic solution forming soluble calcium compounds such as calcium sulfate, calcium acetate, or calcium bicarbonate;
B(II): solutions of oxalic acid and its salts, forming calcium oxalate;
B(III): long-term seawater attack weakening the C-S-H by substitution of Mg^{++} for Ca^{++};
C(1): sulfate attack forming ettringite and gypsum;
C(2): alkali-aggregate attack;
C(3): corrosion of steel in concrete; and
C(4): hydration of crystalline MgO and CaO.

Fig. 2.12 Types of chemical reactions responsible for concrete deterioration. Source: P. K. Mehta and B. C. Gerwick, Jr., "Cracking-Corrosion Interaction in Concrete Exposed to Marine Environment," *Concrete International* 4(10), pp. 45-51, Detroit, Michigan, October 1982; reprinted with permission of author.

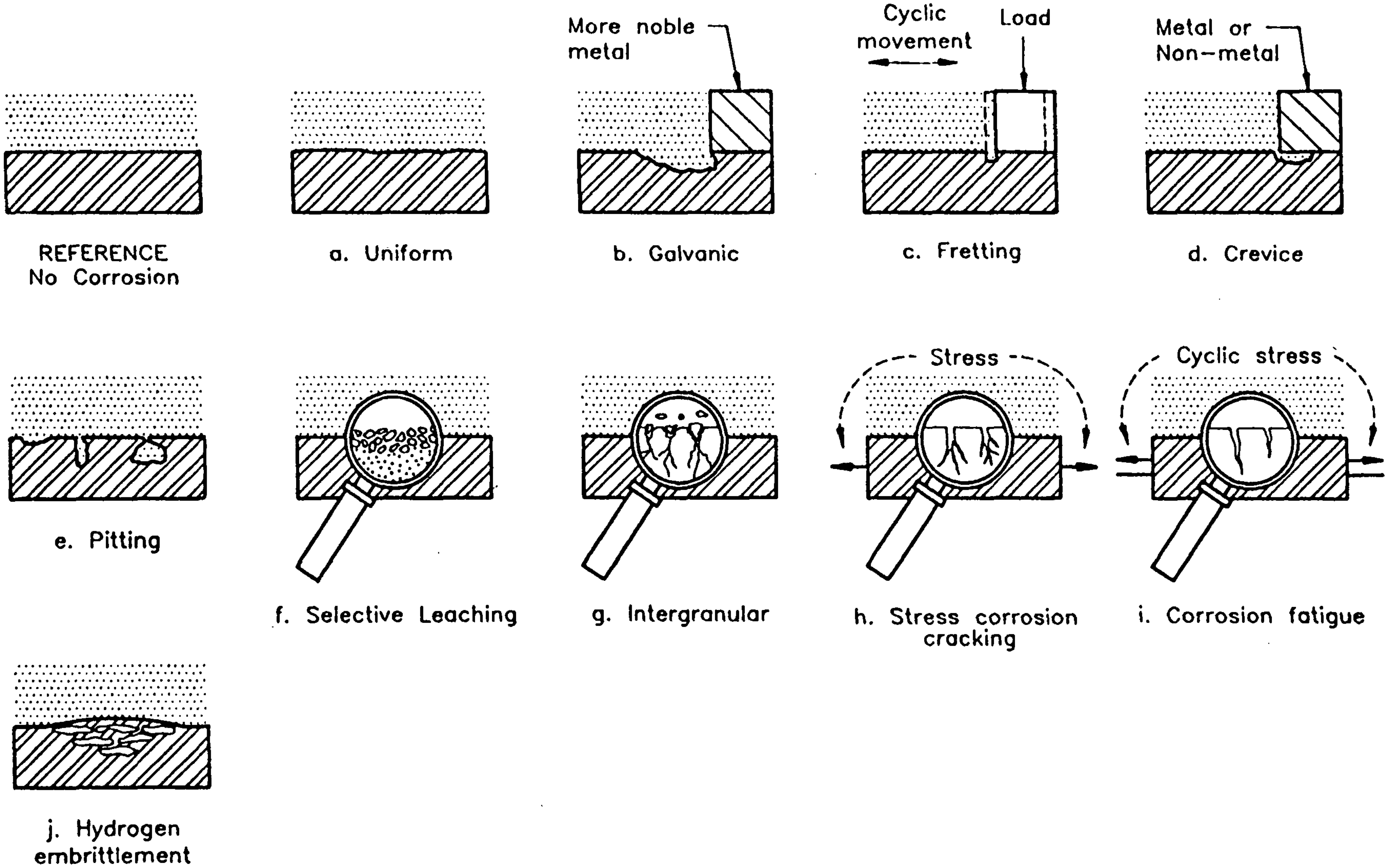


Fig. 2.13 Schematic representations of forms of corrosion that may be found in metals.

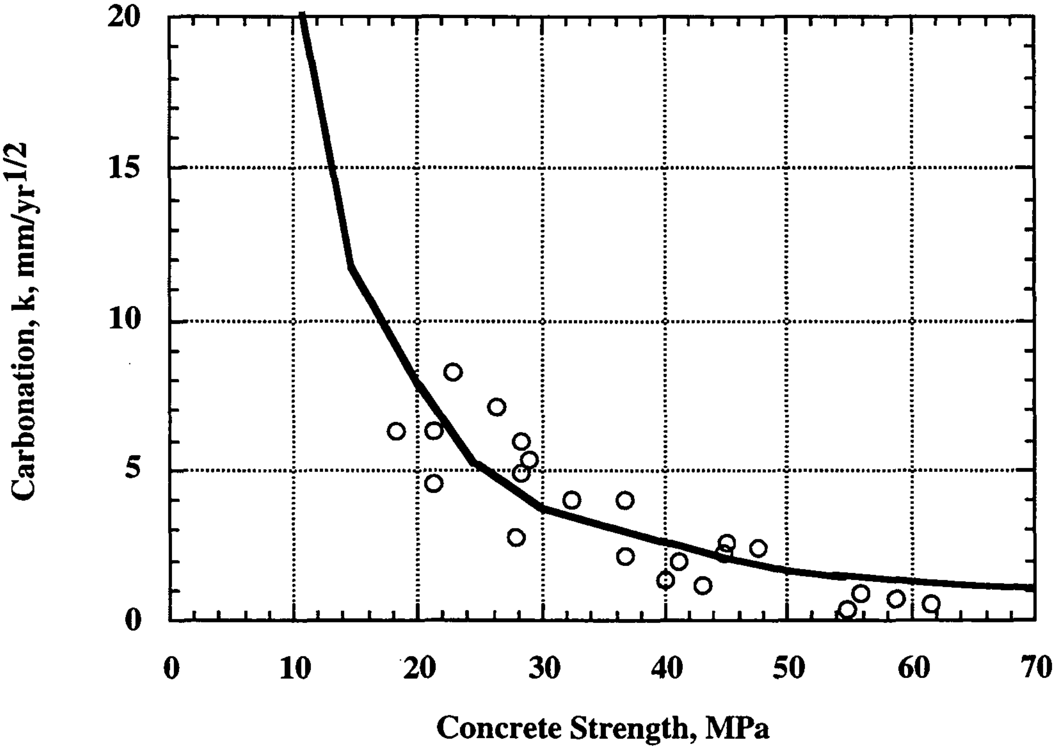


Fig. 2.14 Carbonation data for exterior sheltered concrete.[123]

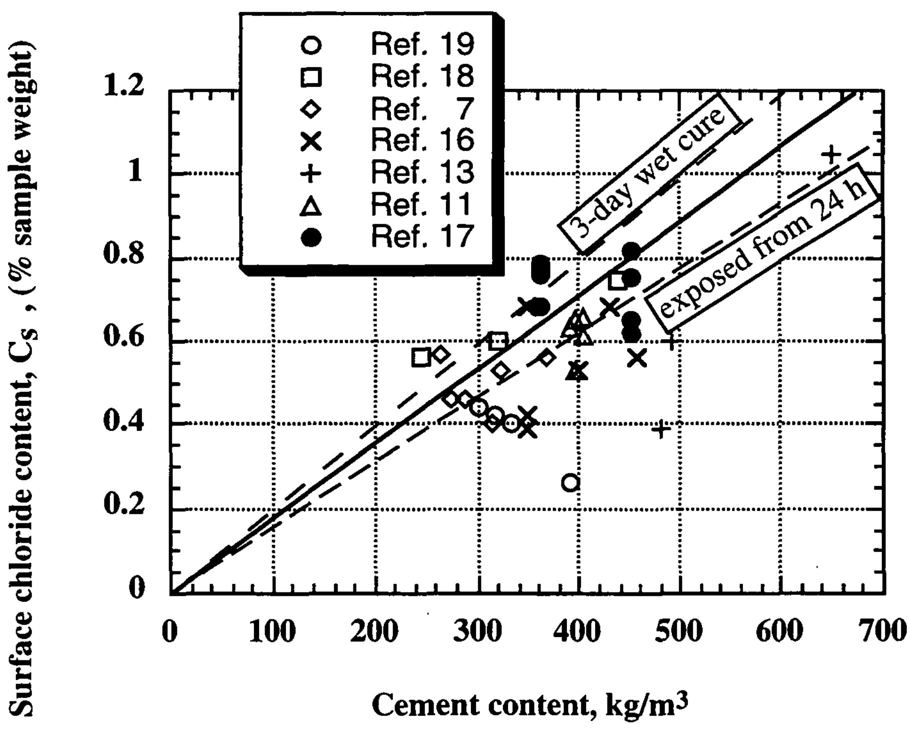


Fig. 2.15 Proposed curves for surface chloride content versus cement content for different curing conditions (Refs. refer to Ref. 123).

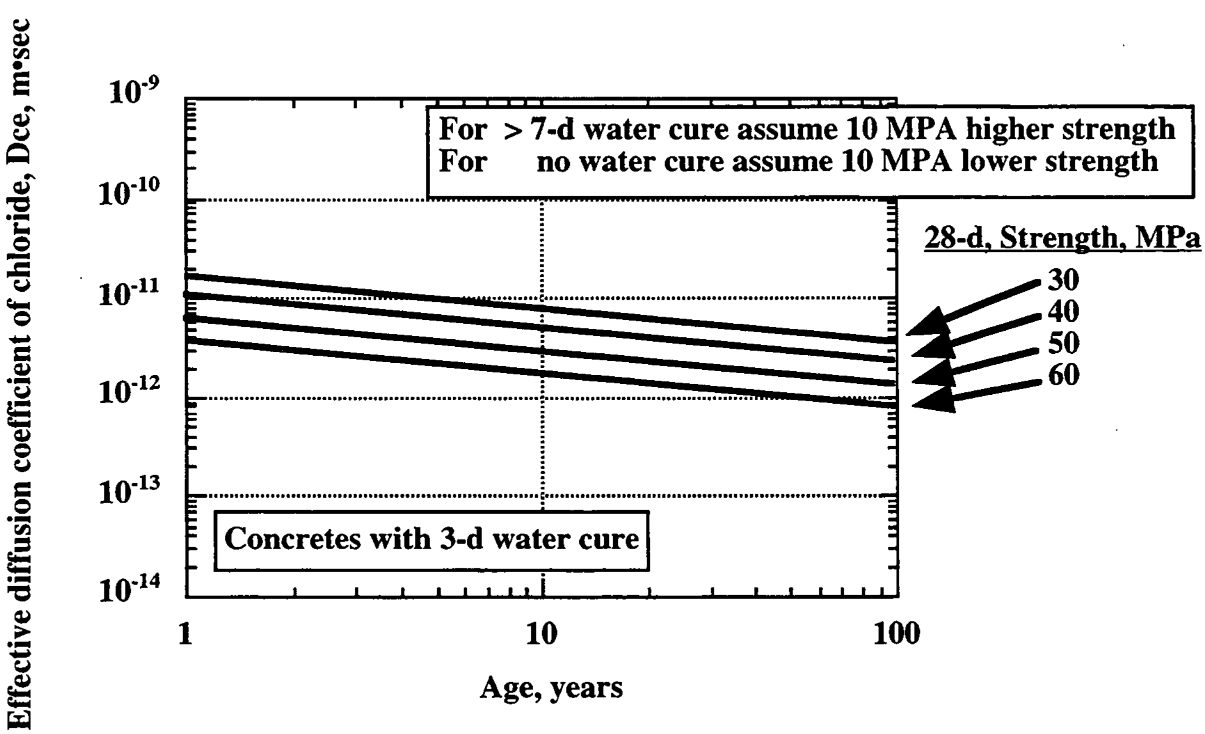


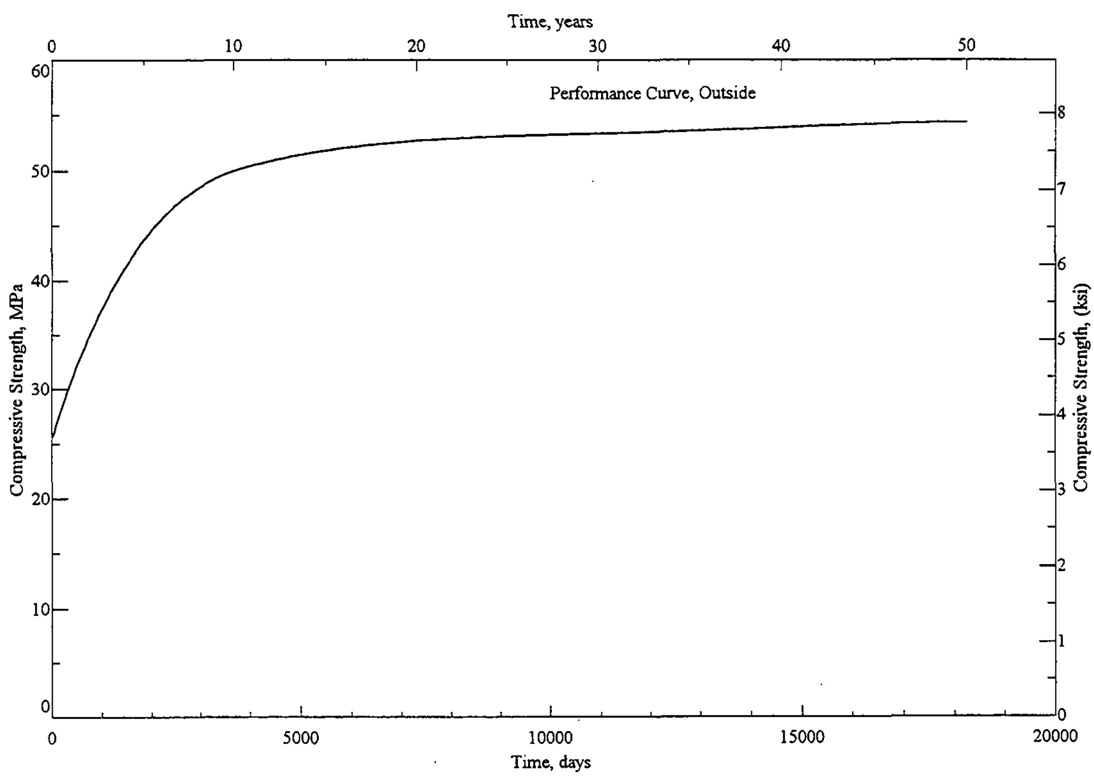
Fig. 2.16 Effective diffusion coefficient of chloride versus time for ordinary portland cement concretes.[123]

STRUCTURAL MATERIALS HANDBOOK — Volume 1 - Performance Values

Material Code 01CB004 — Property Code 3621 — Page 1.1

Portland Cement Concrete Normal-Weight Gravel Aggregate Series B, Janesville, 0.41	Ultimate Compressive Strength versus Time	Update Package Number 2 Revision Control Code 1.0 Quality Level A

University of Wisconsin
Series B, Janesville, Water-Cement Ratio = 0.41



Preparer: C. B. Oland Organization: Oak Ridge National Laboratory	Sources: References 24, 25, 26, 27 and 28

See Page 1.2 for a list of computed compressive strength values and the equations used to generate the ultimate compressive strength versus time performance curve.

Fig. 2.17 Example of page from Vol. 1 (Performance Values) of *Structural Materials Handbook*.

STRUCTURAL MATERIALS HANDBOOK — Volume 2 - Supporting Documentation

Material Code 01CB004 — Property Code 3621 — Page 1.3

Portland Cement Concrete Normal-Weight Gravel Aggregate Series B, Janesville, 0.41	Ultimate Compressive Strength versus Time	Update Package Number 0 Revision Control Code 0.0 Quality Level A

Cement Vendor	Compressive Strength Test Results for Specimens Stored Outside, MPa (psi) at:						
	7 Days	28 Days	1 Year	5 Years	10 Years	25 Years	50 Years
Medusa (3M)	16.0 (2325)	22.9 (3315)	31.6 (4580)	46.7 (6780)	50.1 (7260)	51.0 (7400)	49.0 (7110)
Lehigh (4M)	18.1 (2620)	23.8 (3455)	33.9 (4910)	47.8 (6930)	49.6 (7195)	52.1 (7555)	59.7 (8660)
Universal (5M)	17.4 (2530)	27.8 (4035)	34.8 (5050)	48.0 (6960)	47.6 (6900)	55.1 (7990)	48.7 (7070)
Marquette (7M)	19.3 (2805)	28.2 (4095)	35.8 (5190)	49.3 (7145)	52.5 (7615)	54.1 (7850)	60.1 (8715)
Average	17.7 (2570)	25.7 (3725)	34.0 (4930)	48.0 (6955)	49.9 (7240)	53.1 (7700)	54.4 (7890)

Test specimens were cast with each of these four cements, moist cured for 28 days, and then placed outside in Madison, Wisconsin for long-term storage. Each value listed above is the average compressive strength (Property Code 3023) from five test specimens (Reference 134).

Fig. 2.18 Example of page from Vol. 2 (Supporting Documentation) of *Structural Materials Handbook.*

STRUCTURAL MATERIALS HANDBOOK — Volume 3 - Material Data Sheet

Material Code 01CB004 — Property Code 1000 — Page 2

Portland Cement Concrete Normal-Weight Gravel Aggregate Series B, Janesville, 0.41	General Information	Update Package Number 0 Revision Control Code 0.0 Quality Level A

Property Code 1210	Material Composition		
Constituent Material	Mix Proportions per Unit Volume		Property Code
	kg/m^3	lb/yd^3	
Portland Cement ASTM C 150, Type I	369	622	2001
Fine Aggregate	724	1220	2211
Coarse Aggregate	1236	2084	2222
Water	151	255	2421
Total	2421	4081	

The mix proportions were 1:1.5:3 by volume or 1:1.8:3.35 by weight (derived from Reference 134).

Property Code 1220	Processing Information

Each concrete specimen was moist cured for 28 days and then placed outside in Madison, Wisconsin for long-term storage. Outside storage consisted of placing each specimen on level ground in an uncovered cage having a northeast exposure until 1950, and then each specimen was moved to an uncaged location in an open area for the remaining time. The relative humidity in Madison normally varies from 65 to 100 percent and averages about 75 percent. The annual precipitation including snowfall is about 810 mm (32 in.). Annual air temperatures usually range between -32 and 35°C (-25 and 95°F).

Fig. 2.19 Example of page from Vol. 3 (Material Data Sheet) of *Structural Materials Handbook.*

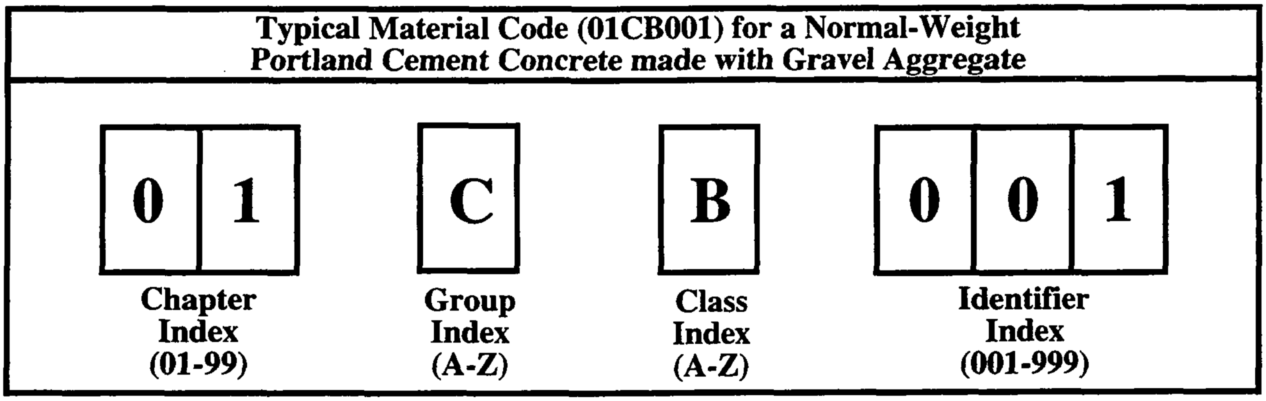


Fig. 2.20 Material code arrangement.

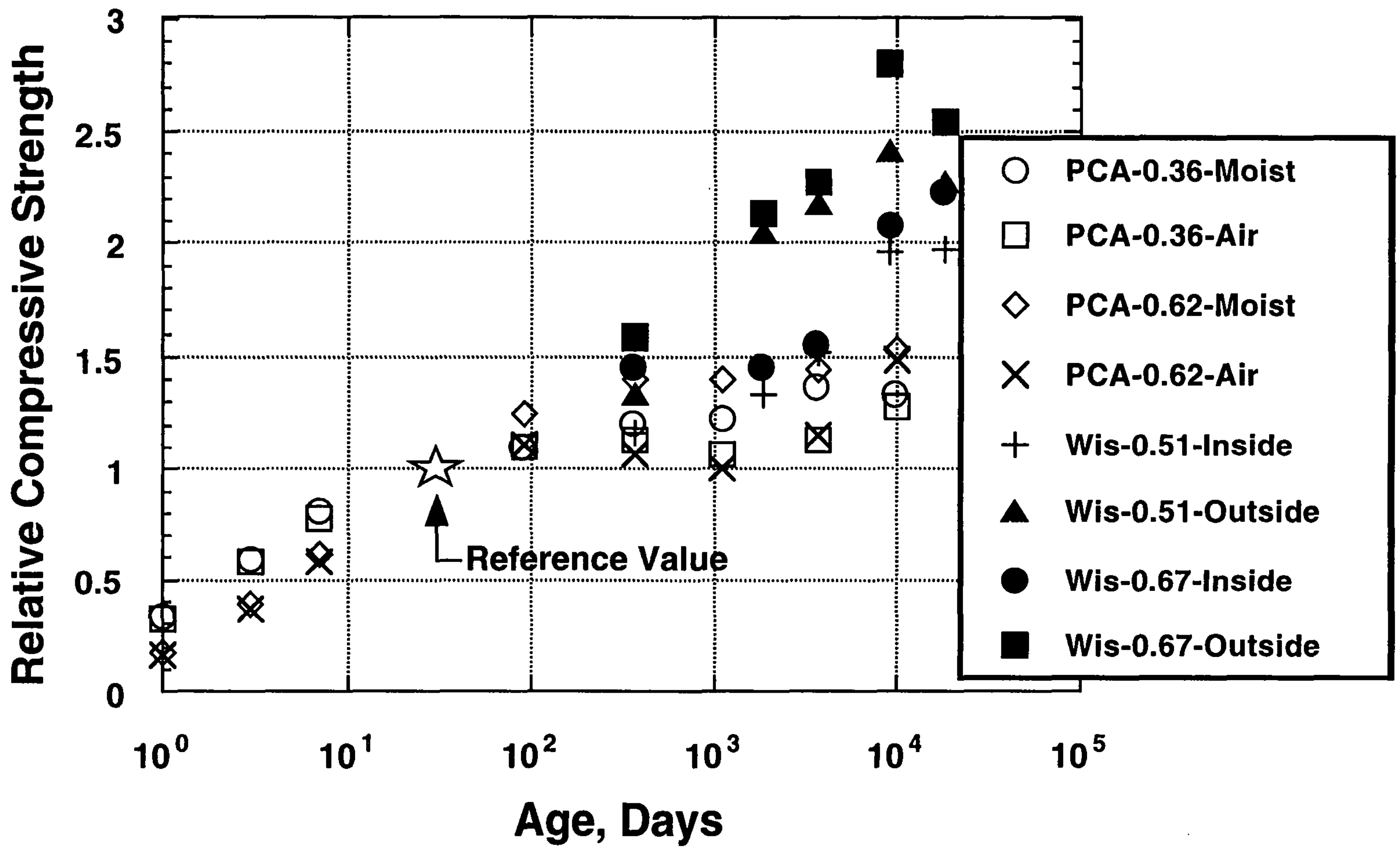
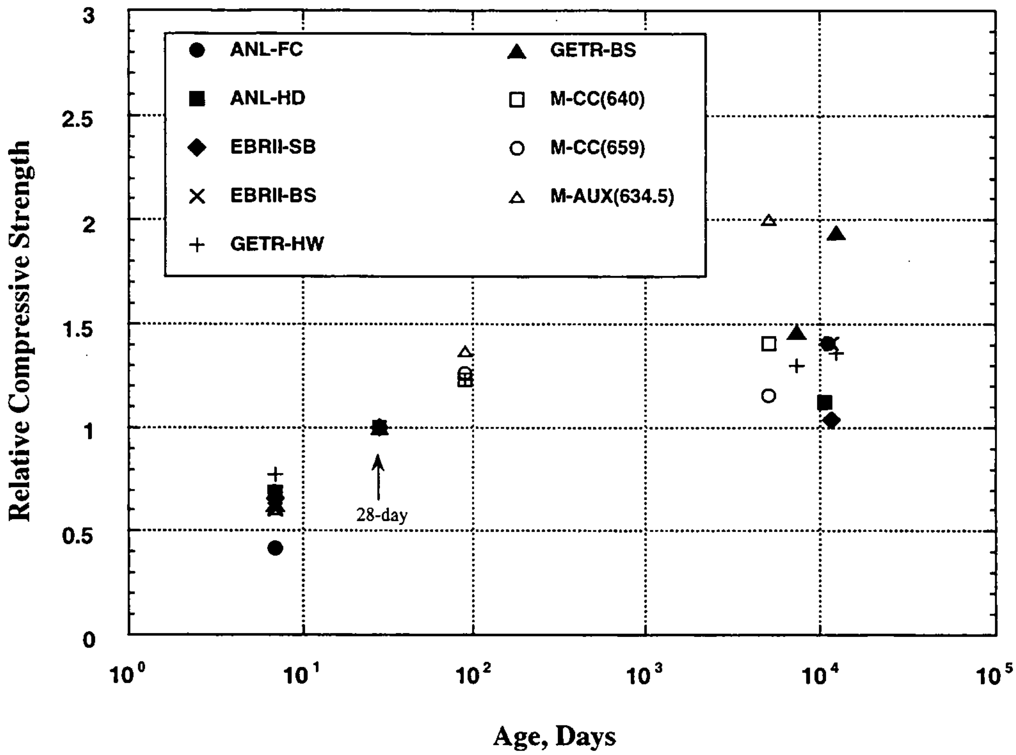
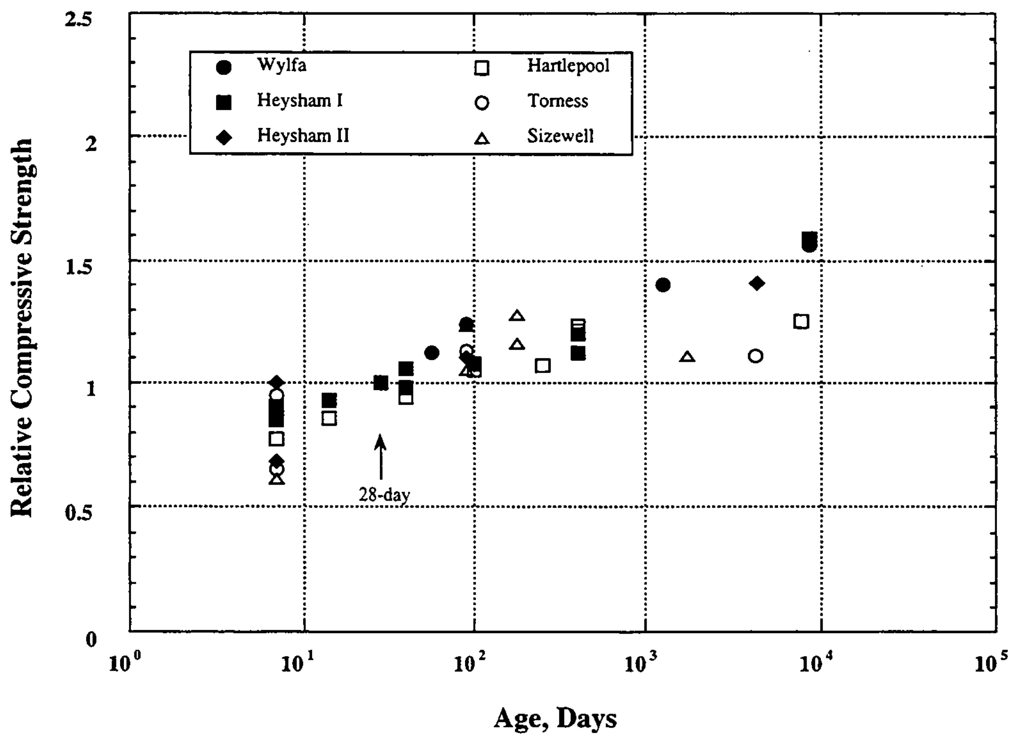


Fig 2.21 Long-term compressive strength results for selected data from University of Wisconsin and Portland Cement Association.



a. Concretes obtained from several U.S. nuclear power facilities



b. Concretes obtained from several U.K. nuclear power facilities

Fig. 2.22 Relative compressive strength for concretes obtained from several U.S. and U. K. nuclear power stations.

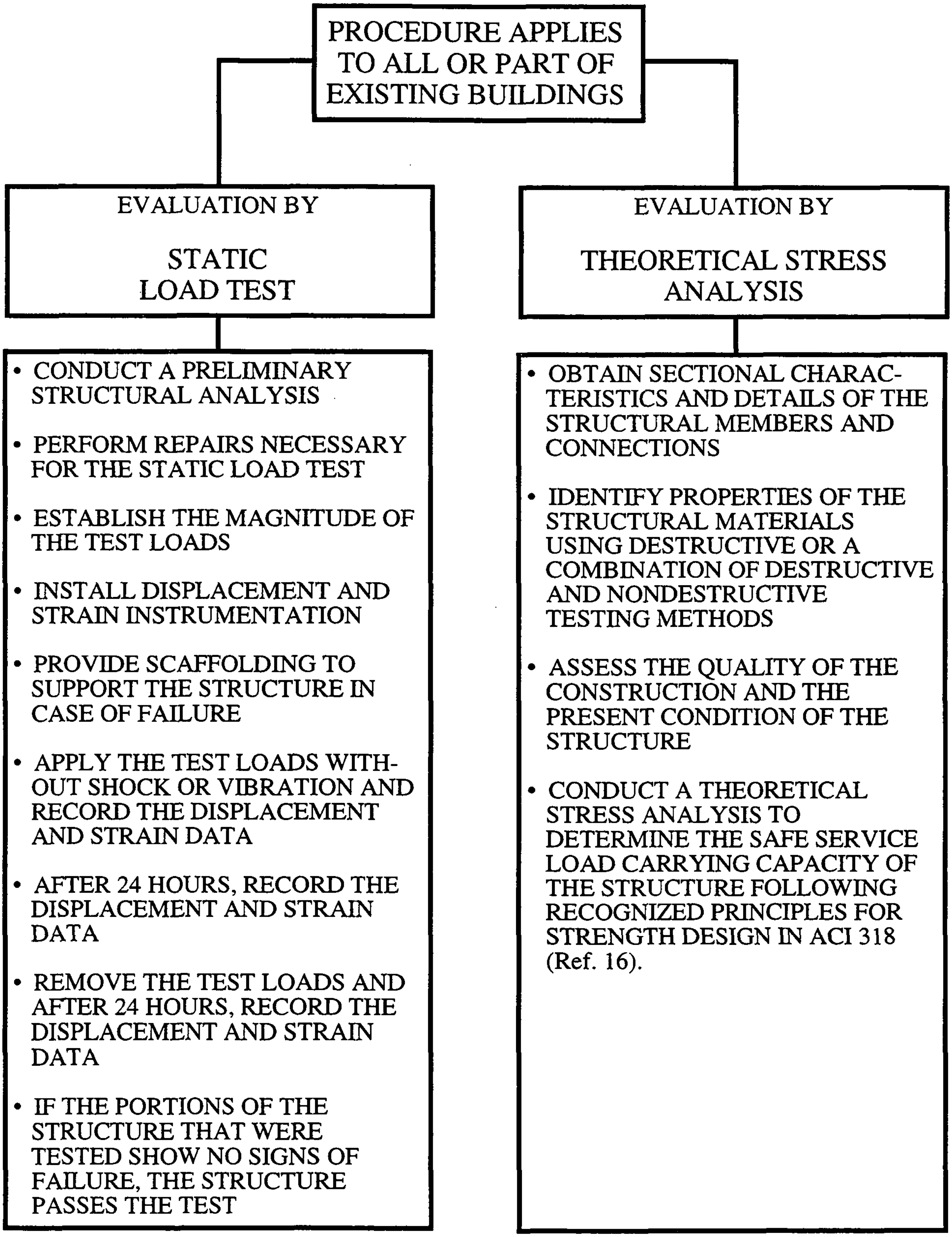


Fig. 2.23 Recommended procedure for strength evaluation of existing concrete buildings.

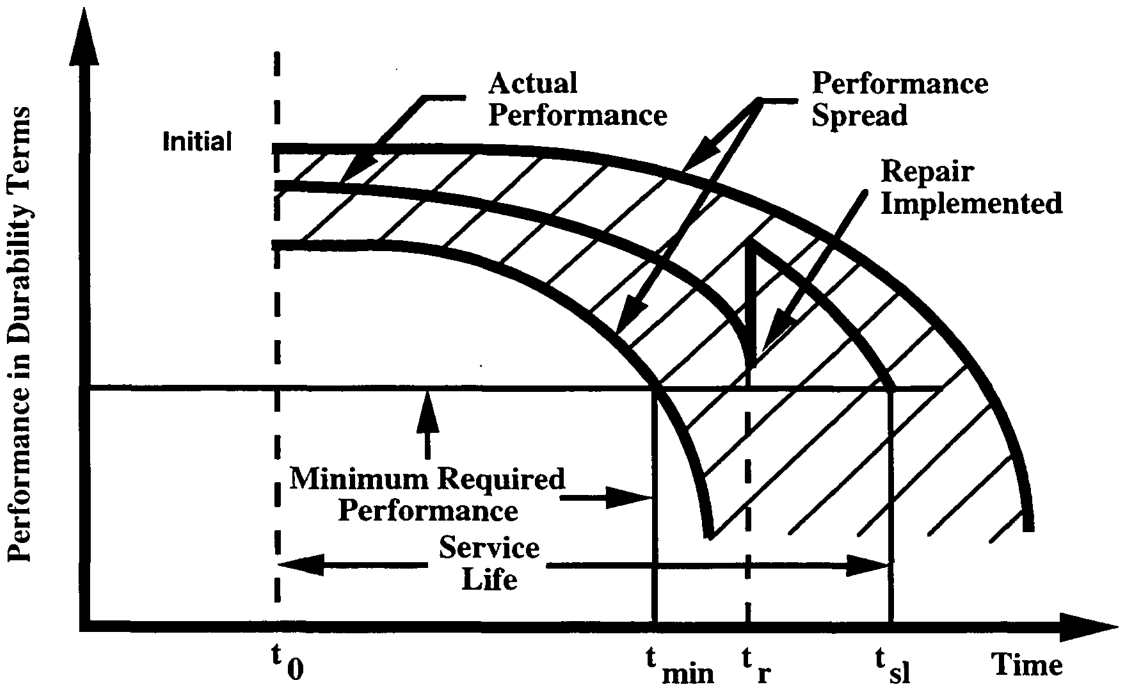


Fig. 2.24 Relationship between concrete performance and service life.

3. IN-SERVICE INSPECTION, CONDITION ASSESSMENT, AND REMEDIAL MEASURES CONSIDERATIONS

As noted in the previous chapter, concrete structures in nuclear power plants (NPPs) are subjected in use to many types of environmental influence that can impact their ability to continue to meet functional and performance requirements. Due to the significant safety as well as economic consequences that could result if these structures were to deteriorate to unacceptable performance levels, it is important that these structures be inspected at regular intervals and maintained. Data and information developed from periodic condition assessments can be used to indicate the current structural condition as well as to estimate future performance (i.e., continued service assessments). Maintenance and repair activities also are essential elements in an overall program to effectively manage the aging of these structures.

3.1 IN-SERVICE INSPECTION TECHNIQUES

Detection of age- or environmental stressor-related degradation, as well as its magnitude and rate of occurrence, are key factors in maintaining the readiness of safety-related concrete structures in NPPs to continue their functions in the unlikely event that a condition such as a loss-of-coolant accident (LOCA) would occur.

3.1.1 Current In-Service Inspection Techniques

Continuing satisfactory service of a reinforced concrete NPP structure over an extended period of time is dependent in large measure on the durability of its constituents. Techniques for detection of degradation of reinforced concrete structures should, therefore, address evaluation of the concrete, steel reinforcement, liner plate, and anchorage embedments. Table 3.1 presents a summary listing of evaluation procedures routinely used to determine concrete properties, the physical condition of concrete, and the occurrence or potential for occurrence of corrosion of steel either embedded in or in contact with concrete.[162] Pertinent information relating to several of these procedures is provided in Table 3.2.[163] A summary description of commonly used techniques is provided below. Additional background information on inspection of concrete materials and structures is available elsewhere.[164–167]

3.1.1.1 Concrete material systems

Primary manifestations of distress that are present or can occur in reinforced concrete structures include (1) cracking, voids, and delaminations; and (2) strength losses. Although not an aging-related phenomenon, whether the concrete was fabricated using the proper constituents and mix design, as well as properly placed, compacted and cured, could also become a service life consideration and should be considered as part of the overall evaluation process. For reinforced concrete structures in NPPs that may not be accessible for inspection without the removal of soil, portions of neighboring structures, etc. (e.g., basemat), inspection can be addressed indirectly through an assessment of exposure conditions.

<u>Detection of Cracking, Voids, and Delaminations</u> Methods used to detect discontinuities in concrete structures generally fall into two categories: direct and indirect. Direct techniques involve a visual inspection of the structure, removal/testing/analysis of material(s), or a combination of the two. The indirect techniques generally measure some parameter from which an estimate of the extent of degradation can be made through existing correlations. Quite often, however, evaluation of concrete materials and structures requires the use of a combination of test methods since no single testing technique is presently available that will detect all potential

degradation factors. Detection of concrete cracking, voids, and discontinuities generally involves either visual inspection, nondestructive testing, destructive testing (e.g., removal and examination of cores), or a combination of these methods.

Visual Inspection

Periodic visual examination of exposed concrete provides a rapid and effective method for identifying and defining areas of distress (e.g., cracking, spalling, volume change, or cement/aggregate interaction). By locating, marking, identifying by type and orientation, and measuring and recording conditions associated with cracks (e.g., seepage, edge spalling, differential settlement, etc.), a history can be established that will be of assistance in identifying the cause and establishing whether a crack is active or dormant. A crack comparator capable of width determinations to an accuracy of 0.025 mm can be used to establish cracks that are above a critical size that permit the entry of hostile environments to attack either the concrete or its steel reinforcement.[168,169] Subsurface cracking, delaminations and voids, and extent of cracking, however, cannot be established through visual examination.

Nondestructive Testing

Nondestructive testing techniques are available that can be utilized to determine the presence of internal cracks, voids or delaminations, and the depth of penetration of cracks visible at the surface. Ultrasonic pulse velocity techniques are generally used, but acoustic impact, radiography and radar, and thermal techniques have also been used.

Detection of cracks or voids in concrete by using ultrasonic through-transmission measurements is based on the principle that the amplitude and direction of travel of ultrasonic compressional waves propagating through concrete will be changed when they encounter a discontinuity.[170] The ultrasonic pulses are emitted by a transmitter, and the transit time to a receiver is measured by electronic means in terms of either transit time (microseconds) or path length. Large internal flaws in concrete can be detected by an abnormally long transmission time and/or a large decrease in the magnitude of the ultrasonic pulses as they pass around a discontinuity. The technique can also be used to indicate concrete changes due to freezing and thawing or other aggressive environments. Primary advantages of this technique are that it is an excellent method for rapidly estimating the quality and uniformity of concrete and a low level of user expertise is required to make measurements. Disadvantages are that sound transmission through concrete is influenced by a number of conditions (e.g., aggregate characteristics, mix designs, presence of reinforcement, etc.) and quantitative interpretation of results can be difficult.

Stress wave reflection/refraction methods are based on monitoring the interaction of acoustic (or stress) waves with the internal structure of an object.[171] Striking an object by mechanical impact or an electromechanical transducer and listening to or recording and analyzing the stress waves is a common method for detecting the presence of voids, cracks, or delaminations. Stress wave propagation methods include pulse-echo, impact-echo, and stress wave refraction. In the pulse-echo method, a stress pulse is introduced into an accessible surface by impact and as the stress wave propagates into the test article with flaws or other interfaces causing reflections. The surface response in the form of the reflected wave is monitored by either the transmitter acting as a receiver (true pulse-echo) or by a second transducer located near the pulse source (pitch-catch). Receiver output is displayed on an oscilloscope and, by using a time base display, the round-trip travel time of the pulse to the reflecting interface can be determined. Pulse-echo techniques have been used to measure the thickness of thin concrete sections and to identify and locate defects and discontinuities. Interpretation of results can be difficult with optimum performance requiring calibration and referencing. The impact-echo system consists of an impact source (e.g., an instrumented hammer having a characteristic transfer function), a receiving transducer, and a digital processing oscilloscope or wave form analyzer that is used to capture the transient output of the transducer, store the digitized wave forms, and perform signal analysis.[172] Reflections of P- and S-waves by internal interfaces or external boundaries are sensed at the receiving transducer as displacements and recorded by the oscilloscope. Examination of the amplitude spectra from scans taken at several positions on the test article surface are used to indicate the approximate size of defects. Impact-echo can detect various types of interfaces and defects (e.g., cracks, voids, and honeycombed concrete), the depth of surface-opening cracks, voids in prestressing ducts, and thickness of wall and slab sections.[173] Interpretation of the force-time history requires relatively complex signal processing. Stress wave refraction uses a hammer to transmit signals into a material. Receivers are located in four directions at equal distances from the impactor. The signal travels through the material as spherical and surface waves and the time required to travel from the point of impact to the closest transducer is measured.[174] Deteriorated concrete will cause an increase in transit time making the method capable of indicating the relative quality and detecting cracking at different locations in a structure.

Audio or acoustic impact methods utilize the frequency characteristics or "ringing" resulting from striking a concrete surface with a hammer, chain, rod, etc., to detect cracks, voids, and delaminations. The theory underlying this technique is that when a metallic object strikes a solid (nondelaminated) area, a metallic ringing sound will be heard. If a defect is present, a "hollow" sound would be emitted. Advantages of the technique include relative ease of use and simplicity of the equipment. However, the technique requires experience to interpret results, it can miss small delaminations, and results can be affected by geometry and mass of the test object.

Radioactive (X- and gamma-ray) techniques consist of a beam from a radiation source, the object examined, and an image collector. These systems employ a photographic film placed on the opposite side of the specimen from the source as a detector. The intensity of radiation passing through the test article varies according to the article's thickness, density, and absorptive characteristics. Applications of X-ray radiography in the field, because of its relatively high initial cost and limited mobility of testing equipment, have been limited to establishing steel reinforcement location, investigating bond stress in prestressed concrete, and showing concrete density variations. Gamma-ray radiography, because of its use of less costly portable equipment and its ability to make measurements up to concrete thicknesses of 450 mm, has been widely used to determine the condition and position of steel reinforcement, voids in post-tensioning tendon grout, and variable compaction of concrete.[175] Depending on the source, structures up to 1-m thick can be examined and voids as small as 5 mm detected. Radiography, however, is fairly expensive, skilled operators and a license to operate are required as well as appropriate radiation safety protection, and thin cracks or planar defects parallel to the radiation beam may not be detected.

Penetrating- or impulse-radar is based on the principle of electromagnetic wave reflection and consists of a main radar unit that radiates short-time-duration electromagnetic pulses (0.5 to 1.5 nanoseconds) into the material examined, a graphic printer or computer-aided analyzer, and an antenna. The main radar unit is the primary processing section of the radar equipment. The antenna houses the transmitter as well as the receiver. When an electromagnetic wave travels through different mediums (different dielectric constants), a portion of the wave is reflected back to the antenna and the remainder of the wave is refracted into the next medium. Loss of strength of the reflected wave form, measured in volts, is used in the evaluation of data. Voids in concrete are indicated by differences in dielectric constants between concrete and air. Advantages of the technique are that large areas of concrete can be rapidly surveyed and internal construction details and defects identified. Also, depth of penetration can be changed by changing the antenna. Disadvantages are that where material differences are small – such as a crack in sound material or a contact delamination – transmission differences are hard to detect and evaluate, material permittivity must be known to determine the interface depth, and results are influenced by the presence of moisture and steel reinforcement.

Infrared thermography is based on heat-transfer principles with subsurface anomalies detected as irregular surface temperature distributions (i.e., discontinuities do not transmit heat as fast). A thermographic scanning and analysis system consists of (1) an infrared scanner head and detector, capable of detecting temperature variations as small as 0.1°C; (2) real-time microprocessor coupled to a monitor that displays different temperature levels as different colors or as gray-tone images; (3) data acquisition and analysis equipment consisting of an analog-to-digital converter, computer with high resolution color monitor, and data storage and analysis software; and (4) image recording and retrieving device.[176] Advantages of infrared thermography include the ability to scan a large area quickly using portable equipment to determine the location and horizontal dimensions of a defect, and testing can be done without direct access to the surface. Disadvantages are that the equipment is expensive, extensive operator experience is required, and it can not provide information on the cause and extent of deterioration or the depth or thickness of a void.

Destructive Testing

Visual and nondestructive testing methods are effective in identifying areas of concrete exhibiting distress, but often cannot quantify the extent or nature of the distress. This is generally accomplished through removal and visual examination of core samples.[177]

In Situ Strength Determinations In general, limited attention is given to the in situ compressive strength of existing concrete structures because 28-day (or older) moist-cured control specimens were used to demonstrate that the required strength was achieved,* and very few concrete structures actually fail.[179,180] However, for a structure that either has had a change in functional or performance requirements, been subjected to a less than ideal operating environment, or is being evaluated to provide an indication of its service life, the in situ strength takes on new meaning. Available methods used to indicate concrete strength include both direct (testing of core specimens) and indirect (ultrasonic pulse velocity, surface hardness, penetration, pullout, and breakoff resistance). The indirect methods measure some property of concrete from which an estimate of the strength is made through correlations that have been previously developed.

When core samples are removed from areas exhibiting distress,[177,181] a great deal can be learned about the cause and extent of deterioration through strength and petrographic studies.[182] Additional applications of concrete core samples include calibration of nondestructive testing devices, chemical analyses, visual examinations, determination of steel reinforcement corrosion, and detection of the presence of voids or cracks. Additional information on the use of core samples is presented elsewhere.[183,184]

The ultrasonic pulse velocity methods are based on the fact that the velocity of sound in a material is related to the elastic modulus and material density.[185] The test equipment consists of a means of producing and introducing a pulse into the concrete (pulse generator and transmitter) and a means of accurately measuring the time taken for the pulse to travel through the concrete to the receiver. Introduction of the pulse can be either direct, semi-direct, or indirect, with direct being

---

* In situ compressive strength of 28-day concrete is normally less (20 to 25%) than the 28-day standard control specimen strength of the same concrete because of different compaction and curing conditions.[178] Also, systematic variation of concrete occurs in a structure because of segregation that can reduce concrete strength at the top of a lift by 15 to 30%.[179]

the preferred method since the maximum energy is transmitted. Because the pulse velocity depends only on the elastic properties of the material and not on the geometry, it is very convenient for evaluating concrete quality. Advantages of the technique are that it is a rapid and cost-effective method for measuring in situ concrete uniformity, the method is totally nondestructive, and it can be utilized to "estimate" concrete strength within about 15 to 20% if a good correlation curve has been developed. Disadvantages are that results are affected by factors such as contact surface smoothness (i.e., coupling), aggregate and concrete mix characteristics, curing conditions, concrete age, and presence of steel reinforcement.

Surface hardness methods impact the concrete surface using a given mass activated by a given energy. The size of the indentation is used to indicate the concrete compressive strength. A special form of surface hardness method is the Schmidt rebound hammer that impacts a spring-loaded weight against the concrete surface and a rebound number is obtained. Concrete strength is determined either from a manufacturer-supplied chart or from a laboratory-generated calibration chart. The primary usefulness of this device is in assessing concrete uniformity in situ, delineating zones (or areas) of poor quality or deteriorated concrete in structures, and indicating changes with time of concrete characteristics. Advantages include minimal user expertise requirements and a large amount of data can be obtained in a short period of time at minimal expense. However, for results obtained in the field the probable variation in compressive strength is ± 25%.[186] Test results are affected by factors such as smoothness of the test surface, specimen geometry and rigidity, moisture condition, and mix characteristics.

Probe penetration techniques involve measurement of the resistance of concrete to penetration of a steel probe driven by a given amount of energy.[187] The most common device of this type is the Windsor Probe, consisting of a powder-activated driving unit that propels a hardened alloy probe into concrete and a gage for measuring depth of penetration. Compressive strength is derived from calibration curves based on depth of penetration. Advantages of the technique are that it is simple to operate and correlates fairly well with core compression strength results. Limitations of the method are related to size of specimen that can be tested (minimum distance between test locations and edge of specimen is 150 to 200 mm; minimum specimen thickness of about three times depth of penetration), and results are influenced by aggregate characteristics.

The pullout test measures the force required to pull an embedded metal insert with an enlarged head from a concrete specimen or structure.[188] The method provides a measure of the shear strength of concrete that is converted to tensile or compressive strength through correlations. The method was developed primarily for determining the in-place strength of concrete during construction because of the requirement to embed or cast the insert into the concrete. Recently, new techniques have been developed in which holes are drilled into the hardened concrete and either pullout inserts or split-sleeve assemblies/wedge anchors are installed and pulled. Advantages of the method are that it is the only nondestructive (actually, semi-destructive) method that provides a direct measure of in-place strength, and it is rapid and economical. It does not measure the interior strength of mass concrete, minor surface repairs are required, and results are affected by aggregate characteristics.

Mix Composition Analysis Questions concerning whether the concrete structure was cast using the specified mix composition can be answered through examination of core samples.[189] By using a "point count" method,[190] the nature of the air void system (volume and spacing) can be determined by examining under a microscope a polished section of the concrete. An indication of the type and relative amounts of fine and coarse aggregate, as well as the amount of cementitious matrix and cement content, can also be determined.[182,191] Determination of the original water-cement ratio is not covered by a standard test procedure, but the original water (volume of capillary pores originally filled with capillary and combined water) can be estimated.[192] A standard method also does not exist for determination of either the type or amount of chemical admixtures used in the original mix. Determination of mix composition becomes increasingly difficult as a structure ages, particularly if it has been subjected to leaching, chemical attack, or carbonation.

3.1.1.2 Conventional mild steel reinforcing

Assessments of mild steel reinforcing materials are primarily related to determining its characteristics (e.g., location and size) and evaluating the occurrence of corrosion.

Steel Reinforcement Characteristics Equipment for determining the location, size, and depth of mild steel reinforcement in concrete are based on the induction principle.[192] Magnetic induction is only applicable to ferromagnetic materials. The equipment consists of a primary coil connected to a power supply that delivers a low frequency (10 to 50 Hz) alternating current and a second coil that feeds into an amplifier circuit. When a ferromagnetic test object, such as a rebar, is introduced near the coils, a much higher secondary voltage is introduced with the amplitude of the induced signal being a function of the magnetization characteristics, location, and geometry of the object. Steel reinforcement between 9.5 and 57 mm (No. 3 to No. 18 bars) and as deep as 30.5 cm can be detected in concrete.[193] Although a variety of probes can be used with a meter, each probe needs to be calibrated. Also, several factors can affect the magnetic field within the range of the instrument (e.g., presence of multiple rebars, bar supports, and temperature).

Corrosion Occurrence The primary distress to which mild steel reinforcement would be subjected is corrosion. Implications of safety and serviceability of structures undergoing appraisal as a result of rebar corrosion should consider effects on three levels: (1) effect on rebars themselves (cross section reduction), (2) development of fine hairline cracks in concrete cover parallel to rebars (indication of deterioration), and (3) structural cracking or voids (preferential corrosion sites). Safety aspects of corrosion depend primarily on the structural form or system of construction; second, on the way in which the geometry of the structural components may be affected; and, third, to a lesser degree, on the total amount of corrosion of the rebars.[194] As shown in Fig. 3.1, a variety of techniques are available establishing both the depth of penetration of carbon dioxide or chloride ions, and the extent of steel corrosion.[195] Methods available for corrosion monitoring and inspection of steel in concrete include (1) visual inspection, (2) mechanical and ultrasonic tests, (3) core sampling with chemical and physical testing, (4) electrical methods, and (5) rate of corrosion probes. The mechanical and ultrasonic methods were discussed previously.

Visual inspection generally provides the first indication of a corrosion problem. Buildup of corrosion products around reinforcement will eventually reach a point where the internal tensile forces generated form hairline cracks in the concrete following the line of reinforcement. Rust staining and spalling also occur as the corrosion progresses. Chipping of concrete cover to expose rebar will indicate the degree of corrosion and may provide clues to its cause.

Chemical analysis of materials obtained during a coring process can be used to obtain information on chloride distribution. Depth of carbonation can be easily determined by treating a freshly broken surface with phenolphthalein — the carbonated portion will be uncolored. Periodic measurements can be used to establish rate of penetration information that is useful in predicting the onset of degradation (e.g., time required for $C\ell^-$ to reach level of steel reinforcement).

Electrical methods use resistance and potential difference measurements of a structure to determine the moisture content, estimate thicknesses, estimate probability of significant corrosion, and indicate the current rate of corrosion. The four-electrode method measures the resistivity of concrete and relates it to the probability of presence of steel corrosion.[196] Four contact points, linearly arranged and equally spaced in the concrete at a depth of approximately 6.4 mm, are used. Alternating current is passed through the outer electrodes and the potential difference determined using the inner electrodes. Depending on the resistivity measured, corrosion is likely, corrosion will probably occur, or corrosion is certain to occur. Potential measurements using a reference copper-copper sulfate half cell can be used to estimate the likelihood of corrosion, but can not indicate the corrosion rate or amount of corrosion present.[197] The half cell consists of a copper rod submerged in a capsule filled with copper sulfate. An electrical connection is made to the reinforcement at a convenient position enabling electrode potentials to be measured at any desired location by moving the half cell over the concrete surface in an orderly manner. Temperature and presence of moisture can influence the results.

Two types of probes are available that can be embedded into concrete to provide an indication of the rate of corrosion.[198] The first type uses two to three electrically isolated short sections of steel wire or reinforcing steel and linear polarization techniques. The second device uses a steel wire or hollow cylinder embedded into concrete to provide cumulative rate of corrosion data from periodic measurements. One of the primary applications of these devices has been to evaluate the effect of rehabilitation procedures on the corrosion rate.

3.1.1.3 Prestressing system components

Prestressing system components in NPPs are routinely inspected to identify the occurrence of corrosion or excessive loss of prestressing force. In the U.S., the condition and functional capability of the unbonded post-tensioning systems must be periodically assessed. This is accomplished, in part, systematically through an ISI program that must be developed and implemented for each post-tensioned concrete containment. Guidelines for development of tendon surveillance programs are provided in Refs. 105, 106, and 157.* Basic components of these documents include sample selection, visual inspection, prestress force monitoring, tendon material tests and inspections, and evaluation of the tendon sheathing fillers (i.e., corrosion inhibitors). Reference 155, presents results of a study conducted specifically to identify aging concerns, if any, for the NPP post-tensioning systems (see Sect. 2.3.2.2), and describes the tendon surveillance program in detail (i.e., examination schedule; tendon sample size requirements; determination of prestressing force; tendon material visual examination and property determinations; and sheathing filler analysis and limits for water content, reserve alkalinity, and concentrations of chloride, nitrate, and sulfide ions). Additional information pertaining to inspection of existing prestressed concrete structures is provided in Ref. 200.

3.1.1.4 Liner plate

Inspections of metallic liners and penetration liner plates of NPP concrete components are performed in compliance with requirements contained in Subsection IWE of Section XI of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code.[201] Contained in this reference are examination and inspection requirements, acceptance standards, repair requirements, and system pressure test requirements for these components. Inspection generally involves a visual examination with supplemental surface (e.g., liquid penetrant or magnetic particle), volumetric examinations (e.g., ultrasonic or radiographic), or an engineering evaluation required if abnormal conditions are noted. Although abrasion or impact damage and fatigue are possible degradation mechanisms for metallic liner materials, corrosion is the largest threat to the ability of the liner to provide a leaktight pressure boundary. Techniques for inspection

* The USNRC has issued a draft regulation[199] that will, when finalized, endorse the requirements for containment examination contained in this reference.

of inaccessible regions of the pressure boundary (i.e., portions embedded in concrete) require development. The integrity of coatings applied to liner materials to inhibit corrosion can be evaluated visually with supplemental testing in suspect areas to measure dry-film coating thickness and determine if holidays are present.

3.1.1.5 Embedment steel

Failure of anchorage embedments will generally occur as a result of either improper installation or deterioration of the concrete within which it is embedded. Visual examinations can be used to evaluate the general condition of the concrete near an embedment and to provide a cursory examination of the anchor or anchor plate to check for improper anchor embedment, weld or plate tearing, plate rotation, or plate buckling. Mechanical testing can be used to verify that pullout and torque levels of embedments equal or exceed minimums required by design. Welds or other metallic components can be inspected using magnetic particle or liquid penetrant techniques for surface examinations, or if a volumetric examination is required, radiographic, ultrasonic, and eddy current techniques are available. Additional information on anchorage to concrete is available elsewhere.[202]

3.1.2 New and Emerging Techniques

Several nondestructive testing techniques in the developmental stage possess potential for application to evaluation of NPP reinforced concrete structures.

3.1.2.1 Magnetic methods

Leakage Flux When ferromagnetic materials are magnetized, magnetic lines of force (i.e., flux) flow through the material and complete a magnetic path between the poles. When the magnetic lines of flux are contained within a ferromagnetic object, it is difficult to detect them in the air space surrounding the member. If the surface of the member contains a crack or defect, however, its magnetic permeability is changed and leakage flux will emanate from the discontinuity.[193] Measurement of the intensity of the leakage flux provides a basis for nondestructive identification of such discontinuities.

A magnetic field method based on leakage flux has been investigated for use in detecting loss of area (> 10%) due to corrosion and fracture of reinforcing bars and prestressing strands.[203] A steady-state magnetic field is applied to the member under inspection, and a scanning magnetic field sensor (Hall-Effect device) is used to detect protuberances in the applied field caused by anomalies such as deterioration or cracks. The technique has been demonstrated in the laboratory relative to its ability to detect varying degrees of deterioration, influence of adjacent unflawed steel elements, type of tendon duct, type of reinforcing steel, transverse rebar configuration, etc. The laboratory results indicated good overall sensitivity to loss-of-section and excellent sensitivity to fracture. Field tests have produced similar results. Additional work has confirmed the ability of the method to detect fractures in reinforcing strands developed due to fatigue loadings.[193] The capability of this technique in testing concrete components with congested reinforcement has not been reported.

Nuclear Magnetic Resonance Nuclear magnetic resonance (NMR) is an electromagnetic method capable of determining the amount of moisture present in a material by detection of a signal from the hydrogen nuclei in water molecules. The method exposes the material examined to a static magnetic field and to a pulsed radio frequency (RF) magnetic field corresponding to the NMR frequency of the nuclei of interest.[204] A prototype NMR moisture measurement system has been developed that utilizes a two-pulse sequence to provide the capability of distinguishing between NMR signals from free moisture and signals from bound hydrogen. The system utilizes a sensor assembly in which the test article is external to both the RF coil and magnet structure. The system specifications indicate that it can detect a moisture content range from 1 to 6% by weight of material with an error of ± 0.2% for depths up to 70 mm and ± 0.4% for depths from 70 to 95 mm. No information regarding the accuracy of this method in massive concrete structures is available.

3.1.2.2 Electrical methods

Capacitance Instruments Experimental investigations at frequencies up to 100 MHz have shown that components (real and imaginary) of the dielectric constants of building materials increase significantly with increasing moisture contents.[205–207] The dielectric constant provides a measure of the ability of a material to store a charge. Several capacitance instruments are available, having various electrode configurations, to measure moisture content of building materials. The electrodes are attached to a constant frequency alternating current source and establish an electric field in the material to be tested. The current flow power loss is used to indicate moisture content. Reference 193 indicates that moisture content of laboratory concrete specimens could be determined to ± 0.25% for values less than 6% using a 10 MHz frequency. Reference 208 suggests that further study is required to establish the reliability of the method. Also, for best results, the instruments should be calibrated to the material.

Polarization Resistance A system has been developed for measuring the corrosion rate of steel in concrete.[209,210] The system is computer controlled and uses the polarization resistance to compute the corrosion current which indicates the rate at which steel is corroding. The polarization technique employs a three electrode system using the specimen as one electrode, a voltage reference as the second electrode, and a counter electrode from which polarizing current is applied to the specimen. The reliability and accuracy of the system are being assessed.

An electrochemical impedance system, somewhat similar to the system described above, has been developed that can be used for locating deteriorated areas and for corrosion rate measurements of rebar embedded in concrete.[211] Corroded and noncorroded rebars are distinguished based on different impedances in a low-frequency region. Corrosion reaction of rebars is indicated by differences in impedances between the low-frequency region and the high-frequency region. The method was investigated using a 20-year-old precast concrete slab reinforced by welded wire fabric. Results showed that the electrical potential best serves as an indicator for a threshold value for corrosion to occur, and that the reciprocal value of polarization resistance of the corrosion reaction correlates well with actual corrosion rate (e.g., at high corrosion rates the potential is low and reciprocal value of polarization resistance is high). The system is undergoing evaluation and is not presently commercially available.

Two instruments, also somewhat similar to the system above and based on the linear polarization method, have been developed to measure the instantaneous corrosion rate of steel in reinforced concrete.[212] The first device uses a counter electrode and reference electrode in conjunction with the working electrode (i.e., corroding rebar). The second device incorporates a guard ring with the counter and reference electrodes to confine the polarizing current being applied to the rebar from the counter electrode. As a result, the area of rebar being measured is known. After polarization of the rebar, the change in potential and input current is obtained, recorded, and corrosion rate determined.

Exploratory work has been completed on use of ultralow frequency alternating current (AC) impedance spectroscopy to characterize corrosion of steel reinforcement in concrete.[213] The principle of operation is that corrosion may be located by imposing a sinusoidal current at a monitoring point on the surface to measure the concrete/rebar impedance as a function of frequency (typically from 10^4 to 10^{-3} Hz). Location of corrosion is estimated on the basis that the distance traveled by the AC wave down the rebar increases as the frequency is lowered. At some characteristic frequency, the AC wave intersects the corroding region resulting in a sudden, perceptible change in measured impedance. Impedance is used to estimate the true polarization resistance of the rebar from which the corrosion rate is determined. It has been demonstrated experimentally that the electrical impedance of the rebar/concrete system is sensitive to the presence of corrosion and that by scanning the reference electrode used to detect the alternating voltage across the surface, areas of corrosion in a reinforced concrete structure can be resolved spatially (i.e., survey structure for corrosion damage). Although the feasibility of this approach has been demonstrated, further development and field testing are required.

Half-Cell Potential Test Using Impulse Radar When performing half-cell potential measurements using the copper-copper sulfate method to detect the potential for corrosion of steel reinforcement, one end of the cell is connected to the reinforcement and the other end is placed in direct contact with the test area. The most recent addition to this technique is the capability to measure the resistance to passing a charge through the concrete. Because of the "electric potential" of the concrete, it is of interest to correlate the return data from the impulse radar to that of the copper-copper sulfate half cell. The theory behind this concept is that as the resistance of the concrete increases, the dielectric constant of the concrete decreases. The opposite occurs when the resistance of concrete decreases. The change in dielectric constants can be detected by the impulse radar. This method is still in the experimental stage, and at present, no literature is available to assess its application to massive concrete structures.[214]

3.1.2.3 Ultrasonic methods

Laser Ultrasonics A pulsed Nd-YAG laser system has been developed to detect voids, delaminations, and defects in asphalt and reinforced concrete materials.[215] The laser is used to introduce ultrasonic waves in the test sample and a He-Ne output laser interferometer is used to measure the minute movements in the surface caused by the ultrasonic waves. The system can be operated in three modes: direct transmission, near-coincident, and same surface-separated. Moderate success was obtained in the laboratory in detecting steel reinforcement and simulated voids in sections up to 200-mm thick. Development of a portable more powerful laser system and improved signal enhancement and recognition capabilities (i.e. overcoming the effects of surface texture and poor reflectiveness of asphalt and concrete) are required before the system will be suitable for field use.

Acoustic Tomography Imaging A preliminary study has been conducted to determine the feasibility of using acoustic tomographic imaging for defect location and identification of the interior of concrete sections.[216] The method uses a large number of ultrasonic pulse readings obtained on the exterior of a concrete object as input to a tomographic reconstruction computer program to create a map of the velocities of the interior object. From the velocity profile, the location, shape, and type of internal feature can be identified. Preliminary results obtained from medium-sized concrete specimens in which various types of objects simulating flaws were embedded are encouraging (i.e., objects such as steel bars, voids, and zones of low density were identified). Work is presently underway to modify and optimize the computer software to meet the specific needs for examination of mass concrete, and to improve the data acquisition equipment and transducer characteristics.

3.1.2.4 Pulsed infrared imaging method

A preliminary study has been conducted in which thermal pluses and infrared images were used to inspect subsurface features of concrete.[217] The equipment consisted of a controlled heat supply (i.e., bank of tungsten-filament lamps) and a thermal camera to capture the changing surface temperature of the concrete. The depth of concrete observed was controlled through the length of time that the heaters were activated. The method was able to identify the presence and location of steel reinforcement and it was theorized that corrosion could be detected because of the relatively poor conduction characteristics of corrosion products. A commercial version of a pulsed infrared imaging method has been developed for nondestructive examination of metallic and composite materials.[218]

3.1.2.5 Ground-probing radar method

Ground-probing radar[164] is the electromagnetic analog of sonic and ultrasonic pulse-echo methods. It is governed by the process involved in the propagation of electromagnetic energy through materials of different dielectric constants. The systems operate by transmitting a short pulse that is followed by a "dead time" in which the reflected signals are returned to the antenna. As the radiated pulses travel through a material, different reflections will occur at interfaces that represent changing dielectric properties. Each reflected electromagnetic pulse arrives back at the receiving antenna at a different time that is governed by the depth of the corresponding reflecting interface and the dielectric constant of the intervening material. A receiver circuit reconstructs the reflected pulses at an expanded time scale using the time-domain sampling technique. The resulting replicas of the received radar signals are amplified and further conditioned in the control unit before they are fed to an output. Applications include detection of voids under concrete, delamination of concrete, scour, thickness measurements, and subsurface soil exploration. Skill is required in interpreting the various radar signatures. With refinements such as increased resolution of the antenna, ground-probing radar may have potential application to inspection of massive concrete structures such as NPP basemats.

3.1.2.6 Ultraviolet radiation method

A simple and rapid test has been developed that provides conclusive evidence of the presence of alkali-silica reactions (ASRs) in in situ concrete structures.[219] The method consists of applying a dilute solution of uranyl acetate on the suspected concrete surface, and viewing the treated surface under ultraviolet light in a dark background. The presence of ASR is revealed by a yellowish-green fluorescent glow caused by incorporation of the solution in the ASR gel product. Some user experience is required and there are precautions associated with handling the uranyl acetate solution.

3.1.3 Commentary on Applicability of Methods to NPP Concrete Structures

Nondestructive testing techniques that have been used on concrete structures include visual, acoustic, ultrasonic, electrical resistivity, electromagnetic, vibration, strain gage, dye penetrant, radar, magnetic particle, infrared thermography, and radiography. The use of any of these methods depends largely on the sensitivity desired, adaptability to field use, cost of instrumentation, and survey requirements. In general, most methods detect defects by the observation of changes in the response of the interrogation medium. The most common forms of defects that can occur in concrete components include cracking, surface scaling, corrosion of steel reinforcement, honeycombing or air pockets, popouts, surface deposits, chemical attack, wear, and erosion. For steel structures, the most common forms of deterioration include corrosion (uniform, pitting, galvanic, or dissimilar metal) and cracking (fatigue or embrittlement). Table 3.3

provides a summary of recommended testing methods to identify and assess the extent of damage resulting from factors that can degrade reinforced concrete.[214]

The ideal nondestructive testing technique for reinforced concrete structures would provide information about the existence and location of deterioration, corrosion, and structural damage.[220] Rates of corrosion are important in estimating service life. Given the variables of exposure, geometry, and materials that are present in NPP concrete structures, there is no single test method that can provide all the desired information. Best results are obtained through use of a combination of testing methods. Many of the nondestructive testing methods require the development of correlation curves that relate the material property of interest to a measured parameter (e.g., compressive strength and rebound number). Best correlation results are obtained when destructive and nondestructive testing are done in tandem using representative materials obtained from the structure of interest.* Detectability functions that relate flaw characteristics to probability of detection require development. Many of these techniques operate on the surface of a structure where the properties can be significantly different relative to internal concrete due to effects such as carbonation. This is particularly true for NPP reinforced concrete structures that tend to be more massive than conventional civil engineering reinforced concrete structures. No "field-ready" nondestructive testing method was identified that can be used to assess the in situ condition of the post-tensioning tendons with respect to occurrence of corrosion. Some encouraging results have been provided by the use of magnetic field disturbance to sense discontinuities (i.e., wire breaks) in prestressing tendons, but considerable development is still required.[203] Also, due to the massive size, steel reinforcement congestion, and accessibility problems associated with the basemats of NPPs, no nondestructive testing techniques are currently available that can reliably inspect these structures. Nondestructive testing methods suitable for inspection of inaccessible portions of the metal pressure boundary where it is embedded in concrete have not been identified. Ultrasonic-based methods have been successfully applied to the interface region between the pressure boundary and concrete to detect corrosion effects, but they are only capable of detecting sharp pits located within a distance equal to about four times the pressure boundary thickness (i.e., about 25 mm for the liner of reinforced concrete containments).[222]

Despite the limitations associated with many of the nondestructive testing methods, their proper use and application provides vital input for accessing the structural condition of reinforced concrete structures. Nondestructive testing methods provide a vital element in an aging management program to maintain required safety margins, (i.e., they aid in the diagnosis of problems, specifying repair activities, and quantifying the extent of adverse conditions and deterioration).

3.2 CONDITION ASSESSMENTS

Determining the existing performance characteristics and extent and causes of any observed distress is accomplished through a condition assessment. Several documents have been prepared to aid in conduction of condition surveys: *Guide for Making a Condition Survey of Concrete In-Service*;[223] *Causes, Evaluation, and Repair of Cracks in Concrete*;[224] *Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions*;[225] *Strength Evaluation of Existing Concrete Structures*;[226] *Guide for Concrete Inspection*;[227] *Guidelines for Structural Condition Assessment of Existing Buildings*;[163] *Strength Evaluations of Existing*

* In many of the NPP concrete structures (e.g., containment and biological shield) it may not be possible to obtain the samples required to develop the correlation curves. For this situation statistical data have been developed for use with techniques commonly used to indicate concrete compressive strength (i.e., ultrasonic pulse velocity, rebound hammer, breakoff, pullout, and Windsor probe methods).[221]

Buildings;[147] and *Guide for Evaluation of Structures Prior to Rehabilitation.*[162] General information pertaining to condition surveys of reinforced concrete structures, current inspection requirements for NPP reinforced concrete structures, and needed developments follows.

3.2.1 Condition Surveys of Conventional Reinforced Concrete Structures

Common in the condition assessment approaches proposed by several of the references noted above is the conduct of a field survey involving visual examination and application of nondestructive and destructive testing techniques, followed by laboratory and office studies. Results of the field survey, and laboratory and office studies are analyzed to either verify the adequacy of the structure to successfully meet its functional and performance requirements, or to provide data and information necessary for implementation of a maintenance or repair activity. Figure 3.2 presents a flow diagram of one procedure proposed for use in a general structural assessment and evaluation of an existing building.[163]

3.2.1.1 Field survey

Visual Survey General direction on conducting surveys of concrete structures is provided in Ref. 228 from which information presented in the balance of this section has been abstracted. 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. This initial visual examination is used to document easily obtained information on instances that can result from or lead to structural distress such as cracking, spalling, leakage, and construction defects such as honeycombing and cold joints in the concrete. The instances of cracking, spalling, leakage, delamination, efflorescence, chemical attack, or structural distress are observed and documented.

Crack Survey Cracking can be due to effects such as shrinkage of the concrete, thermal, effects, corrosion of embedded steel, alkali-aggregate reaction, or structural loads. 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 are noted. Photographs or video recordings are made to provide a permanent record of this information, and notes are made on the survey sheets to indicate the area photographed.

Delamination Plane Survey After the visual survey has been completed, the need for additional surveys may be indicated. One such survey is for internal delaminations that are not visible. These internally cracked regions are usually caused by corrosion of embedded metals or internal vapor pressure. For horizontal surfaces, chains can be dragged over the concrete surface to quickly locate hollow sounding delamination planes. Hammers or lengths of steel bar impacted against the surface sometimes result in more distinctive differences in tone. On vertical or overhead surfaces, a hammer or bar must be used to locate the hollow sounding areas.

If a more detailed study is then warranted, the results of the visual and delamination surveys are used to select portions of the structure that will be studied in greater detail. The detailed study on the suspect areas can include measuring half-cell potentials, removing and testing of concrete and steel samples, removing concrete powder samples for chloride analyses, locating embedded steel, additional nondestructive testing evaluations and, in extreme cases, load testing.

Corrosion Survey Concrete normally provides a high degree of corrosion protection to embedded metals because of the high pH environment within the concrete. Steel in high pH concrete is polarized anodically by a thin protective film of iron oxide that forms on its surface. However, the passivity of this film can be disrupted by a reduction in the pH value of the moisture within the concrete due to carbonation or by the ingress of sufficient quantities of chloride ions, or other ions, to the steel surface.

This disruption of the protective film causes some steel areas to become more anodic, creating an electrochemical cell with the more cathodic unaffected steel, causing corrosion to occur. The electromotive force in these cells depends on the pH value, the concentration of chloride, and the moisture content adjacent to the anodic steel and on the transfer of oxygen through the concrete to the cathodic areas. 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 corrosion activity probability of embedded reinforcing steel (or other metals) can be made. If sufficient readings are taken on a grid pattern, a diagram can be prepared that resembles a contour map. On such a diagram, points of equal electrical potential are connected by isopotential lines, permitting areas of high potential or high corrosion probability to be readily identified.

Reinforcing Bar Location Survey A pachometer survey may be performed as part of the detailed study. The pachometer consists of a search coil that emits a magnetic field and is connected to electronic circuitry that senses any disruption in this field. A display dial is graduated to indicate the depth of the steel reinforcing bars, if the size of the bar is known. The equipment can be calibrated for depth using a reinforcing bar and various thicknesses of an inert material, or on the job by drilling or coring to the depth of the steel.

Where there is evidence of severe corrosion, the steel bar should be uncovered to allow visual inspection and measurement of cross-sectional area loss. In many cases, old delaminations or spalls will have already accomplished most of the concrete removal, aiding in the bar examination. A concrete coring rig may also be used to retrieve steel samples.

Concrete Chloride Survey Concrete power samples or cores should be removed from the structure where significant chloride penetration is suspected. The samples are usually removed from several depths, extending to and beyond the embedded outer layer of reinforcing steel. If the concrete contains more acid soluble chloride than about 300 ppm by weight (0.7 kg/m^3), as measured by ASTM C1152,[229] it is considered to contain sufficient chloride to support electrochemical corrosion of embedded steel when in a moist environment that has oxygen availability. Particularly susceptible to damage by chloride ions are dissimilar metals. Where aluminum or galvanized electrical conduit has been embedded in concrete and is in contact with the reinforcing steel, the conduit will corrode rapidly, acting as an anode to the steel.

In elements exposed to a marine environment, most of the concrete elements will eventually contain significant amounts of chloride. Water intake and discharge structures present unique and severe environmental conditions where water with dissolved chlorides, sulfates, and other minerals and salts is routinely in contact with the concrete. Some regions are always wet while other regions may experience wet and dry periods. Experience has shown that regions experiencing wet and dry cycles exhibit the greatest distress.

Concrete columns and walls contain capillary channels that can cause saltwater to wick upward for several feet. The columns and walls in such structures can exhibit delaminations and spalls caused by corrosion due to this upward moisture movement. Chloride contents should even be determined in indoor structures if cracking patterns suggest reinforcing steel corrosion. Numerous high-rise office buildings in the U.S. have suffered severe corrosion due to the inclusion of concrete admixtures containing chloride ions.

3.2.1.2 Laboratory tests, office studies, and design verification

Laboratory Tests During the site survey, samples of concrete and steel are collected for testing within the laboratory. Concrete cores are removed for examination of the steel, compressive strength determination, petrographic examination, or laboratory sampling for chloride contents or other chemical ingredients known to create distress. The samples taken from a structure may be investigated using many different techniques, such as

> Petrographic methods: thickness, distribution of cement, aggregate studies, estimation of water-cementitious materials ratios, air-void distribution, types of distress, recognition of unstable aggregates, deterioration mechanisms, and age at which cracking occurred.
>
> Chemical techniques: chemical constituents of the cementitious materials, characteristics of the paste and aggregates, presence and quantity of chemical admixtures, quantification of chemical compounds within the cement paste, efflorescence, and carbonation effects.
>
> Concrete strength testing: compressive strength, modulus of elasticity, tensile strength, flexural strength, and bond strength of patches or overlays.
>
> Steel materials: yield and ultimate strengths of reinforcing bars per ASTM A615,[31] or prestressing tendons per ASTM A421.[39]

Petrographic examination[182] can determine the quality of the cement paste, whether or not the aggregates are unstable, if the concrete contains entrained air, or if it has been damaged by freezing and thawing, chemical attack, or other deterioration mechanisms. Petrographic examination of core samples is recommended for any concrete that may be showing distress or when the concrete components or durability of the concrete is not known. A trained petrographer experienced in concrete deterioration is needed for this evaluation. The petrographic examination is done on a section of concrete that is usually sawed vertically through the center of the core, then polished to a high luster. Examination is done both with the naked eye and through a low power microscope.

Office Studies 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. Chloride ion results are plotted vs 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 to the effects of deterioration, are identified and appropriate calculation checks made. If the calculations are inconclusive, suitable load testing may be indicated.

Design Verification Based on physical test results, chemical analysis of materials, and present condition of the structure, a redesign of various questionable elements should be accomplished to verify compliance with current codes and design requirements. Based upon the initial survey results, additional testing such as impact-echo, linear polarization, infrared thermography, X-ray, or numerous other destructive or nondestructive tests may be appropriate.

From design documentation and from measurements made in the field, structural analyses may then be accomplished. Compressive and tensile strengths and elastic properties of materials may be determined from laboratory measurements and used in the structural analyses. These analyses may identify distress in the structure that has been caused by structural overload and indicate safety factors.

3.2.1.3 Report

After all of the field and laboratory results have been collected and studied, and all calculations have been completed, the report is prepared. It should start with an introduction stating how and by whom the work was instigated, who did the investigation, why the investigation was performed, and when. A brief description of the structure should be included. A photograph of the entire structure is helpful.

A description of the field investigation follows, with a short explanation of the various testing techniques used and a short summary of findings. Photographs of significant features and exploratory excavations should be included, along with maps showing crack, delamination, and spall locations, and where core and powder samples were removed.

The testing techniques used and the results determined in the laboratory should then be described, and the results interpreted. Any structural analyses performed should be presented and discussed. A general discussion that summarizes all of the findings and characterizes the condition of the structure should follow. Any unsafe conditions should be identified, and temporary corrective actions suggested.

The final section of the report should discuss the possible repair techniques, and which appear to be appropriate in view of the results of the investigation and the environment of the structure. Appendices may be added if a complete compilation of the data is desired. A condition survey done in this manner will provide information in which a sound, economical repair specification can be based.

3.2.2 Condition Assessments of NPP Reinforced Concrete Structures

ISI programs for NPPs traditionally have focused on the periodic inspection of safety-related components at various intervals or frequencies depending primarily on component importance; however, passive components such as safety-related concrete structures have generally received minor attention in the overall ISI strategy.

The concrete containment vessel is generally the only structure included in a defined ISI program. This has resulted from the requirement contained in General Design Criteria 53, "Provisions for Containment Testing and Inspecting," to 10 CFR Part 50 that the reactor containment be designed to permit periodic inspection of all important areas and an appropriate surveillance program.[4] Condition assessments of the containment are conducted in conjunction with the performance of periodic Type "A" leak-rate tests required by Appendix J to 10 CFR Part 50. This only includes a general visual inspection of the accessible interior and exterior surfaces of the containment structures and components. For conventionally-reinforced (non-post-tensioned) concrete containments, a general visual inspection currently is the only in-service inspection that is required for the concrete structures.

Examinations of the unbonded post-tensioning systems are conducted at regular intervals to assess the state of the hardware as well as the level of prestressing force in the containment. These examinations are mandated by the Plant Technical Specifications. Detailed examination procedures and acceptance criteria are provided in Regulatory Guide 1.35 (Ref. 105), and the ASME Boiler and Pressure Vessel Code Section XI, Subsection IWL.[157] These examinations typically consist of a visual examination of accessible hardware; measurement of tendon end anchorage force; measurement of prestressing wire (or strand) strength and ductility; and analysis of sheathing filler for water content, concentration of corrosive ions (i.e., chlorides, sulfides, and nitrates), and reserve alkalinity. The visual assessment considers cracks in the bearing zone concrete; cracking, corrosion, and deformation of load bearing metallic hardware; the presence of water within the end caps or ducts; and the degree of sheathing filler coverage as well as sheathing filler leakage.

Currently under development by the American Concrete Institute (ACI) is a guideline for use in evaluation of existing nuclear safety-related concrete structures, other than the containment.[230] Contained in the guideline is information related to determination of critical structures for evaluation, potential degradation mechanisms, inspection techniques and frequencies, and evaluation of results. Two evaluation methods are proposed: selective and periodic. The selective evaluation method is used to investigate a specific structure or subelement to provide existing materials or condition data, input for structural reanalysis or design modification, or some other specific purpose. Generally the desired end result is predefined for this method. The periodic evaluation method is used to demonstrate satisfactory performance, identify the presence and activity of age-related degradation, or provide data and information to support continued service (i.e., aging management program). Desired end results from this method are generally not predefined. Evaluation criteria are provided in terms of acceptance without further evaluation, acceptance after review (i.e., additional inspection and testing to identify cause, activity, and effect of deterioration), and further evaluations required (i.e., more extensive application of testing and analytical methods to assess current capabilities and develop a remedial measure program, when required). Also included in the guideline are suggested periodic evaluation intervals (i.e., generally at 5- or 10-year intervals, but shortened if deterioration has occurred), qualification requirements of the evaluation team, and considerations for damage mitigation (i.e., repair, monitor at increased frequency, or replacement).

3.2.3 Commentary on Condition Assessments

Condition assessment methodologies have been used effectively to determine the performance characteristics and the cause and extent of observed distress in reinforced concrete structures. The common approach is to start with a general visual examination of accessible concrete surfaces. These examinations provide a rapid and effective means for identifying areas of distress because most of the degradation mechanisms of concern for reinforced concrete structures manifest themselves in the cover concrete in the form of cracks, spalls, or stains. For structures that are essentially inaccessible for visual examinations, either the surfaces must be exposed for examination, or an indirect approach must be used (e.g., evaluation of the environment adjacent to the structure for its potential to cause degradation). Identified suspect areas are then investigated in more detail through nondestructive, destructive, or a combination of these test methods. Best results are generally provided through application of a combination of testing methods. Although much has been published on the basic methodologies and mechanics of conducting condition surveys on reinforced concrete structures, only limited information is available on classifying or rating the damage state. Where results have been presented, they generally were "application specific" because of the uniqueness of each concrete structure and the environment to which it is subjected. Some general information has been provided in which observed deterioration in the form of cracks, joint deficiencies, surface damage, changes in member shape, and texture features have been ranked through an appearance parameter (e.g., longitudinal cracks are rated through width measurements).[58] Condition assessment programs that address the unbonded tendon systems in NPP post-tensioned concrete containments generally have been shown to be effective.[155] With the exception of the cursory visual examination conducted in conjunction with periodic leak-rate tests, no specific condition assessment requirements exist for the NPP reinforced concrete containments or other safety-related concrete structures. Due to the importance of condition assessments in effectively managing aging of structures and the likelihood for incidences of degradation to increase as the structures age, it seems prudent to periodically assess the condition of these structures. One of the conclusions of a report in which assessments of the in-service condition of structures at several NPPs were performed was that these structures need to be periodically inspected and maintained.[154] The importance of periodic inspections, evaluations, and maintenance in managing the aging of NPP concrete structures was also noted in Ref. 27 where results of observations made at several U.S. and foreign NPPs were presented. ACI 349 (Ref. 230) has drafted guidelines for evaluation of safety-related concrete structures relating damage state (or current condition) to a recommended action defined through acceptance limits. However, further development of damage assessment criteria is required for concrete structures to relate parameters such as damage state to the environment (e.g., extent of remedial action based on a relationship between crack width and chloride ion content of its environment).

3.3 REMEDIAL MEASURES

Reinforced concrete structures almost from the time of construction will start to deteriorate in one form or another due to exposure to the environment (e.g., temperature, moisture, cyclic loadings, etc.).[195] 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 3.3 presents a relationship between concrete performance, time, and repair. Termination of a component's service life occurs when it no longer can meet its functional and performance requirements. As noted by the deterioration of many roadways and bridges in the U.S.,[220] this often occurs prior to achieving the desired service life.

It was noted in the previous section that ISI techniques are available that can indicate the occurrence and extent of age- or environmental-stressor-related deterioration. Periodic application of these techniques as part of a condition assessment program can monitor the progress of deterioration. Results obtained from these programs can be used to develop and implement a remedial action prior to the structure achieving an unacceptable level of performance. Depending on the degree of deterioration and the residual strength of the structure, the function of a remedial measures activity may be structural, protective, cosmetic, or any combination of these three functions. Basic components of a remedial measures program include diagnosis (damage evaluation), prognosis (can repair be made and is it economical), scheduling (priority assignments), method selection (depends on nature of distress, adaptability of proposed method, environment, and costs), preparation (function of extent and type of distress), and application.[231] Figure 3.4 indicates the basic steps of a typical repair strategy.[232]

3.3.1 Initial Repair Considerations

The first step in any repair activity is a thorough assessment of the damaged structure or component including evaluation of the (1) cause of deterioration, (2) extent of deterioration, and (3) effect of deterioration on the functional and performance requirements of the structure or component. Basic elements of the assessment include (1) preplanning and accumulation of background data (e.g., age, previous condition surveys, design documents, as-built drawings, materials data sheets, etc.), (2) visual examination, (3) in situ and laboratory testing, and (4) evaluation of collated survey data and determination of cause(s) of deterioration. From this information a remedial measures strategy is developed based on consequence of damage (e.g., affect of degradation on structural safety), time requirements for implementation (e.g., immediate or future safety concern), economic aspects (e.g., partial or complete repair), and residual service life requirements (e.g., desired residual service life will influence action taken).[233] Basic remedial measures options include (1) no active intervention; (2) carry out repairs to restore deteriorated or damaged parts of structure to a satisfactory condition; (3) if safety margins are presently acceptable, take action to prevent deterioration from getting worse; and (4) demolish and rebuild all or part of structure. Quite often options (2) and (3) are considered jointly.

3.3.2 Typical Remedial Measures Techniques for Reinforced Concrete Structures

Application of the basic remedial measures strategy includes the repair of damaged concrete and mitigation of the cause of deterioration.

3.3.2.1 Techniques and materials for repair of damaged concrete

Deterioration of reinforced concrete structures generally will result in cracking, spalling, or delamination of the cover concrete. Of these, surveys of general civil engineering construction[234] and NPP structures[27,56,235] indicated that cracking was by far the most frequently occurring problem. Seepage of water through construction joints or cracking was also reported in the NPP surveys as well as the presence of honeycombs and voids.

<u>Crack Repairs</u> Cracking in concrete structures can be expected to occur for a number of reasons, including plastic and drying shrinkage,. thermal effects, fatigue, reactive aggregates, and excessive loads, Table 3.4.[56] All concrete structures will have cracks that can be classified into two categories: microcracks and macrocracks. Microcracks form within the cement paste adjacent to the aggregate particles and are discontinuous, very narrow, and require no repair action. The microcracks are important from the standpoint that under increased loadings they become wider and propagate, and can eventually reach a size (i.e., macrocracks) sufficient to deteriorate the concrete, accelerate corrosion of embedded steel, or produce leakage. Macrocracks are important to service life.

<u>Crack Width and Durability</u>

Two theories have been proposed regarding how cracks reduce the service life of reinforced concrete structures.[236] The first is that they permit access of chloride ions, moisture, and oxygen to the steel, not only accelerating the onset of corrosion, but providing space for the deposition of corrosion products. The second theory agrees that cracks may accelerate the onset of corrosion at localized regions on the steel, but after a few years of service there is little difference between cracked and uncracked concrete. Factors that may influence corrosion occurrence include crack arrangement, width, depth, shape orientation with respect to steel, intensity, origin; type of structure; and service environment.[237]

Both the crack geometry (i.e., width and depth) and environmental exposure condition are important to the durability of reinforced concrete structures. A wide surface crack that quickly narrows with depth may not be as detrimental as a narrower surface crack that penetrates to the steel reinforcement. For structures that are located in controlled environments, cracks are primarily an aesthetic concern. Researchers have provided a number of relationships between maximum permissible crack width and exposure condition. Table 3.5 presents a summary listing of permissible crack widths (and environmental factors) that have been proposed to prevent corrosion.[56] Larger cracks widths increase the probability of corrosion,[238] however, as noted in Ref. 224, the values of crack width are not always reliable indicators of corrosion and the deterioration to be expected. In general, Ref. 56 provides the following guidance: for severe exposure to deicing chemicals or for water tightness, widths $\leq$ 0.1 mm; for normal exterior exposures or interior exposures subjected to high humidities, widths ≤ 0.2 mm; for internal protected structures, widths ≤ 0.3 mm; and for structures containing chemicals or fluids that must remain leaktight, widths ≤ 0.05 mm. These limits are also consistent with those recommended by the Commission of European Communities.[52]

Selection of Crack Repair Technique

After identifying that the crack is of sufficient size to require repair, it is important to determine if the crack is active or dormant. Active cracks are those for which the mechanism causing cracking is still at work, whereas dormant cracks are those caused by a condition that is not expected to recur. A basic procedure for use in identifying the cause of cracks has been developed:[239]

Step 1 Examine the appearance and the depth of the cracking to establish the basic nature of the occurrence, such as pattern or individual cracks, depth of cracks, open or closed cracks, and extent of cracking.

Step 2 Determine when the cracking occurred. This step will require talking with the individuals who operate the structure and possibly those involved in the construction.

Step 3 Determine if the cracks are active or dormant. This step may require monitoring the cracks for a period of time to determine if crack movement is taking place. Also, attempt to determine if the crack movement detected is growth or simply cyclical opening and closing such as caused by thermal expansion. Cracks that are moving but not growing should be treated as active cracks.

Step 4 Determine the degree of restraint. This step will require a thorough examination of the structure and the construction drawings, if available. Both internal restraint (caused by reinforcing steel, embedded items, etc.) and external restraint caused by other elements may be present. A checklist for determining the cause of cracking is presented below. Using the checklist, eliminate as many potential causes as possible. If more than one potential cause remains, the final determination may require a laboratory analysis of concrete samples or a detailed stress analysis.

- Check for major errors in design;
- Check easily identifiable causes, such as
  corrosion of reinforcement,
  accidental or impact loading,
  poor design detailing, and
  foundation movement; and
- Check other possible causes, such as
  incidents during construction,
  shrinkage induced stresses,
  volume changes,
  chemical reactions,
  moisture changes, and
  freezing and thawing.

Having established the cause of cracking, several questions should be addressed:

- Is repair necessary? Repair of cracking caused by expansion products of internal chemical reactions may not be necessary.
- Should repair be treated as spalling rather than cracking? If the damage is such that loss of concrete mass is probable, treatment of the cracks may not be adequate. For example, cracking due to corrosion of embedded metal or freezing and thawing would be better treated by removal and replacement of concrete than by one of the crack repair methods.
- Is it necessary that the condition causing the crack be corrected? Is doing so economically feasible?
- What will be the future movement of the crack?
- Is strengthening across the crack required?
- What is the moisture environment of the crack?

With these questions answered, a repair technique can be selected. Figures 3.5 and 3.6 present repair methodologies for dormant and active cracks, respectively.[239] Typical properties of epoxy and polyester chemical grouts used to repair concrete are provided in Table 3.6.[240] General guidance on crack repair options including perceived durability is presented in Table 3.7.[56] Detailed descriptions of techniques available for repair of dormant or active cracks in reinforced concrete are available elsewhere.[13,56,223,239,241] Final selection of a repair technique should take into account durability, life-cycle costs, and labor skill and equipment requirements.

Spall Repairs Spalls can occur due to impacts, corrosion of embedded metals, erosion, or problems such as alkali-aggregate reactions, freeze-thaw, and fire exposure. Surface preparation is critical to a successful spall repair. The concrete substrate must be sound, and the exposed surface dry and free of grease, oil, and loose particles. Suitable techniques for surface preparation include use of small chipping hammers (followed by abrasive blasting, and removal of dust and chips by compressed air) and high-pressure water blasting. If steel reinforcement is exposed during the removal of degraded material, the excavation should be extended so that the steel will be enclosed in the patch material. If the steel reinforcement is corroded, the corrosion products should be removed and the steel coated with a barrier material such as epoxy resin, or a high electrically-resistant patch material utilized.[56] Corrosion-inhibiting admixtures (e.g., calcium nitrate and organic-based) can also be included in the concrete patch material.* Shallow spalls ($\leq$ 20 mm) are generally repaired using portland cement-based mortar materials. Polymer concretes containing epoxies or methyl methacrylates have also been successfully utilized. Deep spalls are treated in a similar manner to shallow spalls except coarse aggregate is added to the repair material. To ensure good durability of the repair it is important that the repair material have mechanical and physical properties similar to the in-place concrete and that it is properly consolidated and cured. For mass concrete requiring extensive replacement of material, the repair patch may be built up in two or more layers to prevent excessive heat build-up due to cement hydration. Fly ash can be used as a partial replacement for the cement to reduce the maximum temperature build-up of the repair mass. Also, if the spall to be repaired is located in a vertical or overhead surface, special precautions are required because of the increased difficulty in application of material due to gravity effects. Generally, dry packing is used in which a very harsh mix (i.e., dry) is applied and compacted using a blunt instrument.[242] Typical properties of materials commonly used for spall repairs are presented in Table 3.8.[243] General guidance on spall repair options including perceived durability is provided in Table 3.9.[56] Additional information on spall repair techniques is available elsewhere.[13,56,239,242]

---

* Additional information specifically addressing remedial measures for corrosion damaged concrete is presented in the next section.

Delamination Repairs Delaminations are horizontal voids in concrete domes or slabs that commonly occur due to corrosion of steel reinforcement or separation of concrete layers that do not develop adequate bond. Spalls will occur if the delaminations are not repaired. Delaminations can be repaired by removal and replacement of the delaminated concrete using procedures similar to those for repair of spalls. In areas where removal of concrete is not required, the delaminated area can be repaired by injection of an epoxy resin. Several holes are made into the delamination using either a drill with a vacuum attachment to remove the fines developed or by coring. If water is used in either process, the concrete should be permitted to dry prior to epoxy injection under low pressure or by gravity feed. Dowell pins can be used to enhance shear transfer. Additional information on delamination repair techniques is available elsewhere.[13,56,239,241]

Water Seepage Repair Water seepage through construction joints and cracks may result in leaching of the concrete, entry of aggressive environments into the concrete matrix, or unacceptable flow of fluids either into or out of a facility. Long-term reactions that require the presence of moisture such as efflorescence, sulfate attack, or alkali-aggregate reactions also may initiate as seepage into the concrete occurs. Implementation of a repair activity, therefore, can prevent possible future deterioration and the unacceptable migration of fluids either into or out of the facility.

A properly implemented repair procedure first will identify the source and then repair the path. Chemical grouting using silicate, acrylamide, lignin, or resin (e.g., epoxy, polyester, and urethane) systems is the most effective repair technique when moisture is present. The chemical grouts consist of solutions of two or more chemicals that react to form a gel or solid precipitate, as opposed to cement or clay grouts that consist of suspensions of solid particles in a fluid. The reaction of the chemical grout, that may be purely chemical or physiochemical, produces a decrease in fluidity and a tendency to solidify and form occlusions in channels or fill voids in the material.[241] Reaction of the chemical grout can be in the form of either soft flexible, semirigid, or rigid gels. When the seepage is intermittent and the path through the concrete periodically dries, it can be injected with epoxy resins, or water can be incorporated into a urethane injection system to promote expansion and curing to form a flexible foam material.

Honeycomb and Void Repair Nonvisible voids such as rock pockets, honeycomb, or excessive porosity can be repaired by drilling small diameter holes to intercept the voids; determining the extent and configuration of the void system by injection of compressed air or water into the void system, or by visual inspection using a borescope; and, depending on the magnitude of the delamination, injecting either epoxy resin, expansive cement grout or mortar, or epoxy-ceramic foam. Proper injection of the cement grouts requires prewetting of the substratum with excess water removed prior to injection.

3.3.2.2 Remedial measure techniques for corrosion-damaged concrete

Corrosion of steel reinforcement is by far the greatest threat to the durability of reinforced concrete structures. A high percentage of the corrosion damage that has occurred to general civil engineering concrete structures has been the result of insufficient planning, incorrect estimation of environmental actions, and bad workmanship.[195] As a consequence, many of these structures have had to be repaired, or under extreme conditions, removed from service because of corrosion of steel reinforcement. Fortunately, incidences of corrosion of NPP concrete structures have been limited,[56] probably due to more detailed considerations associated with material selection, construction workmanship, and effective use of quality assurance/quality control procedures. However, the history of these structures is somewhat limited and as they age, incidences of corrosion can be expected to increase.

Preventative Measures for In-Service Concrete

Corrosion of steel reinforcement can be expected if the concrete is sufficiently moist and either carbonation or chlorides have reached the surface of the steel. The most cost effective approach to treating corrosion of reinforced concrete structures obviously is to provide preventative protection, or if necessary, intervention early in the process. The application of barriers in the form of sealers, coatings, or membranes to exposed surfaces provides one commonly used measure of intervention.

Sealers are liquids applied to the surface of hardened concrete to either prevent or decrease the penetration of liquid or gaseous media (e.g., water, carbon dioxide, or aggressive chemicals).[244] A number of materials have been applied to concrete (e.g., boiled linseed oil, sodium or potassium silicates, stearates, silicones, asphaltic emulsion, and cementitious formulations). Five categories of sealers have been found to be effective in bridge deck applications – polyurethanes, methyl mathacrylates, certain epoxy formulations, relatively low molecular weight siloxane oligomers, and silanes.[245] Of these, the silanes and oligomers are most commonly used.* Newer formulations of these materials penetrate the concrete surface to some degree, but still permit the transmission of air or water vapor. Therefore, coatings also have to be applied to protect against carbonation.

Coatings and membranes differ from sealers in that they are applied in some thickness, generally measured in hundredths of a millimeter, and generally do not penetrate the concrete. Coating types include epoxy resins, polyester resins, acrylics, vinyls, polyurethanes, and cementitious materials. Membrane types include liquid applied acrylics, urethanes, neoprenes, vinyls, rubberized asphalts, silicones, and preformed membranes such as rubberized asphalts, neoprenes, and butyl rubbers, hypalons, vinyls, and ethylene propylene diene. Characteristics, advantages, and disadvantages of these materials are provided in Ref. 56.

Selection of sealer, membrane, or coating materials involves a number of factors (e.g., compatibility with new or old concrete, compatibility with joint sealant materials, crack-bridging ability, effective service life, weatherabiliy, etc.). Water absorption and vapor transmission properties for selected coating materials are presented in Table 3.10.[56] An indication of the relative performance of several coating systems is provided in Table 3.11.[233] Guidelines for selection of barrier systems for concrete are provided in Ref. 62. Surface preparation is extremely important in the use of any sealer or coating material (i.e., cleanliness and moisture condition). Adhesion of film forming coatings can be evaluated by ASTM D 3359,[246] direct tension (elcometer), or by direct peel. For membrane coatings, 25.4-mm wide strips can be cut in the membrane, clamped to a force gage, and pulled at 180° to the surface.[56]

Remedial Measures for Active Corrosion

In situations where application of preventative measures may not be possible, or the corrosion process has initiated, remedial measures are required. Basic remedial measures to strengthen or repair reinforced concrete structures damaged by corrosion include (1) taking no action, (2) replacement of damaged components, (3) stopping the corrosion process, and (4) reducing the corrosion rate.[232] An example where no action might be taken would be in situations where the structure may be nearing the end of its desired service life and an assessment indicates that it can continue to meet its functional and performance requirements. Some local

* Polyurethanes may degrade under ultraviolet exposure and the methacrylate monomer is highly volatile.

repairs and monitoring may be part of this strategy. When corrosion damage is localized on exposed surfaces, replacement or partial reconstruction may provide the most feasible solution. Often, however, some form of intervention is required to inhibit the corrosion process (i.e., reduce the corrosion rate to negligible values or repassivate corroding areas).

Basically three processes are necessary for corrosion occurrence — anodic, cathodic, and electrolytic. Repair activities are directed at halting one or more of these processes.* Basic principles for halting the anodic and electrolytic processes are presented in Fig. 3.7.[232] Brief descriptions of each of these basic principles are provided below.** More detailed information on each of these principles, including proper application, effectiveness, advantages and disadvantages, and any limitations is provided elsewhere.[87,232,233,247]

- Repassivation — Three basic methods for repassivation of steel reinforcement are available: (1) use of alkaline cement or mortar, (2) electrochemical realkalization, and (3) chloride extraction.

  (1) Use of alkaline cementitious materials involves the placement of a cement-based mortar or concrete in the form of a patch (local) or layer over the entire concrete surface (general) and relies on the migration of alkalis into the old concrete. If cracks or spalls are present, loose material should be removed and loose rust cleaned from steel. The depth of placement of new material should be greater than the estimated depth of carbonation during the remaining desired service life. This method is not effective if carbonation has penetrated greater than 20 mm below the depth of reinforcement or if steel depassivation has been caused by chlorides.[232]

  (2) Electrochemical realkalization restores a high pH to the concrete by generation of hydroxide ions at the steel and transport of alkaline material from the electrolyte (e.g., one molar sodium carbonate) into the concrete by capillary absorption, diffusion, and possibly by electro-osmosis. This method has been shown in the field to arrest corrosion, but its long-term effectiveness is questionable. Also, the method has been shown to be suitable for preventing corrosion in structures where cover concrete has been carbonated, but it is unknown if it can stop ongoing corrosion due to carbonation. Due to evolution of hydrogen during the process, this method is not recommended where prestressing steel is present or where steel-to-concrete bond is important. Also, the potential for alkali-aggregate reactivity is increased by the high concentration of sodium ions.[233]

  (3) Chloride extraction removes chloride ions from the concrete in order to achieve a residual chloride ion concentration low enough to stop the corrosion process. A direct current field is applied through the concrete by means of an external anode on the concrete surface. The chloride ions migrate to the surface where they are captured in the electrolyte and removed when the process is completed. This method has often been combined with electrochemical realkalization and has the same basic precautions.[232,233]

---

* Halting the cathodic process requires the total blockage of oxygen access to the steel reinforcement. Since this can not generally be accomplished, this process will not be addressed.

** Reduction of the moisture content will not be discussed as it is essentially the same as described in the previous section covering sealers, coatings, and membranes as preventative measures.

- Steel Reinforcement Coating — Some repair systems require or include the application of a physical barrier (e.g., epoxy) to protect the steel and provide electrical resistance. The procedure requires the removal of concrete to a depth below the steel, a clean steel surface free of rust, and sometimes, if the steel is badly corroded, replacement of portions of the steel reinforcement. If the remainder of the steel in the structure is uncoated, and the coating in the repaired area is damaged and becomes corrosively active, a high rate of corrosion can occur. Also, in chloride environments, the chlorides may penetrate the ends of the coating to cause crevice corrosion where the steel has not been repassivated. Use of low water-cement ratio cementitious coatings may be preferred in chloride-contaminated concrete.[232]

- Cathodic Protection — The corrosion process can be effectively halted, or its rate decreased, through application of a small direct current to the steel reinforcement to make it slightly cathodic relative to an externally applied anode at or near the concrete surface. Two systems of cathodic protection systems are available: (1) impressed current, and (2) sacrificial anode. Impressed current systems use a direct current power supply (rectifier) to force current flow from a relatively inert electrode (anode) through the concrete to the steel to be protected. In the sacrificial systems, a metal that is more anodic (higher tendency to corrode) than the embedded steel is used as the source of energy. Cathodic protection has been successfully used for several decades and is best suited for applications where the concrete is extensively contaminated and high chloride levels exist at the level of the steel. Removal of contaminated concrete is not required. Special precautions are required when the following are encountered: alkali-aggregate reactivity, lack of steel reinforcement continuity, highly electrically-resistant concrete, epoxy coated rebars, or galvanized steel. Also, generation of hydrogen at the cathode may embrittle prestressing wires or strands and, since cathodic protection systems are direct current, there is the potential for stray current corrosion in other structures.[87,232]

3.3.3 Commentary on Remedial Measures

Reviews of repair procedures for reinforced concrete structures have been conducted under the Structural Aging Program from both the European and North American perspectives.[56,233] Although a number of codes and standards have been developed for new construction, none are presently available that specifically address repair of degraded structures. Activities are presently underway, but this is expected to be a rather lengthy process. However, several documents are available in the form of guidelines or recommended practices.

In the U.S., ACI Committee Reports 201.2[223] and 546[242] discuss concrete repair. The ACI also has produced several documents used in educational seminars that are of use — *Troubleshooting Concrete Problems – And How to Prevent them in the Future*[248] and *Concrete Repair Basics*.[249] Both the U.S. Army Corps of Engineers and U.S. Bureau of Reclamation have produced concrete repair manuals.[239,241] The manual by the Corps of Engineers provides a standard format for repair techniques and includes chapters on evaluation of concrete structures, causes of distress and deterioration, selection of materials and methods for repair or rehabilitation, maintenance of concrete, specialized repairs, and case histories. Information on material applications and limitations is somewhat brief, however, and service life of repairs is not discussed. The Corps of Engineers has recently developed a notebook[250] in the form of a computer data base that provides material data sheets on specific products.

In Europe, the most widely developed regulations for repair of concrete have been prepared by the German Committee on Reinforced Concrete.[251] The German guidelines address four major areas: general regulations and basic design rules, design and performance, quality assurance and execution, and technical delivery conditions and test regulations. Outside Germany, the Austrians have revised their standards to define an "orderly" basis for the future repair of concrete structures and a basis for the evaluation of existing structures.[252] Guidelines or recommended practices have been produced by the *Concrete Society*,[253] *The Construction Industry Research and Information Association*,[247] *The United Kingdom Department of Transport*,[254,255] *Comite Euro-International du Beton*,[256] and *Fédération Internationale de la Précontrainte*.[200] Most of the European regulations address repair of corrosion-damaged concrete, indicating the size of the problem.

Although codes and standards have not been developed, sufficient documentation, such as noted above, is available to develop an effective repair strategy for safety-related NPP concrete structures. As structures in the general civil engineering community have aged and incidences of degradation have increased, there is increasing awareness of potential problems and research being conducted to address these problems can be transferred to NPP concrete structures. The basic mechanisms leading to concrete degradation are generally understood. The importance of recognizing the critical role played by the environment, at both the macro and micro level, is being recognized, as well as the importance of workmanship during installation. Repair strategies are becoming more global in that they are looking at the entire repair process as opposed to merely the selection and application of a repair material based on information provided by the vendors. Long-term data on the effectiveness or durability of various remedial measures is required. Knowledge of the durability of various repair materials is required to formulate the most effective repair.

Table 3.1. Commonly used evaluation procedures to assess concrete properties and physical condition.

a. Evaluation of properties of concrete.

CHEMICAL AND PHYSICAL PROPERTIES \ EVALUATION PROCEDURE	ACOUSTIC IMPACT	AIR CONTENT TEST	CEMENT CONTENT TEST	CHEMICAL TESTS	CORE TESTING	ELECTRICAL POTENTIAL MEASUREMENTS	ELECTRICAL RESISTANCE MEASUREMENTS	FLEXURAL TESTS	FREEZE-THAW TEST	GAMMA RADIOGRAPHY	NUCLEAR MOISTURE METER	PERMEABILITY TEST	PETROGRAPHIC ANALYSIS	PULLOUT TESTING	REBOUND HAMMER	ULTRASONIC PULSE	WINDSOR PROBE
ACIDITY				●									●				
AIR CONTENT		●											●				
ALKALI-CARBONATE REACTION													●				
ALKALI-SILICA REACTION													●				
CEMENT CONTENT			●	●									●				
CHEMICAL COMPOSITION				●									●				
CHLORIDE CONTENT				●	●								●				
COMPRESSIVE STRENGTH					●									●	●	●	●
CONTAMINATED AGGREGATE				●									●				
CONTAMINATED MIXING WATER				●									●				
CORROSION ENVIRONMENT				●		●											
CREEP					●												
DENSITY					●					●							
ELONGATION					●												
FROZEN COMPONENTS													●				
MODULUS OF ELASTICITY					●											●	
MODULUS OF RUPTURE					●			●									
MOISTURE CONTENT					●		●				●						
PERMEABILITY												●	●				
PULL OUT STRENGTH														●			
QUALITY OF AGGREGATE													●				
RESISTANCE TO FREEZING AND THAWING					●				●				●				
SOUNDNESS					●					●			●				
SPLITTING TENSILE STRENGTH					●												
SULFATE RESISTANCE				●									●				
TENSILE STRENGTH					●			●									
UNIFORMITY	●												●		●		●
WATER-CEMENT RATIO													●				

From ASCE 11-90, *ASCE Guideline for Structural Condition Assessment of Existing Buildings*, American Society of Civil Engineers, New York, New York, August 1, 1991; adapted with permission of ASCE, 1996.

Table 3.1. (Cont'd)

b. Evaluation of physical conditions of concrete.

PHYSICAL CONDITION \ EVALUATION PROCEDURE	ACOUSTIC EMISSIONS	ACOUSTIC IMPACT	CHEMICAL TESTS	CORE TESTING	FIBER OPTICS	GAMMA RADIOGRAPHY	INFRARED THERMOGRAPHY	LOAD TESTING	PETROGRAPHIC ANALYSIS	PHYSICAL MEASUREMENT	RADAR	REBOUND HAMMER	ULTRASONIC PULSE	ULTRASONIC PULSE-ECHO	VISUAL EXAMINATION	WINDSOR PROBE
BLEEDING CHANNELS									●						●	
CHEMICAL DETERIORATION			●						●						●	
CORROSION OF STEEL			●	●					●						●	
CRACKING	●	●		●	●		●		●	●	●		●	●	●	
CROSS-SECT. PROPERTIES AND THICKNESS				●		●				●			●			
DELAMINATION		●		●	●	●	●		●		●		●	●	●	
DISCOLORATION			●						●						●	
DISINTEGRATION				●		●	●		●				●		●	
DISTORATION															●	
EFFLORESCENCE			●						●						●	
EROSION									●						●	
FREEZE-THAW DAMAGE									●						●	
HONEYCOMB				●	●	●	●		●				●		●	
POPOUTS															●	
SCALING															●	
SPALLING				●		●	●								●	
STRATIFICATION		●			●									●	●	
STRUCTURAL PERFORMANCE	●							●							●	
UNIFORMITY OF CONCRETE						●			●			●	●		●	●

From ASCE 11-90, *ASCE Guideline for Structural Condition Assessment of Existing Buildings*, American Society of Civil Engineers, New York, New York, August 1, 1991; adapted with permission of ASCE, 1996.

Table 3.2. Description of test methods for concrete.

Method	Applications	Principle of Operation	User Expertise	Advantages	Limitations	Standards
Acoustic Emission	Continuous Monitoring of structure during service life to detect impending failure; monitoring performance of structure during proof testing.	During crack growth or plastic deformation, rapid release of strain energy produces acoustic (sound) waves that can be detected by sensors attached to surface of test object.	Extensive knowledge required to plan test and to interpret results.	Monitors structural response to applied load; capable of detecting onset of failure; capable of locating source of possible failure; equipment is portable and easy to operate.	Expensive test to run; can be used only when structure is loaded and when flaws are growing; interpretation of results requires an expert; currently largely confined to laboratory; further work required.	
Acoustic Impact	Used to detect debonds, delaminations, voids, and hairline cracks.	Surface of object is struck, with the frequency, through transmission time, and damping characteristics of resulting sound giving indication of presence of defects; equipment may vary from simple hammer or drag chain to sophisticated trailer-mounted electronic equipment.	Low level of expertise required to use, but experience needed for interpreting results.	Portable equipment; easy to perform; electronic device not needed for qualitative results.	Geometry and mass of test object influence results; poor discrimination; reference standards required for electronic testing.	
Core Testing	Direct determination of concrete strength; concrete evaluation of condition of aggregate, cement, and other components.	Drilled cylindrical core is removed from structure; tests may be performed on core to determine compressive and tensile strength, torsional properties, static modulus of elasticity, etc.	Obtaining drilled core is routine; moderate level of expertise required to test and evaluate results.	Most widely accepted method to reliably determine strength and quality of in place concrete.	Process of drilling and analyzing cores is expensive; coring damages structures and many may be required; analysis of cores is time-consuming.	ASTM C 42
Cover Meters/ Pachometers	Cover meters measure depth of reinforcement cover in concrete; pachometers measure cover and size of reinforcement, and locations of delaminations.	Presence of steel in concrete affects magnetic field of probe; closer probe is to steel, the greater the effect.	Moderate; easy to operate; training needed to interpret results.	Portable equipment, good results if concrete is lightly reinforced.	Difficult to interpret results if concrete is heavily reinforced or if wire mesh is present.	

From ASCE 11-90, *ASCE Guideline for Structural Condition Assessment of Existing Buildings*, American Society of Civil Engineers, New York, New York, August 1, 1991; adapted with permission of ASCE, 1996.

Table 3.2. (cont'd)

Method	Applications	Principle of Operation	User Expertise	Advantages	Limitations	Standards
Electrical Potential Measurements	Determining condition of steel rebars in concrete or masonry.	Electrical potential of steel reinforcement indicates probability of corrosion.	Moderate. User must be able to recognize problems.	Portable equipment; field measurements readily made; appears to give reliable information.	Information on rate of corrosion not provided; access to rebars required.	ASTM C 876
Electrical Resistance Measurements	Determination of moisture content of concrete.	Determination of moisture content of concrete is based on principle that the dielectric properties of concrete change with changes in moisture content.	High level of expertise required to interpret results; equipment is easy to use.	Equipment is automated and easy to use.	Equipment very expensive and requires high frequency specialized applications, dielectric properties also depend on salt content and temperature of specimen that poses problems in interpretation of results.	
Fiber Optics	To view portions of a structure that are inaccessible to the eye.	Fiber optics probe consisting of flexible optical fibers, lens, and illuminating system is inserted into crack or drilled hole in concrete; eyepiece is used to view interior to look for flaws such as cracks, voids, or aggregate debonds; commonly used to look into areas where cores have been removed or bore holes have been drilled.	Equipment is easy to handle and operate.	Gives clear high-resolution images of remote objects.	Equipment expensive; limited application to concrete members; many bore holes required to give adequate access.	
Infrared Test	Detection of internal flaws, crack growth, delamination, and internal voids; currently used primarily in laboratory.	Flaws detected by using selective infrared frequencies to detect various passive heat patterns that can be identified as belonging to certain defects.	High level of expertise required to interpret results.	Has potential for becoming a relatively inexpensive and accurate method for detecting concrete defects.	Method still in developmental phase, requires dry surface and bright sunshine that limits application.	

From ASCE 11-90, *ASCE Guideline for Structural Condition Assessment of Existing Buildings*, American Society of Civil Engineers, New York, New York, August 1, 1991; adapted with permission of ASCE, 1996.

Table 3.2. (cont'd)

Method	Applications	Principle of Operation	User Expertise	Advantages	Limitations	Standards
Load Testing	Determine performance of structure under simulation of actual loading conditions.	Test load is applied to structure in a manner that will simulate the load pattern under design conditions.	High level of expertise required to formulate the test programs and to evaluate the results.	Provides highly reliable prediction of structure's ability to perform satisfactorily under expected loading conditions.	Very expensive and time-consuming; testing may cause limited or even permanent damage to the structure or some of its elements.	ACI 437R
Nuclear Moisture Meter	Estimation of moisture content of hardened concrete.	Moisture content in concrete determined based on the principle that materials (such as water) decrease the speed of fast neutrons in accordance with the amount of hydrogen produced in test specimen.	Must be operated by trained and licensed personnel.	Portable moisture estimates can be made of in-place concrete.	Equipment very sophisticated and expensive; NRC license required to operate; moisture gradients in specimen may give erroneous results.	
Petrographic Analysis	Used to determine a variety of properties of concrete cores removed from structure; some of these include (1) denseness of cement, (2) homogeneity of concrete, (3) location of cracks, (4) air content, and (5) proportions of aggregate, cement, and air voids.	Used in conjunction with coring; chemical and physical analysis of core is performed by qualified petrographer.	Considerable skill required to perform and analyze test results.	Provides very detailed and reliable information.	Qualified petrographer required; relatively expensive.	ASTM C 856
Pullout Testing Cast in-place	Estimation of compressive and tensile strengths of concrete.	Measure the force required to pull out the steel rod with enlarged head cast in concrete, pullout forces produce tensile and shear stresses in concrete.	Low level of expertise required; an be used by field concrete testers and inspectors.	Only NDE method that directly measures in-place strength of concrete; appears to give good prediction of concrete strength.	Pullout devices must be inserted during construction; cone of concrete may be pulled out, necessitating minor repairs.	ASTM C 900 ACI 503R

From ASCE 11-90, *ASCE Guideline for Structural Condition Assessment of Existing Buildings*, American Society of Civil Engineers, New York, New York, August 1, 1991; adapted with permission of ASCE, 1996.

Table 3.2. (cont'd)

Method	Applications	Principle of Operation	User Expertise	Advantages	Limitations	Standards
Pull-off Testing	Estimation of compressive strength of existing concrete.	Circular steel probe is bonded to concrete. Tensile force is applied using portable mechanical system until concrete fails. Compressive strength can be estimated using calibration charts.	Highly skilled operator is not required.	Simple and inexpensive; can be conducted on horizontal and vertical surfaces.	Standard test procedure not yet available. Concrete must be repaired at test locations.	
Radar	Detection of substratum voids.	Uses transmitted electromagnetic impulse signals for void detection.	High level of expertise required to operate equipment and interpret results.	More time- and cost-efficient and less destructive than "guess and drill" techniques.	Equipment is expensive; reliability of void detection greatly reduced if reinforcement present; procedure still under development.	
Radiographics, X-ray and Gamma-ray	X-ray — density and internal structure of concrete; location of reinforcement. Gamma-ray — location, size, and condition of rebars; voids in concrete; density and thickness.	Based on principle that the rate of absorption of X-rays or gamma-rays is affected by density and thickness of test specimen; X-rays or gamma-rays are emitted from source, penetrate the specimen, exit on opposite side, and are recorded on film.	Use of gamma-producing isotopes is closely controlled by NRC; gamma equipment must be operated by licensed inspectors.	Internal defects can be detected; applicable to variety of materials; permanent record on film; gamma-ray equipment easily portable.	X-ray has limited field application because equipment is heavy and costly; X-ray and gamma-ray sources harmful to organic tissue; requires access to both sides of specimen.	
Schmidt Rebound Hammer	Compares quality of concrete from different areas of specimen; estimates of concrete strength based on calibration curves with limited accuracy.	Spring-driven mass strikes surface of concrete and rebound distance is given in R values; surface hardness is measured and strength estimated from calibration curves provided by hammer manufacturer.	Simple to operate; can be readily operated by field personnel.	Equipment is light-weight, simple to operate, and inexpensive; large amount of data can be quickly obtained; good for determining uniformity of concrete and areas of potentially low strength.	Results affected by condition of concrete surface; does not give precise prediction of strength; estimates of strength should be used with great care; frequent calibration of equipment required.	ASTM C 805

From ASCE 11-90, *ASCE Guideline for Structural Condition Assessment of Existing Buildings*, American Society of Civil Engineers, New York, New York, August 1, 1991; adapted with permission of ASCE, 1996.

Table 3.2. (cont'd)

Method	Applications	Principle of Operation	User Expertise	Advantages	Limitations	Standards
Ultrasonic Pulse Velocity	Gives estimates of compressive strength, uniformity, and quality of concrete; internal discontinuities can be located and their size estimated; most widely used vibrational method for field use.	Operates on principal that vibrational wave propagation is affected by quality of concrete; pulse waves are induced in materials and those reflected back are detected.	High level of expertise required to interpret results.	Equipment relatively inexpensive and easy to operate; accurate assessment of strength and quality; estimates of concrete strength to ± 20%.	Good coupling between transducer and test substrate critical; interpretation of results can be difficult; density and amount of aggregate may affect results; calibration standards required.	ASTM C 597
Visual Examination	Evaluation of the surface condition of concrete (finish, roughness, scratches, cracks, color); determining deficiencies in joints; and determining differential movements of structures.	Visual examination with or without optical aids, measurement tools, photographic records, or other low-cost tools, differential movement determined over long periods of time with surveying methods.	Experience required in order to determine what to look for, what measurements to take, and what follow-up testing to specify.	Generally low-cost; rapid results except for surveying method; evaluation of both surface and interior of concrete member possible.	Trained evaluation required; primary evaluation confined to surface of structure.	ACI 201.1R ASTM C 823
Windsor Probe	Estimates of compressive strength, uniformity, and quality of concrete.	Probes are gun driven into concrete specimen; depth of penetration converted to estimates of concrete strength by using calibration curves provided by manufacturer.	Simple to operate; can be readily operated in the field with little training.	Equipment is simple durable, and requires little maintenance, useful in assessing the quality and relative strength of concrete; does relatively little damage to specimen.	May not yield accurate estimates of concrete strength; interpretation of results depends on correlation curves.	ASTM C 803

From ASCE 11-90, *ASCE Guideline for Structural Condition Assessment of Existing Buildings*, American Society of Civil Engineers, New York, New York, August 1, 1991; adapted with permission of ASCE, 1996.

Table 3.3. Recommended testing methods to assess concrete degradation.

DEGRADATION FACTOR	SYMPTOM	TESTING METHODS	
		TO IDENTIFY OCCURRENCE	TO ASSESS EXTENT OF DAMAGE*
Alkali-aggregate reactivity (concrete)	Cracking Expansion	1. Core/petrography 2. SHRP Rapid Test	1. Visual and petrography 2. Pulse velocity 3. Impact echo 4. Pulse echo 5. Modal analysis
Sulfate attack (concrete)	Cracking Expansion	1. Core/petrography 2. Core/chemical	1. Visual and petrography 2. Pulse velocity 3. Impact echo 4. Pulse echo 5. Modal analysis
Efflorescence and leaching (concrete)	Surface deposits of efflorescence	1. Visual 2. Core/petrography 3. Sample/X-ray diffraction	1. Visual and petrography
Bases/acids/salt crystalization (concrete)	Disintegration and loss of paste	1. Core/petrography 2. Chemical analysis	1. Visual and petrography
Moisture changes (concrete)	Cracking	1. Visual 2. Core/petrography	1. Visual and petrography 2. Infrared thermography 3. Pulse velocity 4. Impact echo 5. Pulse echo 6. Modal analysis
Freeze/thaw (concrete)	Scaling Spalling Cracking	1. Visual 2. Core/petrography	1. Visual and petrography 2. Pulse velocity 3. Impact echo 4. Pulse echo 5. Modal analysis
Thermal exposure/ cycling (concrete)	Spalling Cracking Loss of strength	1. Visual 2. Core/petrography	1. Visual and petrography 2. Pulse velocity 3. Impact echo 4. Pulse echo 5. Modal analysis
Irradiation (concrete)	Spalling Cracking Loss of strength	1. Visual 2. Core/petrography	1. Visual and petrography 2. Pulse velocity 3. Impact echo 4. Pulse echo

* Methods are rated in order of choice based on simplicity of the method and practical experience in applying the method.

Table 3.3. (cont'd)

DEGRADATION FACTOR	SYMPTON	TESTING METHODS	
		TO IDENTIFY OCCURRENCE	TO ASSESS EXTENT OF DAMAGE*
Abrasion/erosion/ cavitation (concrete)	Surface wear	1. Visual	1. Visual and petrography
Fatigue/vibration (concrete)	Microcracking Cracking Excessive deflection	1. Visual 2. Core/petrography	1. Visual and petrography 2. Pulse velocity 3. Impact echo 4. Pulse echo 5. Modal analysis
Creep (conventionally reinforced concrete)	Cracking Excessive deflection	1. Visual	1. Visual and petrography 2. Modal analysis
Corrosion (conventionally reinforced concrete)	Corrosion	1. Visual 2. Core/visual 3. Electrical method 4. Chemical method 5. Air permeability 6. Nuclear	1. Visual and petrography 2. Impact echo 3. Pulse echo 4. Radiography
	Cracking Delaminaion	1. Visual 2. Core/petrography 3. Audio method 4. Impact echo 5. Pulse echo	1. Visual and petrography 2. Infrared thermography 3. Audio method 4. Pulse velocity 5. Impact echo 6. Pulse echo
Corosion/temperature /irradiation (prestressing system)	Corrosion	1. Visual	1. Visual 2. Mechanical testing 3. Chemical analysis
	Loss of force	Lift-off test	Lift-off test

* Methods are rated in order of choice based on simplicity of the method and practical experience in applying the method.

Table 3.4. Causes of cracking.

Cause	Type of Crack		Comment
	Active	Dormant	
Accidental loading		X	
Design error (inadequate reinforcement)	X		Limit loading according to current capacity and repair, or redesign and repair as indicated by the redesign.
Temperature stresses (excessive expansion due to elevated temperature and inadequate expansion joints)	X		It may be desirable to redesign to include adequate expansion joints.
Corrosion of reinforcing steel	X		Simple crack repair methods should not be used as the steel will continue to corrode and crack the concrete.
Foundation settlement	X	X	Measurements must be made to determine if the foundation is still settling.
Alkali-aggregate reaction	X		Concrete will continue to deteriorate as long as moisture is present. Crack repair methods will be ineffective.
Poor construction procedures (inadequate curing, formwork, etc.)		X	
Design faults • use of exposed rigidly connected material to concrete that has a much different modulus of expansion • stress concentrations • faulty joint systems	X		
NOTE: This listing is intended to serve as a general guide only. It should be recognized that there will be exceptions to all of the items listed.			

Source: Corps of Engineers, Washington, D.C.

Table 3.5. Permissible crack widths to prevent corrosion of steel reinforcement.

Author	Environment factors	Permissible width, mm
Rengers	Dangerous crack width	1.0 to 2.0
	Crack width allowing corrosion within 1/2 year saline environment	0.3
Abeled	Structures not exposed to chemical influences	0.3 to 0.4
Tremper	Found no direct relation between crack width and corrosion	
Boscard	Structures exposed to a marine environment	0.4
de Bruyn	Found no direct relation between crack width and corrosion	
Engel and Leeuwen	Unprotected structures (external)	0.2
	Protected structures (internal)	0.3
Voellmy	Safe crack width	up to 0.2
	Crack allowing slight corrosion	0.2 to 0.5
	Dangerous crack width	over 0.5
Bertero	Indoor structures	0.25 to 0.35
	Normal outdoor exposure	0.15 to 0.25
	Exposure to seawater	0.025 to 0.15
Haas	Protected structures (interior)	0.3
	Exposed structures (exterior)	0.2
Brice	Fairly harmless crack width	0.1
	Harmful crack width	0.2
	Very harmful crack width	0.3
Salinger	For all structures under normal conditions	0.2
	Structures exposed to humidity or to harmful chemical influences	0.1
Wastlund	Structures subjected to dead load plus half the live load for which they are designed	0.4
	Structures subject to dead load only	0.3
Efsen	Exterior (outdoor) structures exposed to attack by seawater and fumes	0.05 to 0.25
	Exterior (outdoor) structures under normal conditions	0.15 to 0.25
	Interior (indoor) structures	0.25 to 0.35
Rüsch	Ordinary structures	0.3
	Structures subjected to the action of fumes and sea environment	0.2

Table 3.6. Properties of an epoxy and polyester chemical grout.

Property	Epoxy resin system	Polyester resin system*
Tensile strength, MPa	35.2	55.8
Tensile modulus, MPa	1.17×10^3	—
Elongation, %	15.8	2.6
Flexural strength, MPa	60.0	84.8
Flexural modulus, MPa	1.45×10^3	4.2×10^3
Deflection, mm	13.5	—
Compressive strength, MPa	52	—
Compressive modulus, MPa	1.3×10^3	—
Deflection at yield, mm	4.6	—
Izod impact strength, J/m	55	—
Hardness at 25°C (Shore D)	79	38–40**
Water absorption, %	0.21	0.15
Shrinkage, % (volume)	0.001***	6.0

* Typical values
** BARCOL
*** Effective (after gel information)

Source: *Chemical Grouting*, Engineer Manual EM 1110-2-3504, U.S. Army Corps of Engineers, Office of the Chief of Engineers, Washington, D.C., May 1973.

Table 3.7. General guide to repair options for concrete cracking.

Description	Repair Options	Perceived durability rating (1–5*)	Commentary
Dormant pattern or fine cracking	Judicious neglect	4	Only for fine cracks
	Autogenous healing	3	Only on new concrete
	Penetrating sealers	2	Use penetrating sealer for H_20, Cl resistance
	Coatings	3	Use coating for abrasion & chemical resistance
	HMWM or epoxy treatment	2	Topical application, bonds cracks
	Overlay or membrane	2	For severely cracked areas
Dormant isolated large cracking	Epoxy injection	1	Needs experienced applicator
	Rout and seal	3	Requires maintenance
	Flexible sealing	4	Requires maintenance
	Drilling and Plugging	3	
	Grout injection or dry packing	4	
	Stitching		
	Additional reinforcing	5	
	Strengthening	4	
		3	
Active cracks	Penetrating sealer	3	Cracks less than 0.5 mm
	Flexible sealing	3	Requires maintenance
	Route and seal	3	Use for wide cracks
	Install expansion joint	2	Expensive
	Drilling and plugging	4	May cause new cracks
	Stitching	4	May cause new cracks
	Additional reinforcing	3	May cause new cracks
Seepage	Eliminate moisture source	1	Usually not possible
	Chemical grouting	2	Several applications may be necessary
	Coatings	4	May have continued seepage
	Hydraulic cement dry packing	4	May have continued seepage

*Scale from 1 to 5, with 1 being most durable.

Table 3.8. Typical properties of rapid set patching materials by generic family.[56]

MATERIAL	Approx. working time (min.) @ 22°C	Approx. time to traffic (min.) @ 22°C	Compressive strength (MPa) @ 3 hr	Compressive strength (MPa) @ 24 hr	Abrasion loss (g) @ 24 hr	Flexural strength (MPa) @ 24 hr	Bond strength (MPa) @ 24 hr Dry PCC	Bond strength (MPa) @ 24 hr Wet PCC	E (10^3 MPa)	α (10^{-6} per C)	Linear shrinkage (%)
Inorganic											
PCC w/accelerator	120+	300+	—	20	22	3.0	2.0	2.5	15–40	7–20	0.02–0.08
Magnesium phosphate	15	60	28	42	25	5.6	3.3	1.1	25–35	11–14	0.10–0.15*
High alumina cement	15	60	35	46	20	4.2	2.8	2.6	25–35	7–20	0.02–0.08
Gypsum based	20	60	25	42	18	2.8	2.1	2.6	15–20	7–20	0.03–0.05
Organic											
Epoxy	30–60	90–200	15	55–80	0–1	16–21	Failed in PCC	Failed in PCC	0.7–40	27–54	0.02–0.2
Methacrylate	20–40	60–120	50	55–65	10	14–21	Failed in PCC	Failed in PCC	7–25	13–23	1.5–5.0
Polyester-styrene	15–40	60–120	15	20–35	3	10–14	Failed in PCC	Failed in PCC	7–35	32–54	0.3–3.0
Urethane	5–45	30–90	3 - 15	3–35	3	10–27	Failed in PCC	3.4	0.7–40	54–126	0.02–0.2

E = Modulus of elasticity in compression
α = Thermal coefficient of expansion
*High exotherm

Source: H. Jerzak, "Unpublished Test Data," Transportation Laboratory, California Department of Transportation, Sacramento, California, 1988.

Table 3.9. General guide to repair options for concrete spalling.

Description	Repair Options	Perceived durability rating (1–5*)	Commentary
Shallow Spalling	Portland cement grouts	3	Not good for acid attack
	Polymer-modified grout	2	Different thermal coefficient
	Coatings	4	Limited to shallow areas
	Membranes	3	Acids–epoxy, methacrylate, butyl, neoprene
	Polymer grouts	2	Acids–use polyester grout
Deep Spalling	Portland cement concrete	2	Inexpensive
	Expansive cements	3	Unreliable expansion
	Gypsum-based concrete	5	Do not use in moist environments
	High alumina (modified)	3	Bonds best to dry concrete
	Magnesium phosphate	2	Base concrete must be dry
	Polymer-modified	2	Thermal stress can be high
	Polymer patching materials	3	Less than 40 mm thickness
	Polymer overlays	2	25 to 50 mm thickness
	Latex-modified concrete overlays	2	Greater than 30 mm thickness
	Portland cement concrete overlays	3	Use low water/cement ratio and high-range water reducer
	Silica fume overlays	3	High strength
	Pre-placed aggregate	2	Low shrinkage
	Shortcrete	3	Good for large areas

*Scale from 1 to 5, with 1 being most durable.

Table 3.10. Water absorption and vapor transmission of selected coating materials.

		7-day water absorption, %			7-day water vapor transmission, %			
Generic type	No.	Average	Mean	Range	Average	Mean	Range	Application rate range sq ft/gal
Control	1	4.79	—	—	3.53	—	—	
Acrylics	9	0.82	1.84	0.50–3.08	1.56	1.47	0.83–2.81	75–110
Cementitious	5	2.75	2.94	0.49–4.56	1.80	2.18	1.02–2.28	—
Epoxy	7	0.31	0.16	0.06–0.37	0.20	0.21	0.06–0.46	60–125
Hypalon	2	0.36	—	0.29–0.42	0.23	—	0.18–0.37	75–95
Neoprene	1	1.06	—	—	0.37	—	—	70
Polyurethane	16	0.55	0.20	0.04–3.79	0.60	0.54	0.15–1.54	70–175
Polyester	1	0.12	—	—	0.27	—	—	90
Silicone	1	1.76	—	—	2.76	—	—	80

Table 3.11. Relative performance of several coating systems for concrete materials.

Coating	Application*				Protection*			
	Damp Conditions	Alkali Resistance	Ease to Apply	Low Hazard	Resist $C\ell^-$	Resist CO_2	Vapor Transmission	Resist Rain
Film Forming								
Epoxy resin	4	1	5	5	1	1	5	1
Coal tar epoxy	4	1	5	5	1	1	5	1
Polyurethane	4	1	5	5	1	1	5	1
Chlorinated rubber	2	1	1	5	1	1	5	1
Bituminous	2	3	1	3	3	3	3	1
Acrylic resin	2	3	1	1	3	1	1	1
Densifying								
Silicate/flurosilicate	1	1	1	1	5	5	1	3
Cementitious	1	1	1	1	3	3	1	3
Non-film Forming								
Silane/siloxane	2	2	1	5	1	5	1	1
Oil impregnation	4	5	1	1	3	5	1	4
Silicone	4	5	1	2	3	5	1	3

*1 = very good, 5 = very poor.

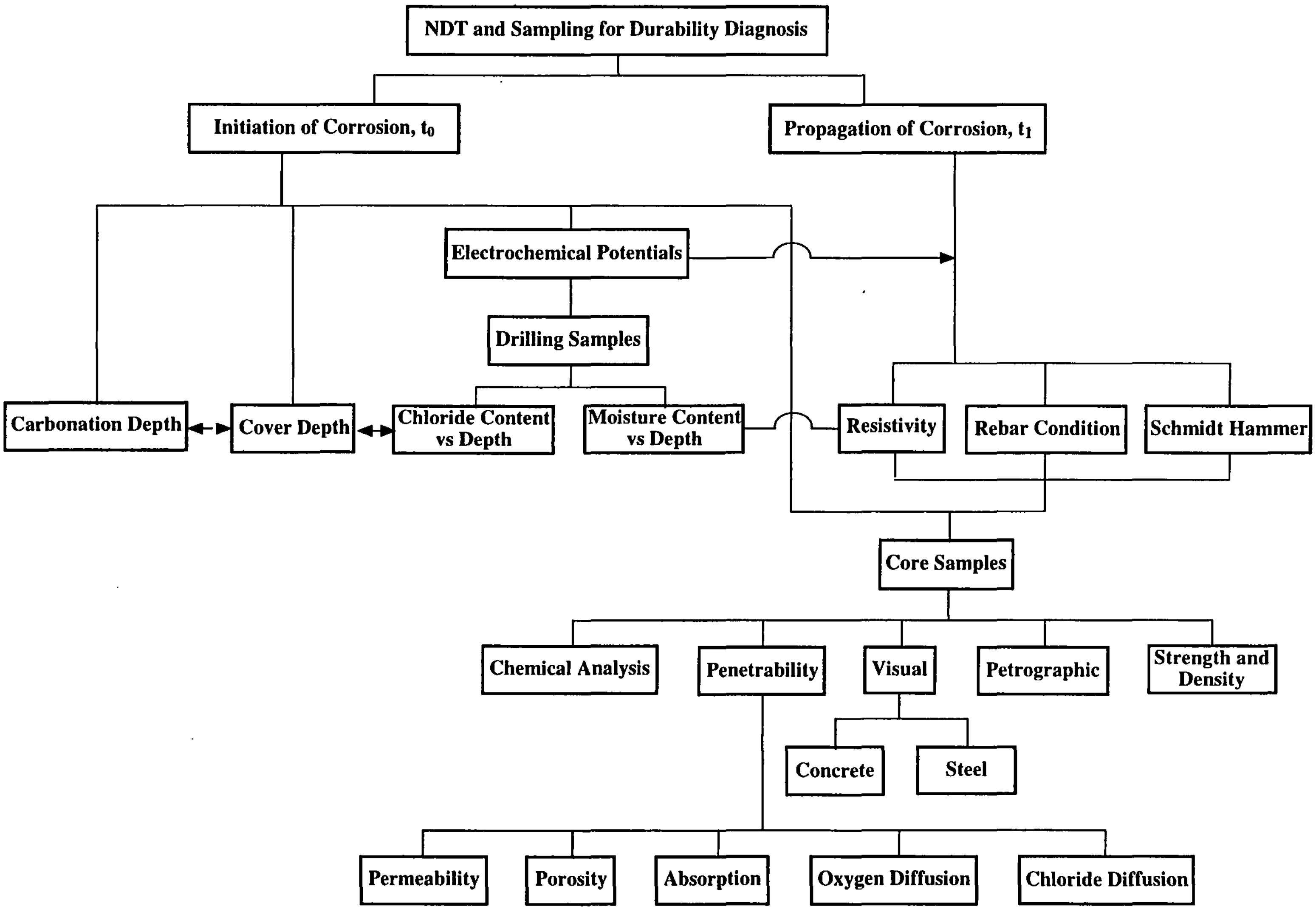


Fig. 3.1 Techniques for assessing deterioration. Source: R. D. Browne, "Durability of Reinforced Concrete Structures," *Pacific Concrete Conference*, Proceedings Vol. 3, Auckland, New Zealand, New Zealand Concrete Society, November 1988; reprinted with permission from author.

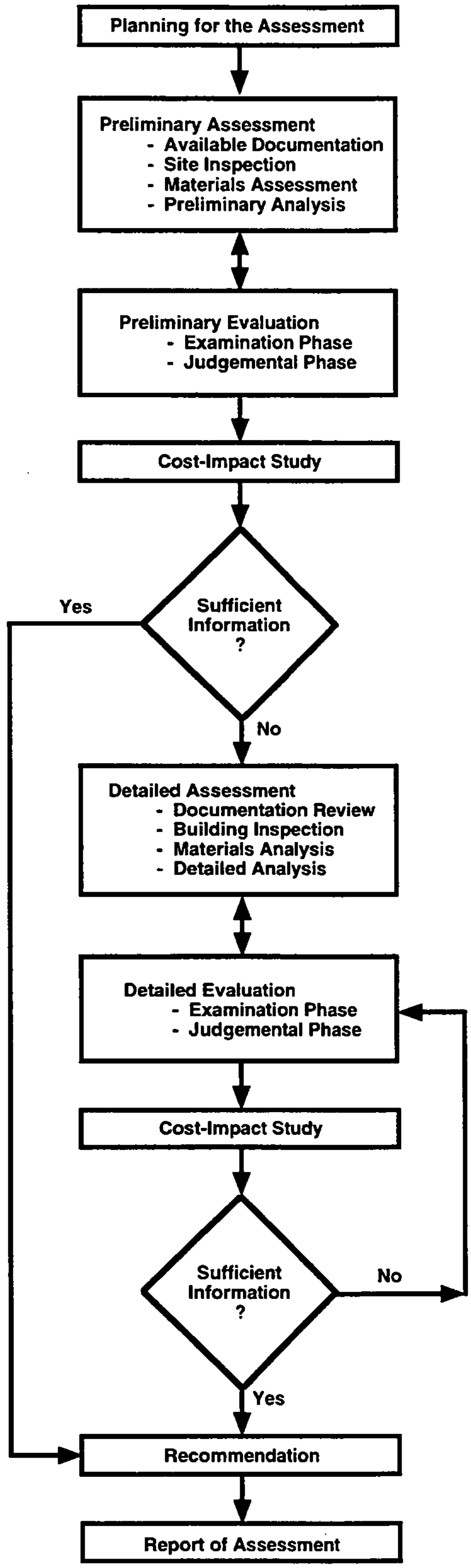


Fig. 3.2 General structural assessment and evaluation procedure for existing buildings. From ASCE 11-90, *ASCE Guideline for Structural Condition Assessment of Existing Buildings*, American Society of Civil Engineers, New York, New York, August 1, 1991; adapted with permission from ASCE, 1996.

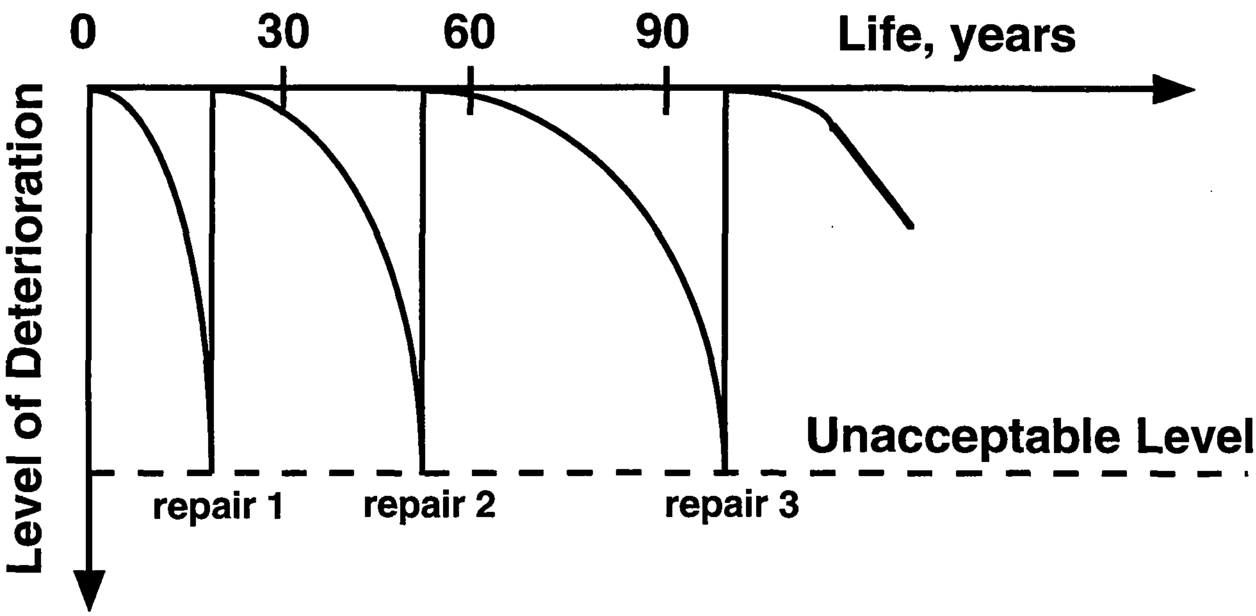


Fig. 3.3 Effect of multiple repairs over the life of a structure.

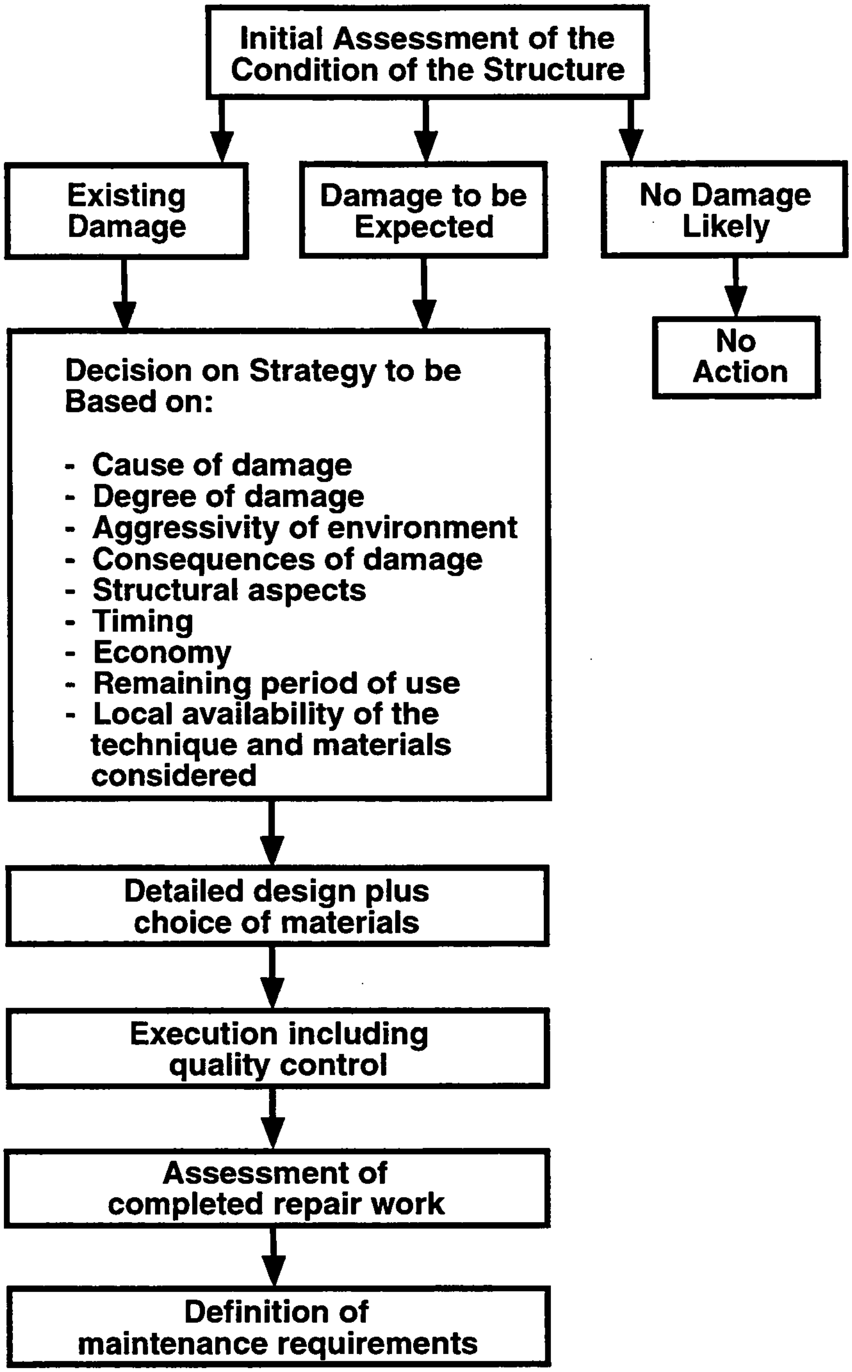


Fig. 3.4 Steps to be taken in a repair process. Source: Technical Committee 124- SRC, "Draft Recommendation for Repair Strategies for Concrete Structures Damaged by Reinforcement Corrosion," pp. 415-436 in *Materials and Structures* 27(171), International Union of Testing and Research Laboratories for Materials and Structures (RILEM), Cachan, France; adapted with permission from RILEM.

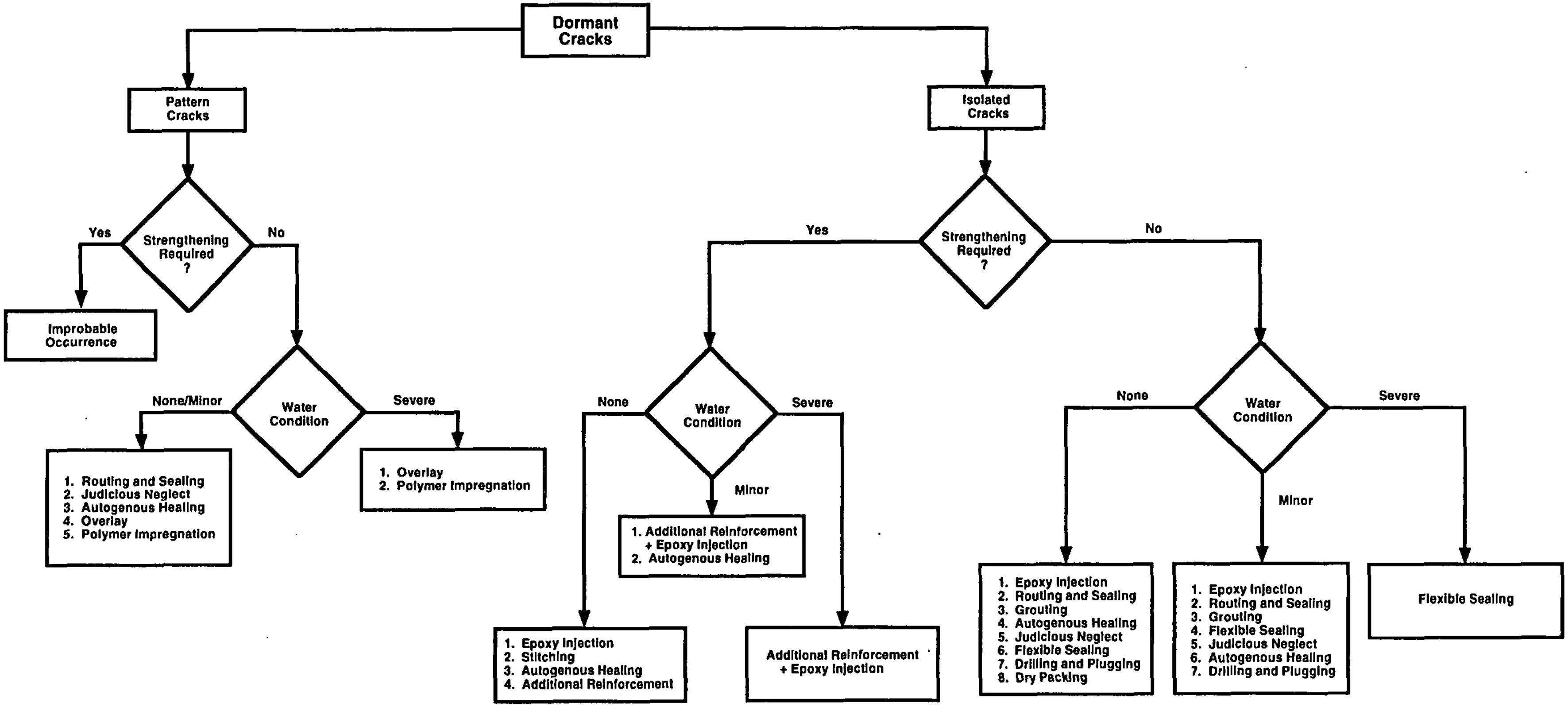


Fig. 3.5 Selection of repair technique for dormant cracks. Source: *Evaluation and Repair of Concrete Structures*, EM 1110-2-2003, U.S. Army Corps of Engineers, Washington, D.C., 1986.

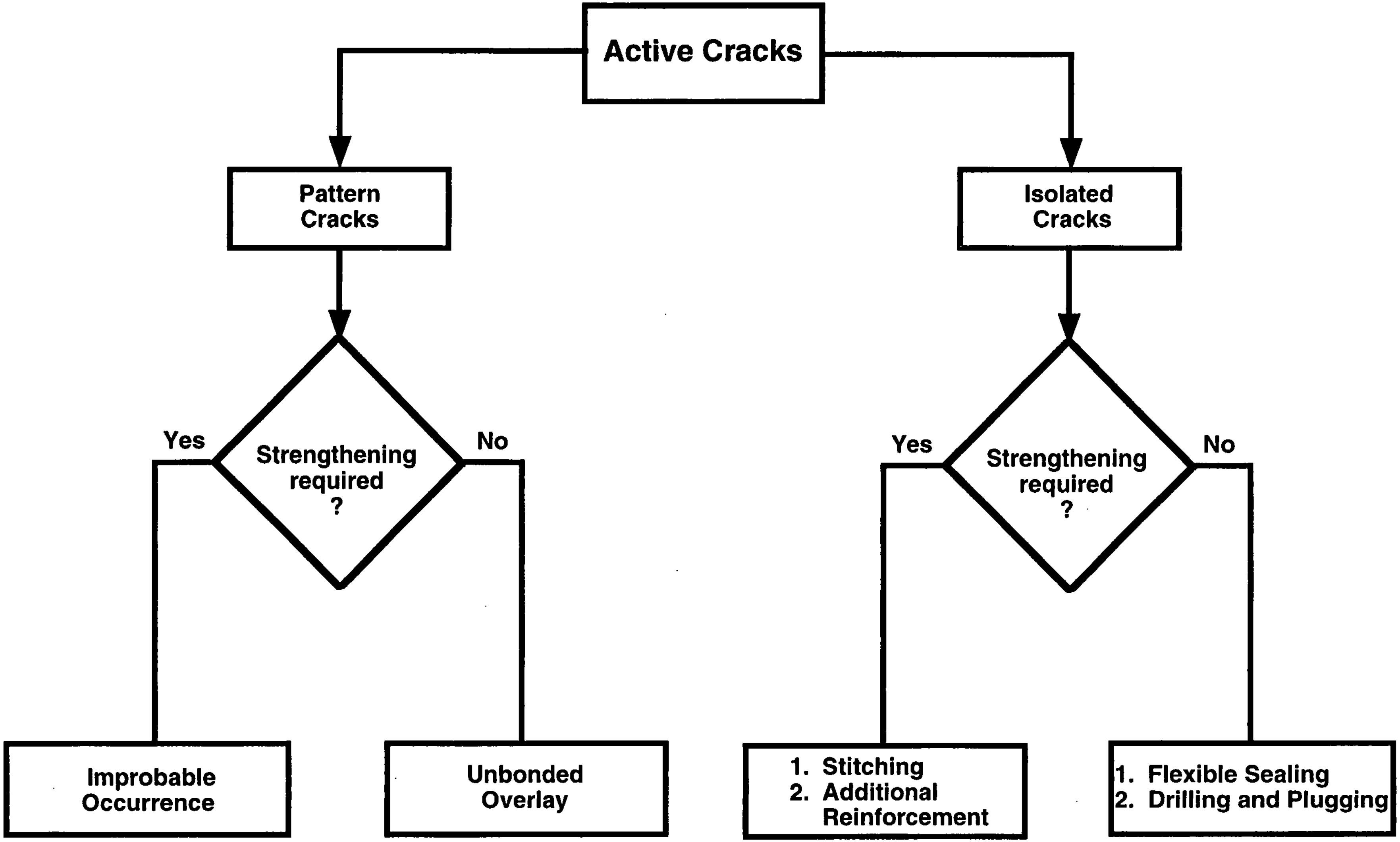


Fig. 3.6 Selection of repair technique for active cracks. Source: *Evaluation and Repair of Concrete Structures*, EM 1110-2-2003, U.S. Army Corps of Engineers, Washington, D.C., 1986.

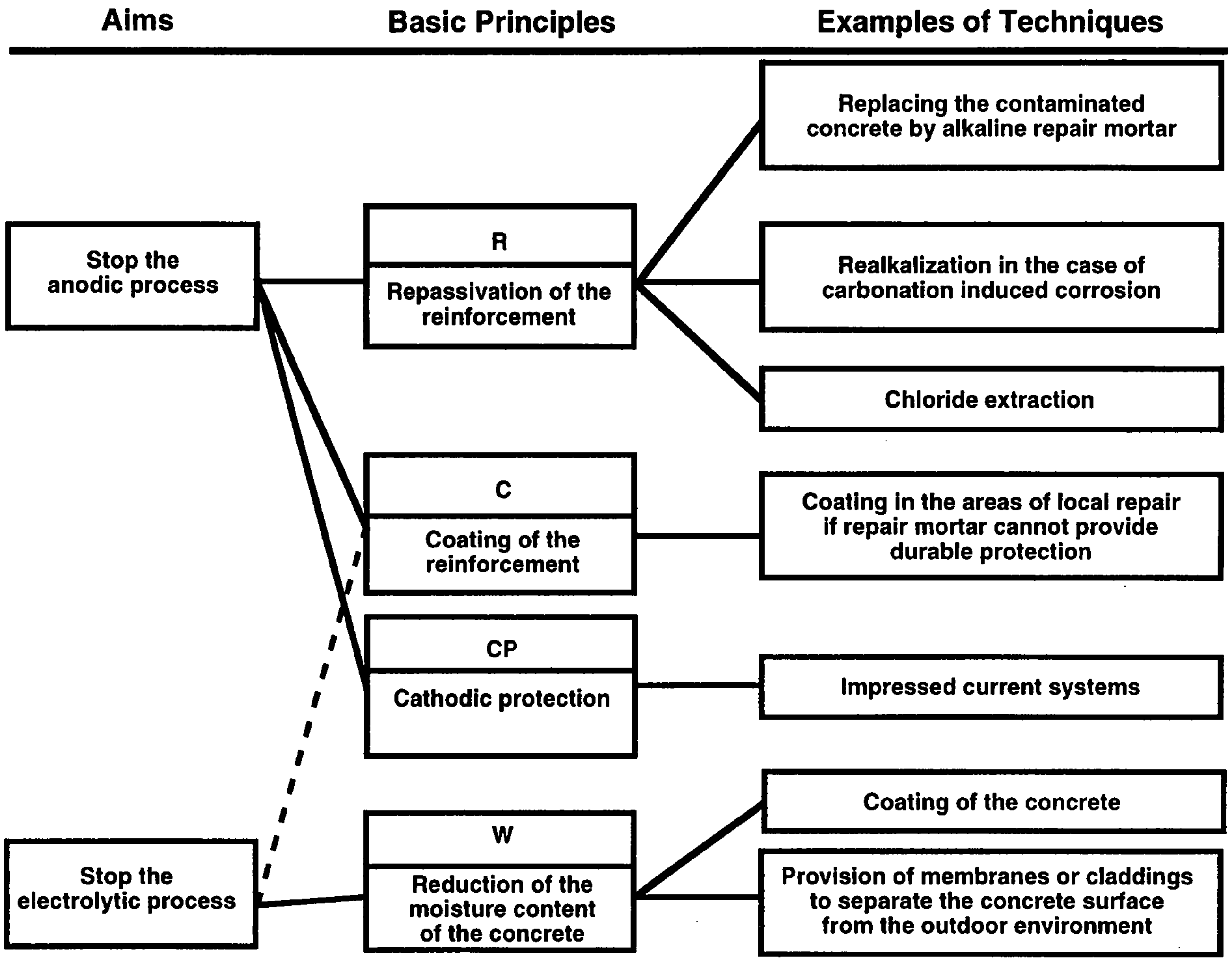


Fig. 3.7 Principles of repair to stop corrosion.

Source: Technical Committee 124- SRC, "Draft Recommendation for Repair Strategies for Concrete Structures Damaged by Reinforcement Corrosion," pp. 415-436 in *Materials and Structures* 27(171), International Union of Testing and Research Laboratories for Materials and Structures (RILEM), Cachan, France; adapted with permission from RILEM.

4. EVALUATION OF NPP REINFORCED CONCRETE STRUCTURES

The importance of reinforced concrete structures to the overall safety of nuclear power plants (NPPs) has been established. In contrast to mechanical and electrical components, civil structures are intended to have service lives on the order of 50 to 100 years or more, and current codes of practice (e.g., Refs. 16 and 20) do not explicitly address aging. Structural systems in a NPP are designed to be safe, serviceable, and durable. However, most structural systems are designed for safety directly, but serviceability and durability issues are dealt with only indirectly, if at all. The inherently conservative nature of design is intended to ensure the structure's performance in service. Structural deterioration is not considered explicitly, and there is no presumption in code development that periodic inspection, maintenance, and repair are carried out. Current periodic inspections only involve assessments of the post-tensioning systems and a general visual inspection of accessible concrete surfaces in conjunction with a leak-rate test. Some utilities, however, do much more.

Properly designed and maintained structures normally perform well over extended periods of time. However, these structures are subject to a phenomenon known as aging, which refers to time-dependent changes that may impact the ability of these structures to withstand various demands from operation, the environment, and accident conditions. The time-dependent behavior of these changes should be considered in the overall condition assessment of a structural system. Failures can occur when excessive degradation takes place, frequently due to design or construction errors or an unanticipated aggressive service environment. Such failures often are related to serviceability rather than to safety. Structural systems and components are considered to be passive (i.e., there is no change of state) in mitigating design-basis conditions. Recent probabilistic risk assessments of NPPs confirm that structural systems are important to the overall safe operation of a NPP.[257] Structural components are more likely to be involved in common-cause failures, since structural failures may affect plant safety systems. Thus, aging in structural components may also be significant in common-cause failures. Moreover, in comparison with mechanical and electrical equipment, structural components are less readily inspectable. Furthermore, it may be very difficult to access and repair structural components and systems without major impact on the operation of the NPP. In fact, replacement of some structural components (e.g., containment and basemat) may not be feasible.

Condition assessment and management of aging in NPP concrete structures requires a more systematic approach than simple reliance on existing code margins of safety.[258] What is required is the integration of structural component function, potential degradation mechanisms, and appropriate control programs into a quantitative evaluation procedure. A methodology for demonstrating the continued reliable and safe performance of these structures should include (1) identification of structures important to public health and safety; (2) identification of environmental stressors, aging mechanisms and their significance, and likely sites for occurrence; and (3) a monitoring or in-service inspection (ISI) based methodology that includes criteria for resolution of existing conditions. Basic background information for formulation of such a methodology has been provided previously in this report. As each NPP is unique, only guidance for development of a plant-specific evaluation program is provided in the balance of this chapter. Utilization of results from the inspection methodology to estimate future performance and evaluate the impact of in-service inspection/repair programs on structural reliability is provided in the next chapter.

4.1 SELECTION OF SAFETY-RELATED CONCRETE STRUCTURES

Category I (safety-related) NPP reinforced concrete structures were described in Chapter 2 and Table 2.1. Tables 2.2 and 2.3 listed primary reinforced concrete structures and their subelement division for boiling-water reactor (BWR) and pressurized-water reactor (PWR) plants, respectively. Because of the complexity of many of the Category I structures, their subelement division was based on geometric differences, structural behavior/performance characteristics, and environmental exposure. Selection of subelements of importance to structural aging for evaluation can be based on an aging assessment methodology, use of probabilistic risk assessments, or a combination of these two approaches.

4.1.1 Aging Assessment Methodology

An aging assessment methodology has been developed that can be used to rank structural components with respect to aging considerations.[259] Basic components of the methodology, determination of structural components important to aging assessments, and examples illustrating application of the methodology are presented below.

4.1.1.1 Basic methodology

Basic components of the methodology are described below.

<u>Subelement Importance (I)</u> The performance, or importance, of the subelements of a Category I structure is difficult to assess quantitatively. Factors related to structural behavior, response to environmental effects, site-specific requirements, etc., must be considered. Additionally, the physical boundaries of each subelement must be defined. A qualitative, or "relative," importance among subelements may be established, however, and associated importance factors assigned. The primary purpose for these importance factors is to provide a measure of the structural contribution of a particular subelement. For simplicity, assignment of importance factors is on a 1 to 10 scale, with 10 being most important. Importance factors that have been assigned to the various subelements of the reinforced concrete structures at typical BWR and PWR plants are also provided in Tables 2.2 and 2.3, respectively.

<u>Safety Significance (SS)</u> The SS of a particular subelement is also of importance in the selection of components for evaluation. Based on criteria provided in sources such as Parts 50 and 100 of 10 CFR,[4] and Refs. 14 and 260, nine specific safety-significance functions have been identified and include

- prevention of uncontrolled liquid or airborne radiation release,
- radiation attenuation and shielding,
- structural support for nuclear steam supply system and containment internal equipment,
- structural support for redundant safety-related equipment and components,
- structural support for ultimate heat sink equipment and components,
- support for new/spent fuel pools and other pool loads,
- protection of safety-related equipment/components,
- separation or "communication" function, and
- failure could damage safety-related component.

A system has been developed to assign a ranking to a subelement in terms of the significance of the safety function(s) that the subelement performs. The SS ranking system is based on a 0 to 10 scale, with 10 being most significant. Table 4.1 lists general criteria for use in assigning a safety significance ranking to a subelement. Definitions of the safety-significance functions and more detailed description of the ranking system is provided in Ref. 259.

Environmental Exposure (EE) Since the environment within which the subelement operates is of great importance to its durability, the severity of the EE must also be included in the selection process. Table 4.2 lists seven EE categories that have been developed. An EE rating has been assigned to each category based on several considerations: (1) historical performance data, (2) degree of exposure of the subelement, (3) accessibility of subelement's surfaces for inspection, and (4) severity of exposure. EE ratings have been assigned to each of the seven categories presented in Table 4.2. The numerical ratings are based on a 0 to 10 scale, with 10 being most severe.

Degradation Factor Considerations The effect of an environmental stressor on a reinforced concrete subelement is initially related to alteration of the component's constituent materials. As the severity of the degradation advances, the integrity, as well as the functional characteristics of the subelement may be compromised. Four criteria are considered to be of importance in assessing the significance of various degradation factors to which NPP reinforced concrete structures can be subjected: (1) rate of deterioration, (2) capability for inspection and early detection of degradation, (3) repairability of the subelement affected, and (4) ultimate impact of the degradation factor(s). The relative significance of these four criteria in terms of degradation factors of most significance to each of the primary materials utilized in the construction of reinforced concrete NPP structures is presented in Table 4.3. The relative effects with respect to the four criteria for each material system degradation factor were assigned on the basis of the potential impact on the subelement's serviceability or integrity [i.e., high (creates a major limiting condition or has a major adverse effect), moderate (creates a minor effect), or low (creates very little or no effect)].

Assignment of numerical values (i.e., degradation factor grading values) to degradation significance has been done through consideration of historical performance of reinforced concrete NPP structures and the potential impact of the specific degradation factor. In developing composite degradation factor grading values, a range of values was developed to accommodate differing conditions at light-water reactor facilities associated with environmental or material characteristics, presence of protective coatings, etc. The limits for the range of possible degradation factor grading values are based on a maximum of 10, indicating potential major significance, and a minimum of 1, indicating minor significance. The ranges of degradation factor grading values presented in Table 4.3 were developed based on information such as provided in Refs. 13 and 23–25.

The net effect of degradation in a reinforced concrete structure may be loss of monolithic behavior, damage to the steel reinforcing system, loss of concrete section, or decreased structural material performance. In order to prioritize the consequent net effects of degradation, relative importance multiplier values were assigned to each of the net effects of degradation (i.e., numerical value from 0 to 1, with 1 most critical). These values are provided in Table 4.4 and are based on a comparative assessment of the potential impact of each of the net effects listed on the performance of the particular subelement. Importance values were then used to prioritize the overall importance of the subelement to the performance of its parent structure. Subelement importance values assigned to pertinent structural subelements (e.g., shell, foundation, etc.) are presented in Table 4.4. The assignment of values is based on a hierarchy of structural subelements.[16] Prioritization of consequent net effects is then developed by multiplying the particular subelement relative importance value by the relative importance multiplier value. The resulting matrix is provided in Table 4.4. The resulting prioritization of consequent net effects also is based on a 1 to 10 scale, with 10 being most important.

These results were used to develop a generic range of degradation factor grading values for specific degradation factor/subelement combinations. The procedure included

(1) identifying specific degradation factors of importance to each subelement and using Table 4.3 to assign appropriate ranges of degradation factor grading values (e.g., corrosion of steel reinforcement indicates a range of 6 to 10),

(2) classifying each subelement into one of six classifications provided in Table 4.4 (e.g., beam or column) and determining "worst case" consequent net effect for each subelement (e.g., corrosion of steel reinforcement of beam results in a value of 7.0),

(3) calculating ranges of degradation factor grading values in terms of degradation factors and structural subelements by multiplying the upper and lower degradation factor grading values obtained from Table 4.3 [e.g., 10 and 6 from step (1)] by the pertinent subelement relative importance value provided in Table 4.4 [e.g., 7 from step (2)] and dividing by 10 [e.g., range is 7 to 4.2 for this example], and

(4) making appropriate adjustments to resulting range of degradation values such as rounding to closest integer and accounting for specific subelement applications and age-related degradation experience data base.

Table 4.5 provides a generic range of degradation factor grading values that was developed for concrete and metallic materials using the above procedure. Selection of a particular degradation factor grading value would be based on an assessment of conditions at the specific plant being addressed.

4.1.1.2 Ranking system to select reinforced concrete components for evaluation

A standard ranking system has been developed to characterize reinforced concrete components in terms of aging importance. The aging assessment methodology takes into account the structural and safety aspects as well as the likelihood for the structure to degrade over time. The quantitative ranking system is based on four criteria: (1) structural importance of subelements, (2) safety significance, (3) environmental exposure, and (4) degradation factor significance. The approach for the methodology is outlined in Fig. 4.1 and involves five primary activities: (1) identification of Category I structures, (2) subelement division of each structure, (3) determination of degradation factor grading values, (4) calculation of degradation significance, and (5) ranking of subelements and primary structures.

Identification of Category I Structures Safety-related reinforced concrete structures in the plant of interest are obtained from documentation such as the plant safety analysis report (SAR), Q-listings, and structural drawings/specifications. Other pertinent information such as design and construction documents, maintenance and in-service inspection records, and operating records should also be collected for later use, as required.

Subelement Division Using structural drawings, original design calculations, and plant specifications, each Category I concrete structure is divided into pertinent subelements. A subelement is defined as a component such as a floor, column, beam, etc., that performs a specific or unique function (structural or otherwise), or that is exposed to a different operating environment. The boundaries for the subelements are established at this stage in the overall procedure. The intent of subelemental division is to enable the most critical components or structures to be identified with respect to structural aging importance.

Constituent materials, as well as their properties and characteristics, should be identified and listed for each subelement. These materials are typically identified in the safety analysis report, structural drawings, specifications, and construction-stage material test reports.

Determination of Degradation Factor Grading Values As noted previously, four criteria are used to develop the quantitative ranking system for use in assessing aging importance of the structures/subelements.

(1) Importance factors for the subelements are available from Tables 2.2 and 2.3 for BWR and PWR plants, respectively.

(2) SS ranking values are obtained from Table 4.1 for each primary structure and the appropriate value is applied to each structure's subelements.

(3) EE ratings are developed for each subelement based on Table 4.2.

(4) Degradation factor grading values are determined. Subelements are evaluated in terms of materials of construction, existing conditions, and environmental exposure. Potential degradation factors are listed and degradation factor grading values assigned using the ranges presented in Table 4.5.

Calculation of Degradation Factor Significance After listing and assigning a degradation factor grading value to each subelement as described in the previous section, a conditional degradation factor significance value is calculated. The degradation factor significance value for a subelement is a simple average of up to three of the most significant degradation factor grading values and is calculated as follows:

$$DFS = \left\{ \sum_{i=1}^{n} DFG_i \right\} / n, \tag{4.1}$$

where

DFS = degradation factor significance value,
DFG = degradation factor grading value, and
n = number of degradation factors, up to a maximum of three.

The resulting DFS value is to be rounded to the nearest integer, with a maximum possible value of 10.

Ranking Of Subelements and Structures Determination of the relative ranks of the Category I concrete structures and subelements is based on the following procedure. A subelement rank within each Category I concrete structure is determined as follows:

$$SR = w_1\,(I) + w_2\,(SS) + w_3\,(DEG), \tag{4.2}$$

where

SR = subelement rank,
I = subelement importance,
SS = safety significance,
DEG = (EE + DFS)/2, rounded to nearest integer,
EE = environmental exposure,
DFS = degradation factor significance,
w_1 = weight factor applied to subelement importance,

w_2 = weight factor applied to safety significance, and
w_3 = weight factor applied to degradation effects.

Use of weighting factors (1 to 10, with 10 highest) permits certain components of Eq. (4.2) to be emphasized. The degradation factor significance was considered to be heavily influenced by the surrounding environmental exposure. Therefore, these two criteria were combined, averaged, and rounded to the nearest integer. Recommended values for w_1, w_2, and w_3 are 4, 9, and 7, respectively. These values are the result of a sensitivity analysis.[259] Therefore, Eq. (4.2) becomes

$$SR = 4\,(I) + 9\,(SS) + 7\,(DEG). \tag{4.2)'$$

Utilization of this equation results in a possible range of subelement ranks between 20 and 200.

The cumulative rank for each Category I concrete structure is determined as follows:

$$CR = \sum_{i=1}^{N} SR_i / N, \qquad (4.3)$$

where

CR = cumulative rank,
SR = subelement rank, and
N = number of subelements.

Application of this equation ensures that the cumulative rank of a Category I concrete structure is based on aging importance rather than total number of subelements. Since the cumulative rank that results for a structure may not adequately reflect the importance of a specific subelement or degradation factor due to the balance of subelements or degradation factors being ranked low, results of both Eqs. (4.2)' and (4.3) should be considered when identifying critical structures and degradation factors of concern.

A computer-based matrix format was developed to provide a simple method for implementing and documenting results of the structural aging assessment methodology. "R:BASE for DOS" was used as the software because of its flexibility of input format (integers, text, etc.), compatibility with other IBM software, and user friendliness. Also, the software allows the programming of important equations [Eqs. (4.1–4.3)] such that the ranks for each subelement/structure are automatically computed after data entry. The end product of the computer-based matrix presentation is a concise listing of Category I concrete structures and their subelements. The ranking values that result may be used to identify structures/subelements and degradation factors that are of most significance with respect to aging management. The resulting list may also be used to assist in determining appropriate methods and schedules for inspection/testing/maintenance.

4.1.2 Probabilistic Risk Assessment

An evaluation of the impact on plant risk due to structural aging can also be used in the selection of structural components for evaluation. Probabilistic risk assessments conducted to date indicate that the structural systems generally play a passive role in mitigating design basis (or larger) internal initiating events; a notable exception being the pressure-retaining function of the containment following a degraded core incident involving failure of the reactor pressure vessel. On the other hand, the structural components play an essential role in mitigating extreme events initiated by earthquake, wind, and other extreme influences, and their failure probabilities due to external events can be higher. Moreover, failure of major structural components may impact the operation of a number of mechanical and electrical systems and lead to so-called "common cause failures." Thus, deterioration of structural components and systems due to aging and other aggressive environmental influences may be more serious in terms of overall plant risk than might be evident from a cursory examination of their role in accident mitigation. The significance of structural aging and deterioration to plant risk can be evaluated by considering the impact that they have on risk associated with external initiating events, especially earthquakes. It is in mitigating the effects of strong ground motion due to earthquakes that structural systems play a particularly significant role.

The increase in risk due to structural aging and deterioration was examined within the framework of a seismic probabilistic risk assessment. Zion was selected as the paradigm NPP because it has been widely reviewed and studied previously, its probabilistic risk assessment (particularly the seismic hazard analysis, fragility analysis, and plant logic) is scrutable, and the core damage and dominant plant damage state depend on a mix of structural, mechanical, and electrical components. The approach was typical of that for a seismic probabilistic risk assessment[261] and involved four primary steps: (1) identify seismic hazard from potential seismogenic sources, historical seismicity in the vicinity of the plant, and attenuation of ground motion to the site; (2) develop plant logic to explain the interaction of various plant components and systems in mitigating the effects of initiating events; (3) develop fragility models to determine capacity of plant components and systems probabilistically; and (4) measure risk by calculating core damage probability and plant damage state. With respect to measurement of risk, probability of seismically-induced core damage on plant damage state leading to release was used as a surrogate. Plant logic models identified the shear wall between the auxiliary and turbine buildings, the roof of the crib house enclosure, and the pressurizer enclosure roof as structural components of most importance. The impact of aging on the fragility parameters was determined. The impact of structural component aging on plant risk and seismic margins was evaluated through changes in the cumulative distribution function of the probability of core damage; the high-confidence, low-probability-of-failure; and a point estimate of risk. Results of this study, presented in Ref. 262 indicate that substantial damage to structural components due to aging leads to less than an order-of-magnitude increase in core damage probability. The apparent impact of structural aging becomes more important if a margins analysis is used to assess suitability for continued service. Sensitivity analysis can help to identify the structures of importance that should warrant particular attention.

4.1.3 Combined Methods

The recommended strategy for use in the selection of structural subelements for evaluation is to use a combination of the aging assessment methodology and probabilistic risk assessment approach. In this manner, in-service condition assessments (or periodic maintenance actions) that may be required for continued service can focus on a selected subset of structural components that have the potential to impact plant safety. Other structural components should also receive at least a visual inspection during routine operation or maintenance, but would require no detailed or invasive inspections/evaluations unless demonstrable problems are apparent. Critical structural components could be identified through a three step process:

(1) apply aging assessment methodology described in Sect. 4.1.1 to develop a ranked list of structures and their subelements;

(2) the plant logic models developed under the Individual Plant Examinations for External Events (IPEEE) program for the particular NPP should be overlaid

with results of step (1) to develop a master list of structural components important to aging, and

(3) sensitivity analyses, as described in Ref. 262, should be performed to identify those components that may be critical in terms of aging.

One of the goals of the sensitivity analysis in step (3) is to establish whether aging of any inaccessible structural component or system is likely to have a significant impact on plant risk. These components will require special consideration for inspection as noted previously.

4.2 CONSIDERATIONS FOR DEVELOPMENT OF ISI PROGRAMS

ISI programs for safety-related NPP reinforced concrete structures have the primary goal of ensuring that these structures have sufficient structural margins to continue to perform in a reliable and safe manner. A secondary goal of these programs is to provide a means to identify any environmental stressor or aging factor effects before they reach sufficient intensity to potentially degrade structural margins.

Results of a survey questionnaire[56] sent to U.S. utilities indicated that only a few utilities are conducting inspections of the safety-related concrete structures beyond the minimum to comply with requirements of the *Code of Federal Regulations*.[4] An approach for use in selecting reinforced concrete structures and identifying potential degradation factors was described in the previous section. General information on an approach for conducting a condition survey was provided in Sect. 3.2.1. Considerations for use in the establishment of an effective in-service inspection program to meet the above goals and to manage the aging of these structures are provided in the form of information on inspection methods, accessibility limitations, and acceptance criteria. Also presented is material pertaining to scheduling and qualification requirements for inspection personnel.

4.2.1 Inspection Methods

Inspection and testing methods generally fall into four categories: (1) visual inspection, (2) nondestructive testing, (3) destructive testing, and (4) analytical assessments. Detailed information on the various testing methods and condition assessments was provided in the previous chapter. General discussions addressing applications of these methods is provided below.

4.2.1.1 Visual inspection

Although relatively simple in principle, 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 scope of the visual inspection should include <u>all</u> 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). Comprehensive direct viewing may require the temporary installation of ladders, platforms, or scaffolding. Use of binoculars, fiberscopes, and other optical aids (e.g., indirect inspection) may be required under certain conditions. Resolution capabilities should be comparable to those contained in Ref. 157. Degraded areas of significance are measured. For crack investigations, a feeler gage, optical crack comparator, or crack width meter can be used to quantify width and depth (if possible).[224] 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. The physical condition and alignment results can be documented through close-range photogrammetry that provides a computer file of the mapped surface or geometry for reference to results obtained from future inspections.[263]

4.2.1.2 Nondestructive testing

Nondestructive testing techniques employ specialized equipment to obtain specific data about the structure in question, and in certain instances (e.g., inaccessible surfaces) its surrounding environment (i.e., structure-specific or environment-specific). The structure-specific methods are used to inspect internal portions of the structure for discontinuities (e.g., presence of voids, cracks, and steel reinforcement) or to provide an indication of constituent material characteristics (e.g., compressive strength, modulus of elasticity, and size of steel reinforcement). Available techniques, their applications, and advantages and disadvantages were described in Sect. 3.1. Generally, the most comprehensive means of assessing structural condition and increasing the probability of defect detection is to use two or more of these techniques in tandem (e.g., ultrasonic pulse velocity and rebound hammer). 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 are used to 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). Methods employed are primarily based on chemical evaluations that provide results such as chloride or sulfide contents of groundwater adjacent to the structure. Table 4.6 provides a listing of several candidate test methods. If results of these tests indicate that the environment adjacent to the structure is not aggressive, there is some justification 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.

4.2.1.3 Destructive testing

Destructive testing involves the removal of samples of material from the structure for the purpose of determining physical, chemical, or mechanical characteristics. These methods were described in Sect. 3.1. Since destructive testing involves a direct examination of the material sample removed, it provides information of significant value for use in aging management programs. Both the presence and impact of deterioration can be determined quantitatively. Also, supplemental testing can be done using these samples to indicate future performance (e.g., durability evaluations through accelerated testing techniques and demonstrating that alkali-aggregate reactions are or are not a future concern). Where material sampling is permitted, generally only a limited number of samples can be removed to minimize the impact on the ability of the structure to meet future functional and performance requirements. However, many structures in NPPs may not be suitable for removal of test samples because of accessibility limitations or structural considerations. One relatively easy and cost-effective approach for providing material samples for testing is to retain materials that are removed during a plant modification. These samples can be used either to evaluate current material characteristics or for potential future requirements.

4.2.1.4 Analytical methods

Analytical methods involve the use of supplemental calculations or analytical procedures to reevaluate the behavior and resistance of the structure (e.g., structural margins). This reevaluation may be required due to either a change in performance requirements (e.g., plant modification) or the identification of deterioration. Finite-element and ultimate strength design methods[16,17] provide two techniques for reanalysis. Reference 264 provides additional information on use of analytical methods.

4.2.2 Accessibility Limitations

Access to the reinforced concrete structures in NPPs may be limited due to a number of conditions: radiation and radioactive contamination (both during normal operations and outages); thermal gradients and gaseous environments; massive size of structures; and presence of surrounding liners, protective coatings, or soils. Few of these structures are accessible on all surfaces for conduct of inspections. Depending on the extent of inaccessibility, this can result in requirements for use of indirect methods such as environmental assessments to supplement results obtained from accessible surfaces. This approach has been used successfully at NPPs to extrapolate limited results to indicate the general condition of the entire structure.[27,123,265,266] If deterioration is identified or suspected, additional testing and evaluation is required. Information pertaining to accessibility of subelements of NPP reinforced concrete structures was presented in Table 2.1.

When the entire structure is inaccessible, or only a very small section is available for inspection, the inspection method shifts from visual-based towards environmental qualification. In this case, the environmental conditions potentially affecting inaccessible portions of the structure are quantified using methods such as listed in Table 4.6. If environmental evaluations indicate that the ambient exposure is non-aggressive, no further action is probably required. However, if the ambient exposure is found to be potentially aggressive, additional testing and evaluation is required. This will involve exposing the structure for visual or nondestructive inspections, removal of material samples for testing, or a combination of these two approaches. The extent of the material removed to expose the structure and the inspection methods utilized will depend on the objective of the inspection (e.g., compressive strength, type of deterioration, detectability requirements, and overall importance of the structure to safety).

4.2.3 Acceptance Criteria

The influence of degradation on the performance and function of reinforced concrete structures is difficult to assess. Material discontinuities, such as steel impurities or local regions of improper concrete consolidation, unless excessive, are generally of minor structural significance. However, errors during construction and the initiation and propagation of various degradation mechanisms may result in loss of function and inability to provide resistance to applied loadings. Often the degradation mechanisms occur at time-varying rates (e.g., chemical attack or migration of chloride ions). In-service inspections of structures at risk are conducted to identify and mitigate the potential degradation factor effects before a repair is required or structural margins have eroded to unacceptable levels. The mechanisms of primary concern for NPP reinforced concrete structures and their manifestations have been identified,[56,214,259] and are summarized below in terms of material affected and potential effect.

Constituent	Mechanism	Structural Effect
Concrete	Chemical Attack Thermal Exposure Irradiation Vibration/Fatigue Cement-Aggregate Reaction	Cracking, erosion, leaching of paste, increased permeability Cracking, loss of mechanical properties Cracking, loss of mechanical properties Cracking, loss of strength and damping Cracking, loss of certain mechanical properties
Conventional Reinforcing Steel	Corrosion Other Mechanisms	Loss of monolithic behavior; loss of strength; cracking and spalling of concrete Loss of bond, change in mechanical properties
Prestressed Reinforcing Steel	Corrosion Fracture Stress Relaxation	Loss of section and capacity Loss of capacity, loss of ductility, increased concrete cracking under load Loss of prestress force, increased concrete cracking under load
Liner and Embedments	Corrosion	Reduction in leak-tightness for postulated loads, loss of section/capacity

Several common characteristics may be observed from this categorization. First, concrete cracking is a very common damage by-product from a large number of 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 determination of cracking cause and means for propagation in concrete structures are of primary interest for future inspections. Second, 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 have been developed for assistance in the classification and treatment of conditions or findings that might emanate from in-service inspections of NPP reinforced concrete structures. These approaches are based primarily on the results of visual inspections since these inspections provide the cornerstone of any condition assessment program for concrete structures. Also, with the exception of some guidance on half-cell potential[197] and ultrasonic pulse velocity measurements,[239] few standards have been published presenting acceptance criteria for results obtained from nondestructive evaluation tests. Background information is presented below for use in formulating acceptance criteria based on results obtained from visual inspections.

4.2.3.1 Visual approach

The visual-based approach uses a "three-tiered" hierarchy similar to that under development by American Concrete Institute (ACI) Committee 349.[230] 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. The three acceptance levels include acceptance without further evaluation, acceptance after review, and additional evaluation required.

Acceptance Without Further Evaluation Conditions presented below are considered to be acceptable and require no further evaluation at present. Reference 223 provides definitions and pictorial representations of typical forms of concrete degradation. In the event that the conditions provided below are exceeded, or observed conditions are determined to be deserving further evaluation, a more detailed review is required. Structures that are partially or totally inaccessible for visual inspections may require supplemental evaluations as environments may be present that are conducive to degradation.

1. Concrete Surfaces

   Concrete surfaces that are exposed for inspection and meet the following surface condition attributes 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, settlements, or other physical movements that may affect structural performance; and
   j. Absence of cement-aggregate reactions, chemical attack, or other active degradation mechanism.

2. Concrete Surfaces Lined by Metal or Plastic

   Concrete structures with 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 leak chases or other detection system components.

---

* Dimensions of degradation presented in this section are meant only as guidelines.

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 of the exposed portions of embedded metal surfaces 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 requirements such as provided in Refs. 105 and 157 are met. These requirements are well defined.

Acceptance After Review Findings listed below require review and interpretation in order to evaluate acceptability. Such a review involves determining the likely source of degradation, its activity level, and its net effect on the component. Based on results of the review and evaluation, possible approaches include acceptance as-is, further evaluation using enhanced visual inspection (e.g., magnification), scheduling follow-up inspections at a later date, or use of nondestructive or destructive testing techniques. An analytical assessment of the necessity for repair may also be required. The analytical assessment should examine the impact of existing degradation on the performance characteristics of the structure. Accessibility of the components in question will also enter into the decision process relative to the action to be taken.

1. Concrete Surfaces

The following surface conditions shall be reviewed to determine if they are either acceptable, require further evaluation, or require repair. Measurable discontinuities exceeding the quantitative limits below require additional evaluation.

* Information on protective coatings for NPP applications is provided in Refs. 267 and 268.

a. Appearance of leaching or chemical attack;
b. Areas of abrasion, erosion, and cavitation degradation;
c. Drummy areas that may exceed the cover concrete thickness in depth;
d. Popouts and voids greater than 20 mm but less than 50 mm in diameter or equivalent surface area;
e. Scaling greater than 5 mm but less than 20 mm in depth;
f. Spalling greater than 10 mm but less than 20 mm in depth, and less than 200 mm in any planar dimension;
g. Corrosion staining on concrete surfaces;
h. Passive cracks greater than 0.4 mm but less than 1 mm in maximum width; and
i. Passive settlements or deflections exceeding the original design limits or expected values.

2. Concrete Surfaces Lined by Metal or Plastic

   a. Without Active Leak Detection System
   Presence of any condition listed in Criteria 2(a) of previous section shall be further evaluated to determine acceptability; and

   b. With Active Leak Detection System
   Presence of leakage in excess of amounts and flow rates committed to in the original design or Plant Technical Specification will necessitate a root cause investigation and assessment of the need for follow-up action. Leakage within the prescribed limits may be acceptable if the source is known and found to be inconsequential.

3. Areas Around Embedments in Concrete

   Presence of any condition listed in Criteria 1 for concrete surfaces shall be further evaluated to determine acceptability.

4. Joints, Coatings, and Non-Structural Components

   Presence of any condition exceeding the descriptions and limits of Criteria 4 in previous section shall be further evaluated to determine acceptability. Any observation of widespread adhesion/cohesion problems, environmental attack, or poor performance indicators are considered unacceptable.

5. Post-Tensioning System

   Presence of conditions exceeding limits provided in references noted under Criteria 5 of previous section shall be reviewed according to criteria provided in these references.

__**Additional Evaluation Required**__ Conditions outside the criteria provided in the previous two sections must be evaluated to determine the appropriate coarse of action. This will generally involve extensive application of both nondestructive and destructive testing methods. Detailed analytical evaluations frequently will 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). If the structure's desired service life is short, and its loss of function due to degradation is occurring at a rate such that sufficient structural margins will be maintained during this period, no action may be required. However, when the opposite is true and loss of function due to degradation is occurring at a rate such that structural margins will not be adequately maintained during the desired service life period, the analytical and test results should be utilized to develop an in-service inspection/repair strategy that will maintain structural margins during the desired service life.

4.2.3.2 Degradation-based approach

The effects of degradation mechanisms on the performance of a structure can range from cosmetic to structurally degrading. Provided below is information that is intended to be of assistance in quantifying the significance of degradation that is detected through visual inspections, nondestructive testing, or a combination of these methods. Forms of degradation considered include concrete cracking and cement-aggregate reactions. Guidance for detection and resolution of degradation in the form of corrosion of post-tensioning system components and loss of prestressing force is not addressed as detailed information on these topics is available in Refs. 105, 106, and 157. Degradation of metal liners or coating materials is also not addressed as these materials were not generally addressed under the SAG Program.

Concrete Cracking Cracking in concrete can result from a number of factors as shown previously in Fig. 2.9. Designs of reinforced concrete structures generally consider that the concrete is incapable of supporting tensile forces. Steel reinforcement is included in the structural members to both carry the tensile loadings and to provide control of cracking (i.e., limit width and spacing of concrete cracks). From an aging perspective, both the width of concrete cracks and the environmental exposure are important.

Limited information on cracking and its classification with respect to damage is available in Ref. 58. In this reference, damage resulting from different crack types (e.g., diagonal and longitudinal) is rated on a one to five scale, with five being the most significant. Table 4.7 presents the classifications and ratings that were developed for concrete cracks and surface defects. Environmental influences have not been included in this classification scheme. The more severely rated (i.e., greater crack widths) cracks generally would be related to an overload condition that would require a structural evaluation. Active cracking, settlements, or deflections that are observed must be carefully classified and evaluated as the cause may continue to act and its effect intensify.

From an aging management perspective, the presence of concrete cracks is of importance because they provide possible avenues of access for environmental stressors (e.g., chloride ions and sulfate solutions). Some work has been done in classifying environmental exposure conditions in terms of degree of aggressivity. Table 4.8 provides an indication of the influence of moisture state on several durability processes.[52] The Comite European de Normalisation has prepared a chart classifying environmental conditions in terms of severities of exposure classes (Table 4.9).[269] Exposure classes have also been developed specifically relating environmental conditions to steel reinforcement corrosion (Table 4.10), [52] and degree of chemical attack of concrete by water and soils containing aggressive agents (Table 4.11).[52,270]

As noted previously, the corrosion of steel reinforcement is one of the more significant (if not the most) forms of degradation that could potentially impact NPP reinforced concrete structures. There have been a number of studies over the years that have related maximum permissible concrete crack widths to environmental factors, and these results were summarized in Table 3.5. Limits in this table were provided to reduce the potential for enhanced degradation through ingress of contaminants, primarily leading to corrosion of steel reinforcement.

Two damage-state charts have been prepared to assist in the resolution of results obtained from in-service inspections or testing.* Figure 4.2 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 4.3 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. Crack width information presented in Table 3.5 and limited industry-published acceptance criteria[197,247] were used to develop the relationships between threshold levels and recommended actions presented in Figs. 4.2 and 4.3. Results are presented in terms of a "three tier" hierarchy: (1) condition acceptable as-is without additional assessment; (2) structure requires further evaluation, supplemental tests, or review after next operating period to classify as acceptable or in need of repair or maintenance; and (3) structure requires repair, maintenance, or replacement. Further evaluation would consider the use of other inspection, testing, or analytical tools to obtain additional information on the current condition of the structure and the potential for further degradation of its functional and performance requirements with time.

Alkali-Aggregate Reactions** Limited research results are available presenting the impact of alkali-aggregate reactions on structural integrity (i.e., alkali-silica, alkali-carbonate, and alkali-silicate). As a result of these reactions, expansion and cracking occurs that can lead to loss of strength, reduced stiffness, or decreased durability of concrete. A quantitative ranking methodology has been developed for beam and plate elements that potentially can be used as guidance for NPP reinforced concrete structures should the presence of alkali-aggregate reactions be confirmed.[271] The criteria below were developed on the basis of visual inspections and petrographic analyses of core specimens removed from a large number of structures exhibiting various intensities of alkali-aggregate reactions.

CATEGORY	VISUAL INSPECTION	PETROGRAPHY RESULTS
1	Crack Width: 0–0.2 mm Crack Depth: Superficial Pop-outs, no.: 0–5 per m^2 Pop-outs, dia.: 0–5 mm	Internal and external circumferential cracks developed. Some gel exuded. Internal cracks in reactive aggregate.
2	Crack Width: 0.2–1.0 mm Crack Depth: Superficial to Deep Pop-outs, no.: 5–20 per m^2 Pop-outs, dia.: 5–15 mm	Gel in air voids and small external cracks. Short cracks open to environment along major axis of structure.
3	Crack Width: 1.0–2.0 m Crack Depth: Deep to Penetrating Pop-outs, no.: 15–30 per m^2 Pop-outs, dia.: 5–15 mm	Gel in many air voids and cracks. Marked increase in crack width (0.05 mm internal). Longer cracks along major axis of structure.
4	Crack Width: > 0.2 mm Crack Depth: Penetrating Pop-outs, no.: > 25 per m^2 Pop-outs, dia.: 5–20 mm	Most reactive aggregate shows signs of reactivity. Larger cracks along major axis intersect with transverse cracks showing pattern. Gel exudation easily seen in cracks and voids.

* The damage-state charts are intended only to represent the type of information that can be developed to aid in the structural condition assessment. Every reinforced concrete structure is unique with respect to its constituent materials and physical characteristics, functional and performance requirements, and environmental exposure conditions.

** The ultraviolet radiation method[219] described in Sect. 3.1.2.6 can be used to detect the presence of alkali-silica reactions in in situ concrete structures.

For structures having sustained Category 1 or 2 damage, the reactions have likely not caused significant structural damage. Structures observed as having damage in Category 1 should be considered for more frequent inspection and possibly for rehabilitative measures similar to that for Category 2 damage. For structures in Category 2, maintenance measures aimed at preventing exposure to moisture, such as adding a protective coating or sealer, should be considered. Additional core samples may be needed to assess the degree of reaction. More frequent inspection is also warranted. Structures in Category 3 and 4 require evaluation for structural repair. Because a single aggregate source was generally used in the construction of a NPP, the balance of plant structures should be inspected if Category 3 or 4 conditions are observed in one structure.

4.2.4 Inspection Scheduling and Personnel Qualifications

4.2.4.1 Inspection scheduling

ISI programs for NPPs have traditionally focused on the concrete containment vessel. A general visual inspection of accessible surfaces is conducted in conjunction with performance of periodic Type "A" leak-rate tests required by Appendix J to 10 CFR Part 50.[4] Examination of the unbonded post-tensioning tendon system is conducted at regular intervals to assess the state of the hardware as well as level of prestressing force in the containment.[155] Schedules for the containment-related inspections are considered to be "outage based" in that most or all of these inspections are performed during planned plant outages (e.g., refueling) to provide improved plant access. The plant owner does have the option of electing to perform certain inspections (e.g., tendon surveillances) at other times as long as the code-mandated frequencies and schedules are met. Only containments are covered by current regulatory requirements.

The ACI[230] is developing recommended frequencies and schedules for conducting inspections of safety-related concrete structures other than containments. These schedules take into account the relative aggressiveness of environmental conditions and physical exposures of these structures, and will help assure that any age-related degradation is detected at an early stage of development so appropriate mitigative actions can be taken. In general, the ACI document proposes that all safety-related reinforced concrete structures be visually inspected at intervals not to exceed 10 years. The frequency of ISI proposed is dependent on the exposure category into which the structure of interest falls. The exposure categories are essentially the same as presented in Table 4.2. Recommended ISI frequencies are provided below.

Structure Exposure Category	Frequency of Visual Inspection
• Below-Grade	10 Years (each ISI Interval)
• Natural Environment (Direct/Indirect)	5 Years (two per ISI interval)
• Inside Primary Containment	5 Years (two per ISI interval)
• Continuous Fluid (without liner)	5 Years (two per ISI interval)
• Fluid/Pressure Retaining (with liner)	5 Years (two per ISI interval)
• Controlled Interior (i.e., secondary containment, auxiliary building, etc.)	10 Years (each ISI interval)

The above frequencies may be modified to smaller intervals if plant environments are particularly severe or degradation has been observed to occur. When the observed degradation exceeds criteria provided previously, increased visual inspections should be supplemented by nondestructive, and possibly destructive testing.

Reliability-based methods can also be used to schedule inspections of safety-related concrete structures. These concepts are discussed in detail in the next chapter and can be used to assess the reliability of the NPP reinforced concrete structures in terms of damage state and rate of degradation, inspection methods, detectability functions, remedial actions, and frequency of inspections. Optimized strategies for inspection and repair can be developed that minimize future costs associated with inspection, repair, and loss of service, while maintaining the component probability of failure at or below a target value over the service life of the structures.

4.2.4.2 Qualifications of inspection personnel

The quality and usefulness of results obtained from inspections of existing NPP reinforced concrete structures are dependent to a great deal on the qualifications and capabilities of the personnel involved. To ensure that these inspections are properly performed, minimum qualifications and skills should be defined. Although guidelines are available for inspection of new concrete construction, few standards exist for qualification and certification of inspectors for existing reinforced concrete structures. Some recommendations have been proposed by the ACI Committee 349[230] and they are summarized below.

As a minimum, the complete inspection team should include both civil/structural engineers and concrete inspectors and technicians familiar with concrete aging and degradation mechanisms and long-term performance issues. The qualifications for the person(s) responsible for the in-service inspection of safety-related concrete structures should meet the minimum recommendations of Ref. 230. The individual responsible for the administration of an ACI-defined evaluation is the responsible engineer, who will possess one of the following sets of qualifications:

A. Registered Professional or Structural Engineer, knowledgeable in the design, evaluation, and in-service inspection of concrete structures and performance requirements of nuclear safety-related structures; and

B. Civil/structural engineering graduate of an accredited college or university who has successfully completed the experience, training, and testing requirements of the ACI Level III Concrete Inspector Program and is knowledgeable of the performance requirements of safety-related structures.

Personnel performing the balance of <u>inspection</u> or <u>testing</u> at the plant, under the direction of the responsible engineer, should meet one of the following qualification sets, or equivalent:

A. Civil/structural engineering graduate (4-year) of an accredited college or university who has over 1 year experience in the evaluation of in-service concrete structures or quality assurance related to concrete structures;

B. Personnel possessing Level I or II Concrete Inspector certification from the plant owner; and

C. Personnel meeting the requirements for Level I or II Concrete Inspector, as defined in Section III, Division 2, Appendix VII.[20]

Personnel inspecting metallic components such as liner plates should be certified in ultrasonic thickness, liquid penetrant, or magnetic particle testing if these methods are to be used.

ISI results may need to be examined in terms of net effect on the structure. Structural calculations addressing the as-designed structure and projected behavior under observed degradation should be performed under the direction of the responsible engineer.

4.3 EXAMPLE ILLUSTRATING DEVELOPMENT OF AN INSPECTION PROGRAM

An approach for establishing an inspection program for safety-related NPP concrete structures is illustrated in the example presented below. The approach presented is applicable to concrete structures in both PWR and BWR plants.

4.3.1 Background on Example PWR Plant

The example plant is a PWR with a large-dry post-tensioned concrete containment located in the midwestern U.S. The containment vessel consists of a conventionally reinforced basemat foundation located on compacted fill material, post-tensioned (vertical and 120° hoop tendons) cylinder wall, and post-tensioned (three groups with each group at 120° with respect to other two groups) dome. The post-tensioning system consists of unbonded wire tendons encased in ducts filled with petrolatum-type corrosion inhibitor. The internal surfaces of the containment are lined with 6.25-mm thick carbon steel plate, while the spent-fuel pool is lined with 6.25-mm thick stainless steel plate. The dome external surface is coated by a high-solids urethane paint system. All concrete structures were constructed of concrete having a minimum compressive strength of 28 MPa and conventional steel reinforcement having a minimum yield strength of 414 MPa. Subterranean structures are protected by waterproofing materials and water stops. External structures are subjected to several annual freeze-thaw and wet-dry cycles. Also, seasonal groundwater fluctuates from one to six meters below grade.

Presently the utility is performing regular in-service inspections that comply with requirements in Refs. 4 (Appendix J leak-rate testing) and 105 (post-tensioning system evaluations). A review of the results of these inspections indicates that no significant degradation has been observed to date. Formal inspections of the other safety-related reinforced concrete structures are not done on a routine basis. Spent fuel pool liner leakage is monitored on a regular basis to maintain cooling water inventory. A settlement monitoring program was in-place early in the life of the plant, but was abandoned after uniform settlements stabilized at about 6 to 13 mm.

4.3.2 Selection of Components for Evaluation

The aging assessment methodology described in Sect. 4.1.1 is used to select structural components for evaluation.* The initial step is to identify all Category I concrete structures. The most recent edition of the plant's safety analysis report and other pertinent plant documentation (e.g., Q-listing) are reviewed as well as structural drawings. Primary structures identified for this plant include (1) containment vessel, (2) containment internal structures, (3) auxiliary building, (4) turbine building (portions of slabs/walls), (5) cribhouse (portions), and (6) intake crib.

* For completeness, results of a probabilistic risk assessment identifying structures that would increase plant risk due to aging and deterioration also should be included in the ranking.

These structures were subdivided into subelements and materials of construction. This information is entered into the matrix format as shown in Table 4.12.* Importance factors are assigned to each subelement using information presented in Sect. 4.1.1.1.

Using conservative judgment, review of plant conditions, and guidelines presented in Sect. 4.1.1.1, safety significance and environmental exposure values are established and entered into the matrix format of Table 4.12 for each subelement. The primary containment structure, as well as many of the other safety-related structures at this plant, are exposed to a natural environment. However, a number of these structures have been protected with coatings to prevent external degradation. As noted, the plant is founded on fill material. Also, cooling is provided by fresh water. Certain groundwater parameters are periodically measured, and the post-tensioning system is partially accessible for inspection. Each of these conditions was considered as being favorable with respect to assignment of criteria values.

As a result of variability in existing protective media and exposure conditions at this plant, evaluation of the degradation factors and their grading values requires careful consideration. Such degradation mechanisms as corrosion of the containment post-tensioning system and irradiation of the reactor pressure vessel pedestal were considered to be relatively important. Key potential degradation factors for other structural subelements are then identified and entered into the matrix format of Table 4.12. Degradation factor grading values for each of the potential degradation factors for each of the subelements are assigned based on information provided in Table 4.5. Based on known existing conditions at the plant, these values may be adjusted appropriately. Equation (4.1) is then utilized to determine a degradation factor significance value for each subelement using up to a maximum of three degradation factor grading values for each subelement.

A ranking of each subelement is determined using Eq. (4.2)' in conjunction with the subelement importance, safety significance, environmental exposure, and degradation factor significance values determined above. Finally, the cumulative rank for each structure is determined by summing the subelement ranks and dividing by the number of subelements. The results of these calculations indicate that the highest ranking primary reinforced concrete structure is the containment vessel, and the highest ranking subelement is the mat foundation of the containment vessel. A complete listing of the ranking of primary structures is provided below.**

1. Containment vessel (dome, ring girder, cylinder walls, and mat);
2. Reactor cavity walls and support;
3. Auxiliary building foundation;
4. Reactor coolant compartment walls;
5. Containment-internal walls and short columns;
6. Fuel pool walls and slab;
7. Polar crane support wall;
8. Containment-internal lower slab;
9. Intake crib foundation;
10. Containment-internal slabs (others);
11. Diesel generator vault walls;
12. Control room walls;
13. Diesel generator vault slabs;
14. Control room slab;
15. Auxiliary building walls;

* Detailed results are presented in Table 4.12 only for the post-tensioned concrete containment. Results for other safety-related concrete structures are available in Ref. 259.

** *Structures ranked higher than 20 have been omitted from the listing.*

16. Crib house walls;
17. Auxiliary building floor slabs (internal);
18. Turbine building safety-related walls;
19. Crib house slabs;
20. Turbine building safety-related floors;

4.3.3 Initial Planning

Using results presented above, planning for development of an inspection program is initiated. The intent of this initial planning is to define the scope and type of inspections to be performed on the structures identified above, establish initial inspection frequencies, set up program documentation requirements, and identify personnel to oversee and conduct inspections. The plan should consider all procedures currently in place to meet Plant Technical Specification requirements. A brief visual inspection and review of pertinent documentation, such as related plant modifications or repair activities, is of assistance in developing the inspection plan. This information is then used to develop a detailed plan for conduct of an initial baseline survey of the structures. Guidance on condition assessments including a visual baseline survey is available in Sect. 3.2. In addition, there are several documents that have been prepared by the Corps of Engineers for use in conducting condition assessments and quantifying degradation in civil works structures that provide information of use in planning inspections of NPP concrete structures.[272–274]

4.3.4 Baseline Inspection Program

Initial planning results are implemented through a baseline inspection program. The emphasis of the baseline inspection program is on the primary structures and subelements that were ranked highest by the aging assessment methodology. The approach developed for the baseline inspections includes the following.

- Post-tensioned concrete containment has been inspected at regular intervals under a well-defined in-service inspection program. Since no degradation has been experienced, these inspections will be continued unchanged.

  1. Leakage-rate testing at current specified intervals according to Plant Technical Specification requirements,
  2. General visual inspections of liner and external concrete surfaces, and
  3. Post-tensioning system inspections according to Plant Technical Specifications.

- High-ranking safety-related concrete structures (nos. 2–10 in Sect. 4.3.2) will be visually inspected.

  1. General visual inspection using visual-based approach acceptance criteria in Sect. 4.2.3.1,
  2. Detailed documentation of findings for future reference, and
  3. Reinspect structures using inspection scheduling criteria presented in Sect. 4.2.4.1, or at more frequent intervals for suspect areas.

- Remaining safety-related concrete structures (nos. 11–20 in Sect. 4.3.2) will also be visually inspected, but not in the detail that the previous structures received.

  1. General visual inspection to provide initial baseline information, and
  2. Reinspect structures at five-year intervals with inspection interval increased to ten years if first three inspections indicate no potential deterioration.

The baseline inspection program approach was implemented to examine the 20 highest-ranking concrete structures noted previously. In addition, portions of the intake structure below the water line were inspected, surface hardness measurements (rebound hammer) were made on exposed concrete surfaces to provide an indication of concrete quality, and dry-film measurements were made of coating thicknesses. Environmental exposure information for the structures was estimated from existing plant data. Conclusions from the baseline inspection program were that

1. All safety-related structures were found to be in acceptable condition except for the lower tendon gallery vaults, which were found to have been damaged by leaching, cracking, and steel reinforcement corrosion. Several wall cracks were over 4-mm wide, and had propagated through-thickness over the full height of the wall. Intruding groundwater had caused significant leaching of the concrete, and staining from steel reinforcement corrosion was severe. This cracking was not a current safety issue, but should be addressed in the near future.

2. Significant concrete cracking was discovered in the intake structure walls, Turbine Building walls and basemat, and lower Auxiliary Building walls and basemat. Signs of groundwater intrusion, efflorescence, and reinforcing corrosion were noted. Measured concrete crack widths were generally under 1.5 mm, except for one crack with a maximum width of 15 mm that appeared to be through-wall.

   Leakage of borated spent fuel pool water was found on many Auxiliary Building slabs. Cracks were also identified in the nuclear steam supply system (NSSS) vault walls and structures supporting NSSS equipment inside the containment vessel, although they appeared to be passive in nature.

3. The environmental exposures of all structures, including below-grade structures, was judged to be "mild" with limited chlorides present in the soil and groundwater. The atmosphere was found conducive to carbonation, given the surrounding heavy industry and environmental exposure witnessed at the plant. The groundwater elevation fluctuates from 1 to 6 meters below grade during the year and was found to have a fairly high dissolved oxygen content.

4. Review of tendon lift-off test results indicated that losses to date were well-within expectations. Lift-off tests and tendon inspections have been performed per Regulatory Guide requirements, although no trending was performed. Some water was found in tendon grease samples at several anchorages. Significant grease leakage was noted from grease cans and through anchorage-area concrete cracks, with measured crack widths to 0.5 mm.

5. The containment liner was found to be locally corroded, with pitting and exfoliation to depths of about 50% of the nominal liner thickness at several locations. Liner coating systems were also degraded beyond acceptance criteria and in need of repair. The liner is not a structural member, but a remedial action should be implemented so that continued corrosion will not penetrate the liner thickness to cause a loss of its pressure-retaining function.

6. One badly cracked wall (non-safety related) was identified in the Turbine Building.

7. Only limited local cracking and impact-related spalling was observed in the containment vessel wall. This damage was primarily within a 3-m high band above grade. One spall from vehicle impact measured 200 mm by 370 mm by 38 mm deep.

8. Containment basemat and lower cylinder walls, where accessible and near grade, were found to be in good condition.

4.3.5 Inspection Program Adjustments

Several modifications were made to the initial inspection program as a result of conditions observed during the baseline inspections. Acceptance criteria presented in Sect. 4.2.3 were used to develop the modified inspection program. Modifications recommended to help ensure continued integrity of structures that showed signs of or exhibited increased potential to degrade included the following:

1. Establishing aggressiveness of soil and groundwater through annual sampling and testing of groundwater and soil around containment basemat perimeter.

2. Conducting inspections at intervals not to exceed two years of structures that exhibited water intrusion (i.e., lower portions of containment vessel walls, Auxiliary Building, Turbine Building, and intake structure).

3. Repairing tendon gallery walls and bottom slab with post-repair inspections conducted annually until a performance data base is developed.

4. Checking integrity of Auxiliary and Turbine Building basemats with impact-echo to examine for subsurface damage.

5. Conducting visual examinations of Turbine Building wall to determine significance of cracks that were observed during inspections. Results will be used to formulate a repair strategy using guidance provided in Sect. 3.3.

6. Developing a program within one year to identify source of leakage of spent-fuel pool water and assess potential impact of borated water on concrete structures at risk (i.e., corrosion potential).

7. Evaluating significance to structural performance of tendon corrosion inhibitor leakage at anchorage zone areas.

8. Implementing a program to repair portions of liner experiencing corrosion, recoat, and inspect on an annual basis until a performance data base is established.

The balance of reinforced concrete structures for which the baseline inspections indicated that no further evaluation or remedial action was required will be examined under the initial program requirements.

4.4 COMMENTARY ON INSPECTION PROGRAMS

Regular and systematic inspections constitute an integral part in maintaining the safety and serviceability of the safety-related concrete structures by providing a link between the environments these structures experience and the manner in which they perform with time. This has been recognized through the "Maintenance Rule" that requires power reactor licensees to monitor the performance or condition of structures, systems, and components (SSCs) against licensee-established goals in a manner sufficient to provide reasonable assurance that the SSCs are capable of fulfilling their intended functions (§ 50.65 to Ref. 4).

Development of an inspection program to evaluate NPP safety-related reinforced concrete structures basically entails the selection of critical components, application of systematic inspection routines (including data recording and handling procedures), determination of the current structural condition, and assessing the ability of the structure in question to continue to meet functional and performance requirements. Methodologies for selection of components important to aging and conducting condition assessments have been developed, or are fairly well established. Although some background information was provided previously on criteria for use in condition assessments, development of acceptance criteria relating the impact of degradation to the performance characteristics of the affected structure in large measure require development. Reinforced concrete structures in NPPs present special challenges for development of acceptance criteria because of their massive size, limited accessibility in certain areas, stochastic nature of past and future loads, randomness in strength, uncertainty in material changes due to aging and possibly degradation, and qualitative nature of many nondestructive evaluation techniques. The most effective approach at present to managing aging of these structures is to conduct periodic inspections, implement maintenance measures to prevent or slow deterioration, and repair defects before they propagate to effect a loss of serviceability.

Table 4.1. Safety significance ranking criteria.[a]

Safety Significance Criteria	Ranking
1. Non-Category I structures	0
2. Category I structures whose relation to safety is due to their failure consequence alone	2
3. Category I structures performing one safety function in addition to "failure consequence" (typically related to environmental protection, etc.)	4
4. Category I structures performing two safety functions of greater importance than "failure consequence" (i.e., secondary containment, radiation shielding, etc.)	6
5. Category I structures performing at least three safety functions of greater importance than "failure consequence" and which are required for primary containment, support of NSSS components, etc.	8
6. Category I structures performing four or more safety functions simultaneously (i.e., the primary containment pressure boundary)	10

[a]Intermediate values between those noted above may be used based on actual safety function of the structure. Non-Category I structures are included for completeness.

Table 4.2. Environmental exposure categories and ratings.

Exposure Condition	Rating
1. Subterranean (below grade)	10
2. Natural environment (direct exposure)	6
3. Natural environment (indirect exposure)	4
4. Continuous fluid exposure (without liner)	7
5. Fluid/pressure retaining (liner provided)	5
6. Inside primary containment	7
7. Controlled interior environment (secondary containment, auxiliary building, etc.)	2

Reference 259 provides more complete descriptions of the exposure conditions and criteria for assignment of a rating number.

Table 4.3. Relative weights[a] assigned to criteria affecting degradation factor gradings.

		Criteria				
Material System	Degradation Factor	Rate of Deterioration	Inspectability and Early Identification	Repairability	Ultimate Impact	Range of Degradation Factor Grading Values
Concrete	Chemical attack	Low to high	Low to moderate	Moderate	Low to high	6–10
	Freeze/thaw cycling	Low	Low	Low	Low	2–6
	Thermal exposure/thermal cycling	Moderate	Moderate	High	Moderate	6–10
	Irradiation	Low	Moderate	High	Moderate	4–8
	Abrasion/erosion/cavitation	Low	Low	Low	Low	2–6
	Fatigue/vibration	Low	Moderate	High	Moderate	4–8
Mild Steel Reinforcing	Corrosion	High	Moderate	Moderate	High	6–10
	Elevated temperature	Low	Low	Moderate	Low	2–6
	Irradiation	Low	Low	Moderate	Low	2–6
	Fatigue	Low	Moderate	Low to high	Low to moderate	4–8
Prestressing	Corrosion	High	Moderate	Low to moderate	High	6–10
	Elevated temperature	Low	Low	Low to moderate	Low	2–6
	Irradiation	Low	Low	Low to moderate	Low	2–6
	Fatigue	Low	Moderate	Low to moderate	Low	2–6
	Stress Relaxation	Low	Moderate	Low to moderate	Moderate	4–8
Liner Plate/ Structural Steel	Corrosion	Moderate	Low	Moderate	High	6–10
	Fatigue	Low to moderate	Moderate	Moderate	Low to moderate	4–8

[a]High = Criterion creates a significant limiting condition or has a major adverse effect that can significantly impact the structure's serviceability or integrity following initiation and may be difficult to identify.
Moderate = Criterion creates a minor impact on structures and requires careful inspection.
Low = Criterion creates either very low or no impact on structures or may be readily observed via inspection.

Table 4.4. Prioritization[a] of consequent net effects by generic subelements.

Consequent Net Effect (relative importance multiplier value)	Subelement (Relative Importance Value)[b]					
	Shell (10)	Foundation (10)	Floor (7)	Beam (7)	Column[c] (9)	Shear Wall (8)
1. Loss of monolithic behavior (1.0)	10.0	10.0	7.0	7.0	9.0	8.0
2. Damage to steel reinforcing system (1.0)	10.0	10.0	7.0	7.0	9.0	8.0
3. Loss of concrete section						
a. Gross loss (0.8)	8.0	8.0	5.6	5.6	7.2	6.4
b. Minor loss (0.7)	7.0	7.0	4.9	4.9	6.3	5.6
4. Decreased structural material performance (0.6)	6.0	6.0	4.2	4.2	5.4	4.8

[a]Prioritization of consequent net effects values in table for each subelement represents product of relative importance multiplier value for particular consequent net effect (0-1) and relative importance value of the particular subelement (0-10).

[b]Number represents relative importance value assigned to corresponding subelement according to Ref. 16. These values are <u>not</u> the same as the importance factors assigned in Tables 2.2 and 2.3.

[c]Column or interior partition wall (concrete).

Table 4.5. Range of values for degradation factor grading in terms of subelement type.

a. Concrete Materials

STRUCTURAL ELEMENT	CONCRETE DEGRADATION FACTORS					
	Chem. Attack	Freeze/Thaw	Thermal	Irradiation	Abrasion	Fatigue
1. CONTAINMENT VESSEL						
a. Foundation	6–10	—	2–6	0–4	—	0–6
b. Tendon Galleries	6–10	—	—	—	—	—
c. Vertical Walls	6–8	2–6	3–9	0–4	0–4	2–8
d. Dome	5–9	2–6	2–6	—	—	0–4
e. Suppression Chamber (BWR)	5–9	—	2–6	0–6	—	0–6
f. Polar Crane Support	2–5	—	2–6	0–4	0–4	4–10
g. Ring Girder (PWR)	5–8	2–6	2–6	—	—	2–6
2. CONTAINMENT INTERNAL STRUCTURES						
a. Bottom Slab	6–10	—	2–6	0–8	2–6	3–6
b. Reactor Pedestal (BWR)	6–10	—	4–9	2–8	—	4–8
c. Primary Reactor Shield	6–10	—	4–10	5–10	—	4–7
d. Floor Slabs	2–7	—	2–7	2–7	0–4	0–4
e. Walls	4–8	—	2–6	2–8	0–4	2–6
f. Columns	4–8	—	4–8	2–8	0–6	2–6
g. Beams	2–6	—	2–6	0–6	—	2–6
h. Fuel Pool Slabs/Walls	4–8	—	2–8	2–8	0–4	4–8
3. SECONDARY CONTAINMENT STRUCTURE						
a. Foundations	6–10	—	0–6	—	—	0–4
b. Slabs	2–6	—	0–4	0–4	0–4	0–4
c. Walls	4–8	2–6	0–6	0–4	0–4	—
d. Columns	4–8	2–6	0–4	0–6	0–6	—
e. Beams	2–4	0–4	0–4	—	—	0–4
4. AUXILIARY STRUCTURES						
a. Foundations	6–10	—	0–4	—	0–4	0–4
b. Walls and Columns	2–8	0–8	0–4	—	0–6	0–6
c. Slabs/Beams/Roof Slabs	4–9	0–6	0–4	—	2–8	0–6
d. Cable Ducts/Pipe Tunnels	5–9	—	0–4	—	—	—
e. Stacks/Cooling Towers	3–7	6–9	0–4	—	0–4	0–4
f. Concrete Intake Piping/Cribs	5–10	—	—	—	4–8	0–4
g. Tanks	2–6	2–6	0–4	—	0–4	0–4
h. Miscellaneous Structures (Water Management)	4–8	6–10	0–6	—	4–10	0–4

Table 4.5. (Cont'd)

b. Metallic Materials

STRUCTURAL ELEMENT	METALLIC DEGRADATION FACTORS								
	MILD REINFORCEMENT, LINER PLATE AND STRUCTURAL STEEL				PRESTRESSING REINFORCEMENT				
	Corrosion	Thermal	Irradiation	Fatigue	Corrosion	Thermal	Irradiation	Fatigue	Relaxation
1. Containment Vessel									
a. Foundation	6–10	—	—	—	0–10	0–4	—	0–4	0–4
b. Tendon Galleries	6–10	—	—	—	—	—	—	—	—
c. Vertical Walls	6–10	2–6	0–4	3–8	7–10	3–6	0–4	2–6	2–8
d. Dome	6–10	0–4	—	2–6	7–10	0–4	0–4	2–6	2–8
e. Suppression Chamber (BWR)	6–10	2–6	0–4	0–4	—	—	—	—	—
f. Polar Crane Support	6–10	2–6	0–4	2–6	—	—	—	—	—
g. Ring Girder (PWR)	6–10	0–4	—	0–4	7–10	0–4	0–4	2–6	2–8
2. Containment Internal Structures									
a. Bottom Slab	6–10	2–6	2–6	0–6	—	—	—	—	—
b. Reactor Pedestal (BWR)	4–10	4–8	4–8	4–10	—	—	—	—	—
c. Primary Reactor Shield	4–10	2–6	4–9	2–6	—	—	—	—	—
d. Floor Slabs	4–10	2–6	0–4	2–6	—	—	—	—	—
e. Walls	6–10	2–6	2–6	2–6	7–10	0–6	2–6	0–6	2–8
f. Columns	6–10	2–6	0–4	3–8	—	—	—	—	—
g. Beams	4–10	2–6	0–4	2–6	—	—	—	—	—
h. Fuel Pool Slabs/Walls	6–10	0–4	2–6	4–10	7–10	0–6	2–6	0–6	2–8
3. Secondary Containment Structure									
a. Foundations	6–10	—	—	—	—	—	—	—	—
b. Slabs	4–10	0–4	—	2–6	—	—	—	—	—
c. Walls	6–10	0–4	—	0–4	—	—	—	—	—
d. Columns	6–10	0–4	—	0–6	—	—	—	—	—
e. Beans	4–10	0–4	—	0–4	—	—	—	—	—
4. Auxiliary Structures									
a. Foundations	6–10	—	—	0–6	—	—	—	—	—
b. Walls and Columns	6–10	2–6	0–4	0–4	—	—	—	—	—
c. Slabs/Beams/Roof Slabs	4–10	0–4	0–4	0–6	—	—	—	—	—
d. Cable Ducts/Pipe Tunnels	6–10	—	—	—	—	—	—	—	—
e. Stacks/Cooling Towers	6–10	0–4	—	0–4	—	—	—	—	—
f. Concrete Intake Piping/Cribs	6–10	—	—	—	—	—	—	—	—
g. Tanks	6–10	0–4	—	—	—	—	—	—	—
h. Miscellaneous Structures (Water Management)	6–10	—	—	0–4	—	—	—	—	—

Table 4.6. Standards for use in environmental assessments.

Medium	Parameter	ASTM Candidate Test Method
Air	Acidity	D 1654, G 50, G 92
	Carbon Dioxide Content	Standard Methods
	Humidity	D 4230, E 337
	Temperature Range	Standard Methods
Soil	Corrosivity/pH	G 51
	Oxygen Content	D 888, D 4646
	Micro-organisms/Bacteria	D 4412
	Sulfide/Chloride Content	D 4542
	Resistivity	G 57
	Moisture Content	D 2216, D 3017
Groundwater	Water Table Elevation/Sampling	D 512, D1293, D 4448
	Corrosivity	D 1067, D 1293, E 70
	Hydrostatic Pressure	Standard Methods
	Dissolved Oxygen Content	D 888

Table 4.7. Classification and rating of cracks and surface damage developed by RILEM 104-DDC.[58]

a. Cracks

Type	Rating	Appearance
Diagonal	1 (very slight)	< 1 mm in width
Longitudinal	2 (slight)	1–10 mm in width
Transverse	3 (moderate)	10–20 mm in width
	4 (severe)	20–25 mm in width
	5 (very severe)	> 25 mm in width, spalling and/or faulting
Craze	1 (very slight)	barely noticeable
Pattern	2 (slight)	clearly visible-no raveling
Checking	3 (moderate)	clearly visible-some raveling
Plastic	4 (severe)	cracks raveled over substantial area
Plastic	5 (very severe)	cracks severely raveled or spalled
Corner crack	1 (very slight)	< 1 mm in width
	2 (slight)	1–10 mm in width
	3 (moderate)	10–20 mm in width
	4 (severe)	20–25 mm in width
	5 (very severe)	> 25 mm in width, spalling and/or faulting
D cracking	1 (very slight)	crack width < 1 mm, effective width < 150 mm from joint or crack
	2 (slight)	effective width < 250 mm from joint or crack, no spalling
	3 (moderate)	as above but with moderate spalling
	4 (severe)	as above but with severe spalling
	5 (very severe)	as above but with very severe spalling

Table 4.7. (Cont'd)

b. Surface Damage

Type	Rating	Appearance
Chalking		Formation of a loose powder resulting from the disentegration of a surface of concrete or of applied coating
Delamination		Separation along a plane parallel to a surface
Dusting		Development of a powdered material at the surface of hardened concrete
Exudation		Liquid or viscous gel-like material discharged through concrete surface defect
Blistering *Cavitation* *Peeling* *Exfoliation*	1 (very slight)	noticeable
	3 (moderate)	thickness of damage < 10 mm
	5 (very severe)	thickness of damage > 10 mm
Popouts	1 (very slight)	barely noticeable
	2 (slight)	noticeable
	3 (moderate	holes up to 10 mm in diameter
	4 (severe)	holes between 10 and 50 mm in diameter
	5 (very severe)	holes > 50 mm in diameter
Scaling	1 (very slight)	noticeable
	2 (slight)	loss of surface mortar without exposure of coarse aggregate
	3 (moderate)	loss of surface mortar 5 to 10 mm in depth with exposure of coarse aggregate
	4 (severe)	loss of surface mortar 10 to 20 mm in depth surrounding coarse aggregate
	5 (very severe)	loss of coarse aggregate and mortar to a depth in excess of 20 mm
Spalls	1 (very slight)	barely noticeable
	2 (slight)	clearly noticeable
	3 (moderate)	holes larger than popout of coarse aggregate
	4 (severe)	holes 150 mm in diameter and at least 150 mm deep
	5 (very severe)	holes larger than 150 mm
Loss of coarse aggregate	1 (very slight)	barely noticeable
	2 (slight)	noticeable
	3 (moderate)	pock- marked appearance
	4 (severe)	closely spaced pock-marks
	5 (very severe)	surface has raveled appearance

Table 4.8. Influence of moisture state on durability processes.

Effective relative humidity	Process*				
	Carbonation	Corrosion of steel		Frost attack	Chemical attack
		In carbonated concrete	In chloride contaminated concrete		
Very low (< 45%)	1	0	0	0	0
Low (45–65%)	3	1	1	0	0
Medium (65–85%)	2	3	3	0	0
High (85–98%)	1	2	3	2	1
Saturated (> 98%)	0	1	1	3	3

* 0 = insignificant risk; 1 = slight risk; 2 = medium risk; 3 = high risk.

Source: Comité Euro-International du Béton (CEB), *Durable Concrete Structures–Design Guide*, published by Thomas Telford Services Ltd., London, England, 1989; reprinted with permission from CEB and Thomas Telford Services Ltd.

Table 4.9. Relationship between environmental conditions and aggressivity to reinforced concrete.

Exposure Class		Environmental conditions
1		Dry environment; e.g., — interior of buildings for normal habitation or offices — exterior components not exposed to wind and weather or soil or water — localities with higher relative humidity only for a short period of the year (e.g., > 60% RH for less than 3 months per year)
2	a	Humid environment without frost;* e.g., — interior of buildings where humidity is high — exterior components exposed to wind and weather but not exposed to frost — components in non-aggressive soil and/or water not exposed to frost
	b	Humid environment with frost;* e.g., — exterior components exposed to wind and weather or non-aggressive soil and/or water and frost
3		Humid environment with frost* and de-icing agent; e.g., — exterior components exposed to wind and weather or non-aggressive soil and/or water and frost and de-icing chemicals
4	a	Seawater environment; e.g., — components in splash zone or submerged in seawater with one face exposed to air — components in saturated salt air (direct coast area)
	b	Seawater environment with frost;* e.g., — components in splash zone or submerged in seawater with one face exposed to air — Components in saturated salt air (direct coast area)
		The following classes may occur alone or in combination with the above classes
5†	a	Slightly aggressive chemical environment (gas, liquid, or solid)
	b	Moderately aggressive chemical environment (gas, liquid, or solid)
	c	Highly aggressive chemical environment (gas, liquid, or solid)

* Under moderate European conditions.

† See International Standards Organization (ISO) classification of chemically aggressive environmental conditions affecting concrete. The ISO standard is still to be established. See also Table 4.11.

Source: Comité Euro-International du Béton (CEB), *Durable Concrete Structures–Design Guide*, published by Thomas Telford Services Ltd., London, England, 1989; reprinted with permission from CEB and Thomas Telford Services Ltd.

Table 4.10. Exposure classes for steel reinforcement-related environmental conditions.

Exposure Class		Environmental conditions
1		Dry environment: generally dry localities of fairly constant humidity where the relative humidity only infrequently exceeds 70%; e.g., interiors of buildings for normal habitation or offices.
2	a	Environments with infrequent major variations in relative humidity, giving only occasional risk of condensation.
	b	Environments with frequent major variations in humidity, giving frequent risks of condensation.
3		Humid environment with frost* and de-icing agents; e.g., exterior components exposed to wind and weather or non-aggressive soil and/or water and frost and de-icing chemicals
4		Seawater environment; e.g., — components in splash zone or submerged in seawater with one face exposed to air — Components in saturated salt air (direct coast area)

* Under moderate European conditions.

Source: Comité Euro-International du Béton (CEB), *Durable Concrete Structures–Design Guide*, published by Thomas Telford Services Ltd., London, England, 1989; reprinted with permission from CEB and Thomas Telford Services Ltd.

Table 4.11. Assessment of the degree of chemical attack of concrete by waters and soils containing aggressive agents.

	Exposure class* 5a	Exposure class* 5b	Exposure class* 5c	
Type of attack	**Weak Attack**	**Moderate Attack**	**Strong Attack**	**Very Strong Attack**
Water				
pH value	6.5–5.5	5.5–4.5	4.5–4.0	< 4.0
Agressive CO_2: mg CO_2/ℓ	15–30	30–60	60–100	> 100
Ammonium: mg NH_4^+/ℓ	15–30	30–60	60–100	> 100
Magnesium: mg Mg^{2+}/ℓ	100–300	300–1500	1500–3000	> 3000
Sulphate: mg SO_4^{2-}/ℓ	200–600	600–3000	3000–6000	> 6000
Soil				
Degree of acidity according to Baumann–Gully	> 20	X†	X†	X†
Sulphate: mg SO_4^{2-}/kg of air-dry soil	2000–6000	6000–12000	12000	X†

* See Table 4.9

† X = conditions of attack that are not found in practice.

Source: Comité Euro-International du Béton (CEB), *Durable Concrete Structures–Design Guide*, published by Thomas Telford Services Ltd., London, England, 1989; reprinted with permission from CEB and Thomas Telford Services Ltd.

Table 4.12. Results of application of structural aging assessment methodology: PWR prestressed large-dry plant*

Primary Structure	Cumulative Rank	Function
Containment Vessel	171	The containment vessels provide structural support and radiation shielding and containment for safe operation of the PWR. This containment is of sufficient volume to support pressure requirements of a pipe break accident. The vessel supports all internal equipment and transfers loading into the underlying soil. These structures were all cast-in-place.

Subelement	Importance	Environment Exposure	Safety Significance	Subelement Rank
Dome	8	6	10	178

Composition	Key Degradation Factors	Rating
Prestressing Reinforcement, Deformed Reinforcement, Portland Cement Concrete, Liner Plate, Structural Steel	Chemical Attack	6
	Freeze/Thaw	5
	Thermal Exposure	3
	Fatigue	3
	Reinforcing Corrosion	9
	Reinforcing Therm. Exposure	3
	Prestressing Corrosion	10
	Prestressing Fatigue	2
	Prestressing Relaxation	7

Subelement	Importance	Environment Exposure	Safety Significance	Subelement Rank
Vertical Walls (including buttresses)	8	7	10	178

Composition	Key Degradation Factors	Rating
Prestressing Reinforcement, Deformed Reinforcement, Portland Cement Concrete, Liner Plate, Structural Steel, Waterproofing	Chemical Attack	8
	Freeze/Thaw	4
	Thermal Exposure	5
	Fatigue	4
	Reinforcing/Liner Corrosion	10
	Reinforcing Therm. Exposure	4
	Prestressing Corrosion	10
	Prestressing Fatigue	2
	Prestressing Relaxation	8
	Abrasion	3

* Results are presented only for the post-tensioned concrete containment. Results for other safety-related reinforced concrete structures are provided in Ref. 259.

Table 4.12. (Cont'd)

Subelement	Importance	Environment Exposure	Safety Significance	Subelement Rank
Mat Foundation	10	10	10	200

Composition

Deformed Reinforcement, Portland Cement Concrete, Waterproofing, (prestressing reinforcement — local)

Key Degradation Factors	Rating
Chemical Attack	10
Thermal Exposure	4
Irradiation	3
Reinforcing Corrosion	10
Prestressing Corrosion	6
Prestressing Relaxation	4

Subelement	Importance	Environment Exposure	Safety Significance	Subelement Rank
Ring Girder	9	6	10	182

Composition

Prestressing Reinforcement (anchorages), Deformed Reinforcement, Portland Cement Concrete

Key Degradation Factors	Rating
Chemical Attack	5
Freeze/Thaw	4
Fatigue	3
Reinforcing Corrosion	9
Prestressing Corrosion	10
Prestressing Relaxation	7

Subelement	Importance	Environment Exposure	Safety Significance	Subelement Rank
Tendon Gallery	3	10	4	118

Composition

Deformed Reinforcement, Portland Cement Concrete, Waterproofing, Structural Steel

Key Degradation Factors	Rating
Chemical Attack	10
Reinforcing Corrosion	10

Primary Structure	Cumulative Rank	Function
Containment-Internal Structures	144	Provide structural support and radiation shielding for NSSS equipment for all operating conditions. Provides human access to primary components and support for refueling operations. These structures are massive in section and were cast-in-place.

•

•

•

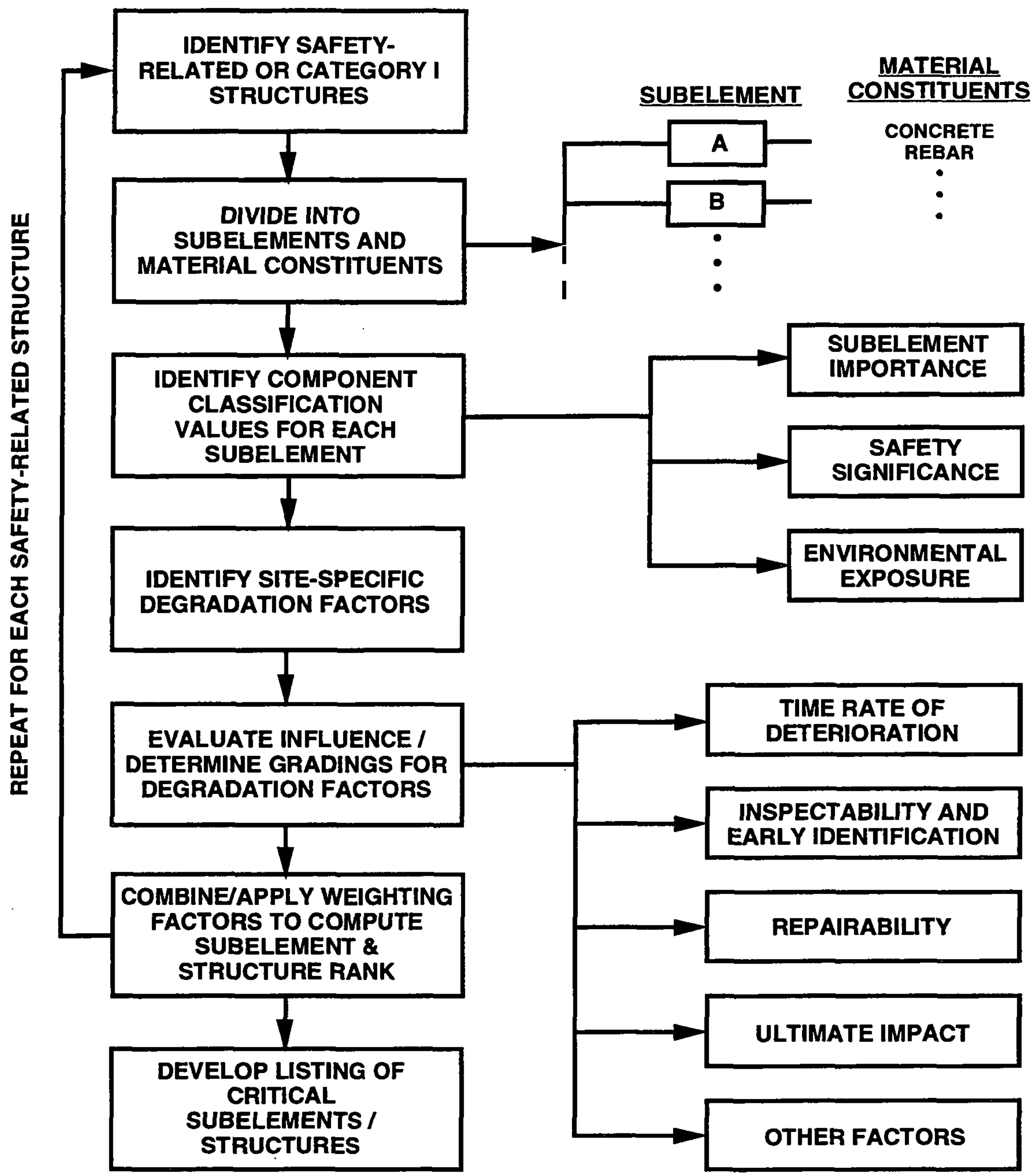


Fig. 4.1. Structural aging assessment methodology flow diagram.

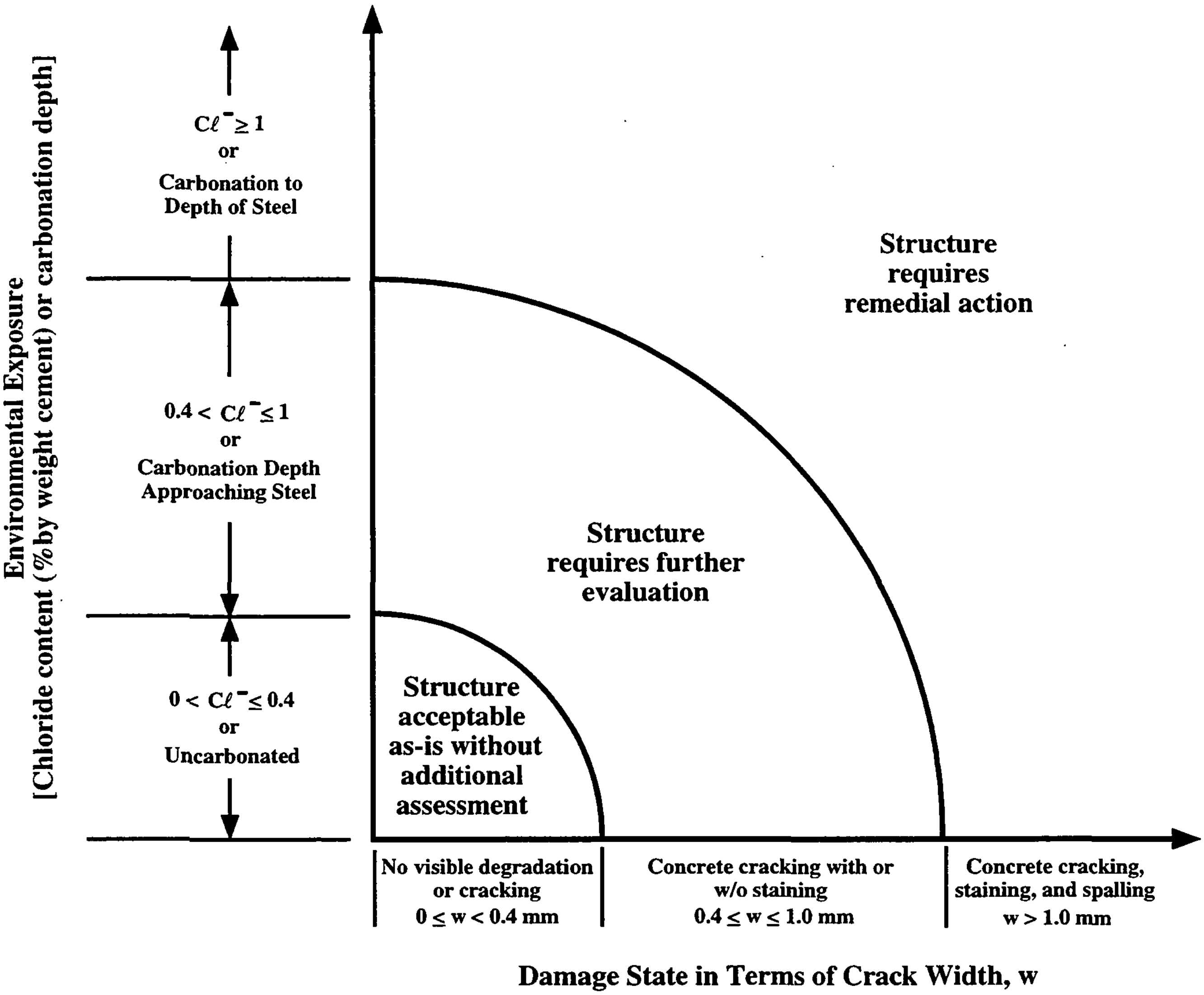


Fig. 4.2 Damage state chart relating environmental exposure, crack width, and necessity for additional evaluation or repair.

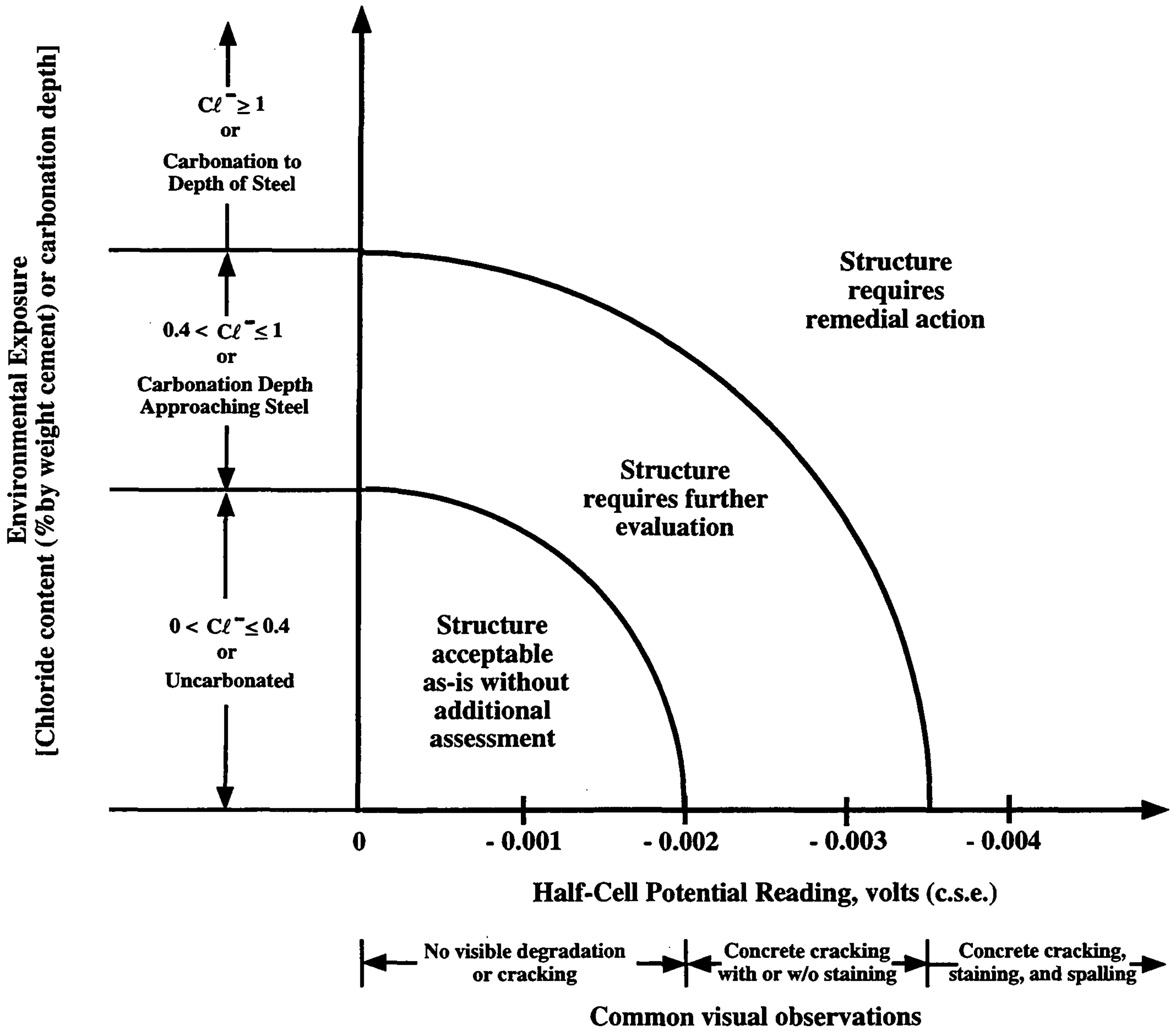


Fig. 4.3 Damage state chart relating environmental exposure, half-cell potential reading, and necessity for additional evaluation or repair

5. RELIABILITY-BASED METHODOLOGY FOR CURRENT AND FUTURE CONDITION ASSESSMENTS

Numerous uncertainties are encountered in trying to assess the current condition and likely future performance of nuclear power plant (NPP) reinforced concrete structures. Such uncertainties arise from the randomness in the initial material properties, operating and environmental loads, material changes and possibly degradation due to aging, mechanistic models used to predict time-dependent structural behavior, and many other factors. Structural reliability methods model the uncertainties due to the above sources with probability distributions and provide a mechanism for dealing with the numerous sources of uncertainty consistently and explicitly. Such methods have been used to develop a first-generation of limit state design codes, in which the safety checks are associated with specific limit state probability or reliability.[275–277] This previous work took into account the stochastic nature of resistance and loads. However, it did not consider the effect of time-dependent changes in the structural properties on reliability of components and systems.

In recent years, however, structural reliability techniques have advanced to the point where it is possible to make a quantitative evaluation of the remaining service life of a reinforced concrete structure based on knowledge of the condition of the structure when built, its service history, its present condition, and projected use during a defined continued service period. The reliability-based methodologies accomplish this through integration of information on design requirements, material and structural degradation and damage accumulation, environmental factors, and nondestructive testing technology into a decision tool that provides a quantitative measure of structural reliability under projected future service conditions.[278–281] Time-dependent reliability analysis methods provide a framework for performing condition assessments of existing structures and for determining whether in-service inspection (ISI) and repair are required to maintain reliability and performance at the desired regulatory level. These methods can also be utilized to identify optimum ISI/repair strategies that minimize cost while ensuring that the structure maintains a sufficient level of reliability for public safety during a prescribed service life.

5.1 RELIABILITY-BASED METHODS FOR CONDITION ASSESSMENT

The performance of a structure in service is assessed by comparing its state of behavior to one of several limit states. When a structure or structural component becomes unfit for its intended purpose, it is said to have reached a limit state. In the reliability-based condition assessment methodology developed subsequently, structural performance in the presence of uncertainty is measured in terms of a limit state probability. Ultimate limit states relate to safety under extreme conditions. A concrete structure is considered to have failed when a load effect (e.g., axial force, moment, shear or some combination of these effects) to which it is subjected exceeds its available capacity at that time. Such an event should have a low probability of occurrence. Serviceability limit states relate to disruption of function under conditions of normal usage. Excessive deflections or crack widths under normal service loads are examples of serviceability limit states in concrete structures.

Aging and durability issues introduce the time factor into limit states design. One cannot deal with durability issues rationally without introducing the notion of design or service life.[282] The limit state probability must be determined with respect to a service life in order to be useful as a decision variable. The service life is defined as that period during which the structure is able to withstand all loads safely. A design service life (or maintenance interval) is that period during which the probability of the structure performing its intended functions is acceptable.

5.1.1 Degradation Mechanisms

Time-dependent reliability analysis and service life predictions for reinforced concrete structures require time-dependent stochastic models of the structural strength. In concept, stochastic strength models can be derived from: (1) mathematical models describing the effects of the aging process resulting from service and environmental factors on steel and concrete materials and component geometry; (2) accelerated life testing; or (3) a combination of the two. At the current state of the art, these effects are often known qualitatively; however, quantitative models that describe material degradation processes often are empirical in nature.[113] The service life determinations often require that these models be extrapolated outside the range of experimental data. Primary degradation mechanisms that can impact the NPP reinforced concrete structures were described in Sect. 2.3.2 along with information on models for service life estimations.

5.1.2 Statistical Data on Loads and Resistance

The condition assessment and time-dependent reliability analysis require probabilistic descriptions of loads and structural strengths. Some of the statistical data needed to support this methodology have been collected as part of previous research to develop probability-based design requirements for ordinary building construction and for NPP structures.[275–277] Load and resistance models and data have been analyzed in detail[283] and are summarized below.

5.1.2.1 Structural loads

The occurrence and variation in intensity of a structural load, $X(t)$, in space and in time is modeled as a stochastic process. It is convenient to classify these loads in a general sense according to the way in which they vary in time, as illustrated by the typical sample functions in Fig. 5.1. Two sets of probability density functions (pdf) are also shown in Fig. 5.1; $f_X(x)$ describes the intensity of the load at any time, while $f_Z(x)$ describes the maximum load within the interval of time $(0, t)$. Permanent loads [Fig. 5.1(a)] are random in intensity but do not vary significantly in time. The probability density function of the load at any point in time, $f_X(x)$, is sufficient to describe the statistical characteristics of the load process, and is identical to $f_Z(x)$. Loads with significant variation in time may act essentially continuously [Fig. 5.1(b)] or intermittently [Fig. 5.1(c)]. The probability density function at any point in time is insufficient to describe these load processes statistically or to determine $f_Z(x)$; information concerning the time-variation of the load process is required as well.

A relatively simple model of the overall temporal variation in structural loads is provided by assuming that the changes in the load magnitude occur in time according to a Poisson process. Under this assumption, the number of load changes, $N(t)$, in the interval $(0, t)$ is described by the probability function,

$$P\left[N(t)=n\right]=\frac{(\lambda t)^n \cdot \exp(-\lambda t)}{n!}; n=0,\ 1,\ 2,\ \ldots \tag{5.1}$$

in which $P[\bullet]$ = probability of the event in the brackets and the parameter λ = mean rate of occurrence of the events. The duration of one load event, T_i, also is a random variable. If the events occur according to a Poisson process with parameter λ and the load is a continuous variable load [Fig. 5.1(b)], the duration has an exponential distribution, with mean value $E[T_i] = \tau = 1/\lambda$. If the load is intermittent $X(t) = 0$ for significant periods of time [Fig. 5.1(c)], the mean occurrence rate is $\lambda = p/\tau$, in which p = probability that the load is nonzero at any point in time.

If the load intensity varies negligibly or slowly during each load event, its effect on a structure is essentially static. For reliability analysis purposes, then, the load intensity can be treated as constant during any event. The actual time-varying load thus is replaced with the sequence of load pulses illustrated by the dashed lines in Figs. 5.1(b) and 5.1(c). It is customary to assume that the pulse magnitudes are identically distributed and statistically independent random variables, each with cumulative distribution function (cdf), $F_X(x)$. Such a load process is referred to as a Poisson pulse process.[284,285] The parameters λ, τ, and p and F(x) provide a complete description of the micro-variation of the load in time.

It is assumed that significant structural loads can be modeled as a sequence of such load pulses. 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. Load statistics used in the subsequent time-dependent reliability analysis are summarized in Table 5.1. These statistics are believed to be reasonable and conservative. Only limited data on NPP loads are available; more detailed descriptions are given elsewhere.[283]

5.1.2.2 Structural resistance

The strength, R, of a reinforced concrete component is described by[286]

$$R = B \cdot R_m(X_1, X_2, \ldots, X_m), \tag{5.2}$$

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.[287] 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 of R directly. Typical statistical data on material strengths and dimensions of reinforced concrete structural elements are summarized in Table 5.2.[283,286,287] The material strengths presented are based on "static" rates of load. Long-term strength changes in the concrete or steel due to maturity of concrete, environmental stressors, and possible corrosion of reinforcement are not reflected in these statistics. When data on the strength of concrete or steel can be obtained by limited in situ sampling as part of a condition assessment of a specific structure, that data should be utilized.

Typical means and coefficients of variation (cov) in strength for reinforced concrete structural elements are summarized in Table 5.3. These estimates were obtained by Monte Carlo simulation using the strength and dimensional variabilities in Table 5.2. The modeling uncertainty was described by a normal distribution, with a mean $\mu_B = 1.05$ and c.o.v. $V_B = 0.06$ for flexure and compression and with $\mu_B = 1.15$ and $V_B = 0.15$ for shear. In all cases, the nominal flexural strength for concrete structural members, M_u, the nominal shear strength, V_u, and the nominal axial strength, R_u, were computed according to American Concrete Institute (ACI) Standards 318[16] and 349.[17]

5.1.3 Time-Dependent Reliability Analysis

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), both are random functions of time. At any time, t, the margin of safety, M(t), is

$$M(t) = R(t) - S(t). \tag{5.3}$$

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

$$P_f(t) = P[M(t) < 0] = \int_0^\infty F_R(x)\, f_S(x)\, dx. \tag{5.4}$$

in which $F_R(x)$ and $f_S(x)$ are the cumulative distribution function of R and probability density function (pdf) of S. Equation (5.4) provides an instantaneous quantitative measure of structural reliability, provided that $P_f(t)$ can be estimated and/or validated.[288]

For service life prediction and reliability assessment, one is more interested in the probability of satisfactory performance over some period of time, say (0,t), than in the snapshot of the reliability of the structure at a particular time provided by Eq. (5.4). 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, ..., S_n occur at times t_1, t_2, ..., t_n during (0,t), the reliability function becomes,

$$L(0,t) = P[R(t_1) > S_1, \ldots, R(t_n) > S_n]. \tag{5.5}$$

If the load process is continuous rather than discrete, an analogous expression derived from an upcrossing analysis[289] can be used to obtain an approximation to Eq. (5.5).

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

$$h(t) = -d \ln L(0,t)/dt. \tag{5.6}$$

Solving for L(0,t) yields,

$$L(0,t) = \exp\left[-\int_0^t h(x)dx\right]. \tag{5.7}$$

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,

$$P\left[T_f \leq t + \Delta t \middle| T_f > t\right] = 1 - \exp\left[-\int_t^{t+\Delta t} h(x)dx\right]. \tag{5.8}$$

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

$$L(0,t) = \prod_t L(t_{i-1}, t_i) \exp\left(\int_{t_i}^{t} h(x)dx\right), \tag{5.9}$$

in which $t_{i-1} = 0$ when $i = 1$. Note that failures in successive intervals, (t_{i-1}, t_i) generally are not statistically independent events.

The hazard function for pure chance failures is constant with time. When structural aging occurs and strength deteriorates, h(t) characteristically increases with time. ISI and repair impact the reliability analysis through the hazard function, causing it to change discontinuously at the time an inspection/repair operation is performed. Undegrading and degrading structural components can be distinguished in a time-dependent reliability analysis 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.

With the assumption that loads can be described by a Poisson pulse process, the reliability function becomes

$$L(0,t) = \int_0^{\infty} \exp\left[-\lambda t\left(1 - t^{-1}\int_0^t F_s[rg(x)dx]f_R(r)dr\right)\right], \tag{5.10}$$

in which $f_R(r)$ is the pdf 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. 5.2). The limit state probability, or probability of failure during (0,t), is F(t) = 1 - L(0,t); F(t) is not the same as $P_f(t)$ in Eq. (5.4).

5.1.4 Illustrations of Service Life Prediction

5.1.4.1 Reinforced concrete slab

Time-dependent reliability concepts are illustrated with a simple conceptual example of a concrete slab drawn from recent research on aging of reinforced concrete structures.[279] The slab was designed using the requirements for flexure strength found in ACI Standard 318[16]

$$0.9\ R_n = 1.4\ D_n + 1.7\ L_n, \tag{5.11}$$

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. 5.2. Two other situations are also illustrated in Fig. 5.2, one in which the strength degrades linearly to 90% of initial strength at 40 years and one in which the strength remains constant with time. In general, the behavior of resistance over time must be obtained from mathematical models describing the degradation mechanism(s) present (see Sect. 2.3.2). The statistics used in this example are contained in Tables 5.1–5.3.

Figure 5.3 compares the limit state probabilities [$F(t) = 1 - L(t)$ in Eq. (5.10)] obtained for the three degradation models considered in Fig. 5.2 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. Neglecting strength degradation entirely in a time-dependent reliability assessment can be quite unconservative, depending on the time-dependent characteristics of strength.

5.1.4.2 Reinforced concrete shear wall

As shown by the previous example, the failure probability of a structural component 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. It was assumed that strength degradation at any section was 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 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 effects of sulfate attack occurring 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.[290] Some results are summarized in the following.

The wall considered is a low-rise wall with a height-to-width ratio equal to unity. It is subjected to vertical load, D, that is uniformly distributed on the top of the wall, and in-plane lateral load, V, that is concentrated at the top of the wall. The shear strength of concrete walls can be estimated from empirical models.[16,291] 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[292–294] determines the ultimate shear strength as the sum of the forces sustained by a truss mechanism, V_t, and by an arch mechanisms, 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,[295] 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.[290] Figure 5.4 shows that the strength predicted by this method compares well to experimental tests of low-rise shear walls.[296,297]

Wall in Shear A wall subjected to sulfate attack or attack by strong acids can suffer 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

$$G(t) = \frac{V_t + V_a(t)}{V_{u0}} = \frac{V_t + G_a(t)\, V_a(0)}{V_{u0}}, \tag{5.12}$$

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.

Wall in Flexure and Compression The ultimate flexure capacity of a cross section is expressed as

$$M_u = T_s\left(\frac{b}{2} - d_c\right) + C_c\left(\frac{b}{2} - k_2 c_u\right) + C_s\left(\frac{b}{2} - d_c\right) \tag{5.13}$$

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_2c_u locates the compressive resultant C_c.

For illustration, assume that

- The wall is subjected to time-invariant dead load, D, that is uniformly distributed on the wall, and intermittent lateral load V, that 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[17]

  $$0.9R_n = E_{ss} \tag{5.14}$$

  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 earthquake load and shear strength are shown in Tables 5.1 and 5.3, respectively. It is assumed that $E_{ss} \approx 3D = 3.21$MN.

- The mean initiation rate of local damage per unit surface area, ν_u, due to chemical attack is time invariant and is $0.1/m^2$/year.

- The defect intensity is modeled as,

  $$Y(t) = C(t - T_I)^2 \tag{5.15}$$

  in which C is a time-invariant random variable described by a lognormal distribution with mean value, m_C, of 2.22×10^{-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.[287] 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 (units of MPa) at time t is estimated based on information provided in Ref. 134 according to*

$$f_c(t) = \begin{cases} 15.51 + 3.95 \ln t & t < 10 \text{ years} \\ 47.91 & t \geq 10 \text{ years} \end{cases}, \tag{5.16}$$

  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 chemical attack is occurring is illustrated in Fig. 5.5, 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. 5.5 are presented in Figs. 5.6 and 5.7, respectively. The increase in failure probability due to the strength degradation is small because of the large variability in earthquake load intensity.[298] 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. 5.8. 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.

5.2 ROLE OF INSPECTION/REPAIR IN MAINTAINING RELIABILITY

Forecasts of time-dependent reliability (e.g., Fig. 5.3) enable the analyst to determine the time period beyond which the desired reliability of the structure can not be assured. At such time, the structure should be inspected and its condition evaluated. Intervals of inspection and repair that may be required as a condition for continued operation can be determined from the time-dependent reliability analysis. Figure 5.9 presents a conceptual illustration of the effect of inspection and repair on the failure rate. The effect of the inspection/repair is to remove larger defects from the structure and upgrade its strength, thus reducing its conditional failure rate. As the structure ages, the failure rate increases until another inspection/repair operation occurs. Such inspection/repair strategies should be designed so that the integrated effect of h(t) in Fig. 5.9 remains at or below an established target value during its required service period. ISI and repair are a routine part of managing aging and deterioration in many engineered facilities;[299] work is already underway to develop reliability-based policies for offshore platforms,[300] and aircraft.[301]

---

* This expression should not be used "a priori" to estimate concrete strength as a function of time for reasons cited in Chapter 2.

5.2.1 Degradation Function Based on Individual Damage Intensities

The damage intensity is modeled 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. Examples of this state variable describing damage would include the ratio of area of reinforcement lost due to corrosion to the original area, the ratio of strength loss of concrete due to sulfate attack to the strength of undegraded concrete, and the ratio of the depth of crack in a member to the member thickness. 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 $(t, t + \Delta t)$ $(t > 0)$ is $\int_t^{t+\Delta t} \nu(\tau) dr$. The damage initiation rate $\nu(t)$ 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

$$X_j(t) = \begin{cases} 0 & ; 0 \leq t < T_{I_j} \\ C_j\left(t - T_{I_j}\right)^{\alpha} & ; t \geq T_{I_j} \end{cases}, \tag{5.17}$$

in which $X_j(t)$ $j = 1, 2, \ldots$ is the intensity of damage at time t, T_{I_j} $j = 1, 2, \ldots$ are the random initiation times of damage, C_j's are damage growth rates that are identically distributed and statistically independent random variables described by a cdf $F_C(c)$, and α is an deterministic parameter. The assumption of independent C_j's provides a conservative estimate of failure probability. Parameters C and α depend on the degradation mechanism (e.g., Refs. 113 and 119). If damage grows linearly, $\alpha = 1$.

(d) The degradation function $G(t)$ for a component is defined in terms of damage intensities as

$$G(t) = 1 - \max_{all\ j}\{X_j(t)\}. \tag{5.18}$$

Consider damages that initiate within interval (t_I, t). Assume first that the number of these damages, $N_I(t_I,t)$, is n. From assumption (c), the cdf of $X_j(t)$, $F_{X_j}(x; t_I, t)$, is expressed as

$$F_{X_j}\left(x; t_I, t\right) = P\left[C_j\left(t - T_{I_j}\right)^{a} < x\right], \tag{5.19}$$

$$= \int_{t_I}^{t} F_C\left(\frac{x}{(t-\tau)^{\alpha}}\right) f_{T_{I_j}}(\tau) d\tau \tag{5.20}$$

where $f_{T_{Ij}}(\tau)$ is the pdf of T_{I_j}. Given $N_I(t_I, t) = n$, the rank-ordered initiation times, T_{I_1}, T_{I_2}, ..., T_{I_m} are n order statistics of random variables W_{I_1}, W_{I_2}, ..., W_{I_n} that are statistically independent and identically distributed with pdf expressed as[302]

$$f_{W_I}(w) = \begin{cases} \dfrac{\nu(w)}{\int_{t_I}^{t} \nu(\tau)d\tau} & ; t_I \le w \le t \\ 0 & ; \text{Otherwise} \end{cases} . \tag{5.21}$$

Therefore, the cdf of the intensity of an arbitrary damage, $X(t)$, that initiates within (t_I, t) is,

$$F_X(x; t_I, t) = \int_{t_I}^{t} F_C\left(\frac{x}{(t-\tau)^{\alpha}}\right) f_{W_I}(\tau)d\tau. \tag{5.22}$$

Note that $f_{W_I}(w)$ is an unconditional pdf. Since the C_j's and W_{Ij}'s are statistically independent of one another, the $X_j(t)$'s are also statistically independent. Accordingly, the cdf of $X_{max}(t_I, t) = \max\{X_J(t)$ that initiated within $(t_I, t]\}$ is,

$$F_{X_{max}}\left(x; t_I, t \middle| N_I(t_I, t) = n\right) = \left[F_X(x; t_I, t)\right]^n. \tag{5.23}$$

Removing the condition that $N_I(t_I, t) = n$,

$$\begin{aligned} F_{X_{max}}(x; t_I, t) &= \sum_{n=0}^{\infty} \left[F_X(x; t_I, t)\right]^n \cdot \frac{\left(\int_{t_I}^{t} \nu(\tau)d\tau\right)^n \cdot \exp\left(-\int_{t_I}^{t} \nu(\tau)d\tau\right)}{n!} \\ &= \exp\left[-\int_{t_I}^{t} \nu(\tau)d\tau\{1 - F_X(x; t_I, t)\}\right]. \end{aligned} \tag{5.24}$$

From assumption (d), the mean and variance of the degradation function are evaluated by

$$\begin{aligned} E[G(t)] &= E[1 - X_{max}(0,t)] \\ &= 1 - \int_0^1 \left[1 - F_{X_{max}}(x; 0, t)\right]dx, \end{aligned} \tag{5.25}$$

and

$$\begin{aligned} Var[G(t)] &= Var[X_{max}(0;t)] \\ &= \int_0^1 2x\left[1 - F_{X_{max}}(x; 0, t)\right]dx - \{E[G(t)]\}^2. \end{aligned} \tag{5.26}$$

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 at g(t).[303]

5.2.2 Degradation Function After Repair

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 5.10 illustrates conceptually the probability, $d(x)$, of detecting a defect of size x. Such a relation exists, at least conceptually, for each ISI technology. The strength following repair depends on both the detectability and repair strategy adopted.

5.2.2.1 Full inspection/repair

Assume that during inspection/repair 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/repair 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.

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

$$d(x) = \begin{cases} 0 \quad ; \quad 0 \le x < x_{th} \\ \qquad\qquad\qquad\qquad , \\ 1 \quad ; \quad x_{th} \le x \le 1 \end{cases} \tag{5.27}$$

where x_{th} is the minimum detectable value of damage (see Fig. 5.10). The same detection threshold values are assumed for all inspections. Following m inspections at $t_R = \left\{t_{R_1}, ..., t_{R_m}\right\}$, 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 that exist at time t and initiated 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 $p \cdot v(w)$ where,

$$p = P\left[\text{A damage is not repaired by } t_{R_m}\right] = F_X\left(x_{th}, 0, t_{R_m}\right), \tag{5.28}$$

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

$$v''(w) = \begin{cases} p \cdot v(w) & ; 0 < w \le t_{R_m} \\ v(w) & ; t_{R_m} < w \le t \end{cases} . \tag{5.29}$$

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, by replacing $v(w)$ by $v''(w)$, and $F_X(x;\, 0,\, t)$ by the updated cdf $F''_X\left(x;\, t_{R_m},\, t\right)$.

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

$$F''_X\left(x;\, t_{R_m},\, t\right) = F_{X_{(A)}}\left(x;\, t_{R_m},\, t\right) \cdot \mathrm{P(A)} + F_{X_{(B)}}\left(x;\, t_{R_m},\, t\right) \cdot \mathrm{P(B)}, \tag{5.30}$$

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 group (A), $X_{(A)}(t)$, and intensity of damages in group (B), $X_{(B)}(t)$, respectively, and P(A) and P(B) are the probabilities that a defect belongs to group (A) or (B),

$$P(A) = P\left[W_1 \le t_{R_m}\right] = \int_0^{t_{R_m}} f_{W_1}(w) dw, \tag{5.31}$$

$$P(B) = 1 - P(A), \tag{5.32}$$

in which $f_{W_1}(w)$ is evaluated by Eq. (5.21) replacing $v(w)$ with $v''(w)$. $F_{X_{(A)}}\left(x;\, t_{R_m},\, t\right)$ is expressed as,

$$F_{X_{(A)}}\left(x;\, t_{R_m},\, t\right) = P\left[X(t) < x \middle| X\left(t_{R_m}\right) < x_{th}\right] \tag{5.33}$$

$$= \frac{\int_0^{t_{R_m}} F_C\left(\min\left\{\frac{x}{(t-\tau)^\alpha}, \frac{x_{th}}{\left(t_{R_m} - \tau\right)^\alpha}\right\}\right) f_{W_1}(\tau) d\tau}{\int_0^{t_{R_m}} F_C\left(\frac{x_{th}}{\left(t_{R_m} - \tau\right)^\alpha}\right) f_{W_1}(\tau) d\tau}. \tag{5.33'}$$

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

$$F_{X_{max(B)}}(x;\, t) = F_X\left(x;\, t_{R_m} \cdot \mathrm{t}\right). \tag{5.34}$$

In general, the detectability function, *d(x)*, is not a step function but rather a non-decreasing function of damage intensity (see Fig. 5.10). Procedures for dealing with this more general detectability function and for partial inspections have been developed and described elsewhere.[303]

Examples Illustrating Effect of Inspection/Repair Operations

Simple conceptual examples are provided below to illustrate the effect of single inspection/repair approaches. The examples consider a beam in flexure designed according to Eq. (5.11) with the load and resistance statistics given in Tables 5.1 and 5.3, respectively.

Single Inspection/Repair Consider the following

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 $\nu = 5/\mathrm{yr}$ that is dependent on the surface area or volume of the structure.

c. Damage grows linearly as a function of time as described by Eq. (5.17) with $\alpha = 1$.

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, $V_c = 0.5$.

The effect on the mean degradation function of inspection/repair described by several detectability functions is illustrated in Fig. 5.11. 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 (that would result in lower reliability); however, the effect of the general shape of *d(x)* (Fig. 5.10) 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)*.

Multiple Inspection/Repair The effect of multiple inspection/repair and the mean degradation function is illustrated in Fig. 5.12, 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.

5.2.2.2 Partial inspection/repair

In many instances, only a small part of the structure or component is inspected to assess its condition because of cost or accessibility constraints. Inherent in this procedure is the assumption that any part of the component is representative of the component as a whole, and the rest of the component is amenable to a statistical representation.[304] At each time when a partial inspection is carried out, we gain some information about the strength and degradation of the component and make a decision whether or not to repair the component, depending on the result of the inspection. Assume that each inspection is carried out using an NDE method with detectability described by a step function. A decision policy at each inspection can be established as follows:

1. Inspect a% of the component (stage I).
2. If the maximum intensity of damages detected at stage I, $X_{maxI}(t_R)$ is less than the predetermined critical value, x_{cr}, then no further inspection is performed until the next scheduled inspection.
3. If $X_{maxI}(t_R) \geq x_{cr}$, then inspect another a% of the component (stage II).
4. If the maximum intensity of damages detected at stage II, $X_{maxII}(t_R)$, is less than x_{cr}, then no further inspection is performed until the next scheduled inspection.
5. If $X_{maxII}(t_R) \geq x_{cr}$, then inspect the entire component and repair all detected damages (state III).
6. If a stage III inspection is carried out at t_{R_j}, then during the rest of its service life the component will be fully inspected and all detected damages will be repaired.

According to this policy, the parameters of the degradation model are updated after each partial inspection (stage I and stage II), but the component is not repaired. Since the portion of a component partially inspected is generally small (10% or less), the impact of not repairing the detected damages on the degradation function and on the failure probability of the component is assumed to be small.

A series of partial inspections using this policy leads to a decision tree as shown in Fig. 5.13. Although other policies can be envisioned (e.g., Ref. 305), this policy is consistent, in a general sense, with acceptance criteria currently in use for weldments and certain other critical structures. The total number of branches in the decision tree after the mth inspection is $2^m + 2^{m-1} + 1$.

Besides the assumptions (a)–(d) made in Sect. 5.2.2.1, assume that the mean damage initiation rate, ν, and the mean of the damage growth rate, $\overline{C}$, are random variables jointly distributed with pdf, $f_{\nu\overline{C}}(\xi, \eta)$. Furthermore, assume that the cdf of the maximum intensity of damages existing at time t without repair can be evaluated approximately by Eqs. (5.21, 5.22, and 5.24) by substituting $\nu(\tau) = \mu_\nu$ and $\overline{C} = \mu_C$, where μ_ν and μ_C are the means of μ and $\overline{C}$, respectively.

After each inspection μ_V and μ_C are updated depending on the decision made at the inspection (i.e., the path to be taken in Fig. 5.13). Based on Bayes theorem,[306] the posterior joint density function of v and $\overline{C}$, $f''_{v\overline{C}|I}(\xi, \eta)$, is,

$$f''_{v\overline{C}|I}(\xi, \eta) = kL(\xi, \eta|I) f'_{v\overline{C}}(\xi, \eta), \tag{5.35}$$

in which k is a normalizing constant, $L(\xi, \eta|I)$ is the likelihood function of taking a path given $v = \xi$ and $\overline{C} = \eta$, and $f'_{v\overline{C}}(\xi, \eta)$ is the prior joint density of v and $\overline{C}$. Then the parameters of the degradation model are updated as,

$$\mu''_V = \int_0^\infty \int_0^\infty \xi f''_{v\overline{C}|I}(\xi, \eta) d\eta d\xi, \text{ and} \tag{5.36}$$

$$\mu''_C = \int_0^\infty \int_0^\infty \eta f''_{v\overline{C}|I}(\xi, \eta) d\eta d\xi. \tag{5.37}$$

At an inspection, one of three decisions is made. The probabilities of making each decision are,

$$P[\text{Stage I}] = F_{Xmax}(x_{cr}; 0, t_R) \tag{5.38}$$

$$P[\text{Stage II}] = [1 - F_{Xmax}(x_{cr}; 0, t_R)] F_{Xmax}(x_{cr}; 0, t_R) \tag{5.39}$$

$$P[\text{Stage III}] = [1 - F_{Xmax}(x_{cr}; 0, t_R)]^2 \tag{5.40}$$

in which $F_{Xmax}(x;\ 0,\ t)$ is evaluated by Eq. (5.24) substituting $v = a\mu''_V/100$ and $\mu_C = \mu''_C$. Therefore, the likelihood function of taking any path is,

$$\begin{aligned} L(\xi, \eta|I) = & \prod_i P\left[\text{Stage I at } t_{R_i} | v = \xi, \overline{C} = \eta\right] \\ & \cdot \prod_j P\left[\text{Stage II at } t_{R_i} | v = \xi, \overline{C} = \eta\right] \\ & \cdot P\left[\text{Result of the } k^{th} \text{ inspection} | v = \xi, \overline{C} = \eta\right]. \end{aligned} \tag{5.41}$$

Example Illustrating Partial Inspection/Repair Operation

The effect of partial inspection/repair on the failure probability of a component is illustrated in Fig. 5.14. The component is designed for flexure by Eq. (5.11). The statistics of the initial resistance and of the loads are the same as used in the previous example. Partial inspections are scheduled at 20, 30, 40, and 50 years with 5% of the component inspected at each partial inspection. As an additional constraint, if a Stage III inspection is carried out at 20 years, the component also is fully inspected/repaired at 40 years. However, if a Stage III inspection is carried out at 30 years or later, the component is not inspected/repaired during the rest of its service life. The critical value for decision at partial inspection, x_{cr}, and the threshold value of damage detection for full inspection are the same at all inspections and equal 0.03 and 0.05. For purpose of comparison, the results of the full inspection/repair strategy (multiple) are also included in the figure. These partial inspection/repair strategies are compatible with full inspection/repair strategies in terms of failure probability for a lifetime of 60 years. Thus, the choice of a strategy should be based on total cost associated with the particular strategy.

5.3 OPTIMUM INSPECTION/REPAIR STRATEGIES

Costs of inspection and repair may be a significant part of the overall lifetime cost of a concrete structure. These costs should be balanced by the benefits to be gained, both in economic and reliability terms. There are tradeoffs between the extent and accuracy of inspection, required level of reliability, and cost. One can perform this tradeoff by defining an objective function that takes into account the cost of failure, cost of inspection, and costs of repair. The failure probability of a component during its service life, $F(t)$, provides one of several constraints on the optimization.

To design an optimum inspection/repair program, the following optimization problem must be solved:

$$\begin{aligned} &\text{Minimize} && C_T \\ &\text{Subject to} && F(t_L) \le P_{f_T}, \end{aligned} \tag{5.42}$$

in which P_{f_T} is an established target failure probability during the service life, t_L, of a component/structure and C_T is the total cost of inspection/repair plus expected losses due to failure of a structure. Other possible constraints might include the minimum and maximum time intervals between inspections, and the minimum threshold value of detection of the NDE method that is available.

Some studies have been done to determine optimal inspection/repair strategies for metallic structures subjected to fatigue, assuming that a component is replaced if the intensity of detected damage exceeds a critical value.[305] The cost of repair (i.e., cost of replacement) was assumed to be constant in these studies. However, a component may not be replaced; instead, only detected damages might be repaired. In this case, the cost of repair would be a function of damage intensities to be repaired. This aspect also should be considered in designing an optimum strategy.

Optimum inspection/repair strategies are illustrated using several parametric representations of relative cost of inspection, repair, and losses due to structural failure. In order to study a simple case, let us assume that:

1. Initiation of damages in a component is described by a stationary Poisson process with mean initiation rate $\nu = 5$/year.

2. Once damage initiates, it grows linearly with time as described by Eq. (5.17) with $\alpha = 1$.

3. 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 a coefficient of variation, $V_C = 0.5$.

4. The detectability of damages is described by Eq. (5.27).

5. All detected damages are repaired.

Assume that the total cost can be expressed as

$$C_T = C_{ins} + C_{rep} + C_f \cdot F(t_L), \tag{5.43}$$

in which C_{ins} is expected inspection cost, C_{rep} is expected repair cost, and C_f is the loss due to structural failure. Assume further that C_f is constant, and C_{ins} and C_{rep} are evaluated by the following formulas:

$$\begin{aligned} C_{ins} &= \alpha_{ins} \cdot \sum_{j=1}^{m} E\left[A_{ins_j}\right]\left(1 - x_{th_j}\right)^b \\ &= \alpha_{ins}\left(1 - x_{th}\right)^b \sum_{j=1}^{m} E\left[A_{ins_j}\right] \end{aligned} \tag{5.44}$$

$$C_{rep} = \alpha_{rep} \cdot \sum_{j=1}^{m} E\left[A_{rep_j}\right] E\left[X_{max}\left(t_{R_j}\right)\right]^2, \tag{5.45}$$

in which α_{ins} and α_{rep} are constants, A_{ins_j}, and A_{rep_j} are the normalized area (or volume) of a component to be inspected and repaired at the j^{th} inspection, respectively, and $X_{max}\left(t_{R_j}\right)$ is the maximum damage intensity prior to the repair at t_{R_j}. In Eqs. (5.44 and 5.45), it is assumed that C_{ins} depends on the quality and extent of inspection, while C_{rep} is a linear function of the area to be repaired and a nonlinear function of maximum damage intensity to be repaired. The parameter, b, in Eq. (5.44) is set equal to 20 so that $C_{ins}|x_{th} / C_{ins}|x_{th=0.01} \approx 0.15$. For strategies involving full inspections,

$$E\left[A_{ins_j}\right] = E\left[A_{rep_j}\right] = 1 \qquad \text{for } j = 1, \ldots, m. \tag{5.46}$$

Future inspection and repair costs have not been discounted to present worth in the interest of simplicity.

The component is designed for flexure by Eq. (5.11), assuming that $L_n = D_n$. The initial resistance, R_0, is described by a lognormal distribution, with mean of $1.15M_n$ and c.o.v. of 0.15. The probabilistic models of load events are presented in Table 5.1. No data could be located to define the costs of inspection, repair, and failure in absolute terms. Thus, several cases were considered in which the relative costs were varied to determine the sensitivity of the optimal strategy to these relative costs. These relative costs of inspection, repair, and losses due to failure determined by α_{ins}, α_{rep}, and C_f in Eqs. (5.43–5.45) are summarized in Table 5.4. P_{f_T} is assumed to equal 4×10^{-4} in 60 years.

The gradient-based nonlinear optimization program MINOS[307] was employed to perform the minimum cost optimization. Since the mathematical programming problem expressed by Eq. (5.42) is nonconvex, the solution depends on the starting point and might be locally rather than globally optimum. To test the validity of a solution, several different starting points were selected. In the following, "optimum solution" means a locally optimum solution.

5.3.1 Full-Inspection Survey

The decision variables in the optimization are the times at which inspections/repairs are carried out, $t_R = \left\{t_{R_1}, ..., t_{R_m}\right\}$, and the threshold value of detection for inspections, $x_{th_1} = \ldots = x_{th_m} = x_{th}$. The number of inspections, m, during the service life of the structure is assumed to be given. The influence of m can be taken into account by solving the optimization problem for a number of different values of m to identify that value for which C_T is minimum. With this approach, the more complicated integer programming problem can be avoided.

Besides the constraint on failure probability in Eq. (5.42), additional constraints are introduced by assuming that performance requirements of the structure preclude inspection more often than once every four years and that x_{th} for the best available NDE procedure, $x_{th_{min}}$, is either 0.05 or 0.005

$$\Delta t_{R_j} \geq 4 \text{ years}, \qquad j = 1, ..., m+1, \tag{5.47}$$

$$x_{th} \geq x_{th_{min}}, \tag{5.48}$$

in which $\Delta t_{R_j} = t_{R_j} - t_{R_{j-1}}$; $t_{R_0} = 0$; and $t_{R_{m+1}} = t_L$.

The optimum solutions for policies involving full inspection/repair with given values of m are shown in Tables 5.5–5.7, and the expected costs associated with these solutions and the mean degradation function associated with the optimum policy are presented in Figs. 5.15–5.17. In the first two examples, the ratios among α_{ins}, α_{rep}, and C_f are different. When the repair is relatively expensive (Table 5.5 and Fig. 5.15), t_{R_j} tends to shift toward the early period of the service life. With this strategy, the component should be repaired when it is lightly damaged, where the cost of repair can be reduced. However, if the service life of the component were to be extended beyond $t_L = 60$ years, a large repair cost would be expected. Because there is some time between the last scheduled inspection/repair and t_L, the component would be degraded relatively heavily at t_L [Fig. 5.15(b)]. If it is considered possible at the time of the reliability analysis that the service life might be extended, an additional constraint in the optimization, such as a lower bound of $g(t)$, might be imposed. Since the inspection is relatively inexpensive, x_{th} has little effect on the total cost, and the optimal x_{th} equals its lower bound [Eq. (5.48)]. Moreover, since only the minimum of repairs is performed to meet the constraint in terms of failure probability, the failure probability equals its upper bound [Eq. (5.42)], here 4×10^{-4} in 60 years. The optimum number of inspections/repairs is three; however, Fig. 5.15(a) shows that the total cost is insensitive to m if $2 \leq m \leq 5$.

When the total expected cost is dominated by the expected losses due to structural failure (Table 5.6 for $m = 1 \ldots, 5$ and Fig. 5.16), inspection/repair should be carried out with the best available NDE method of damage detection at nearly uniform intervals during the lifetime of the component. With this strategy, the maximum effect of repair on reducing failure probability can be obtained. The optimum full inspection policy for $m \geq 6$ is to perform inspections/repairs at uniform intervals during the service life with the best available method of damage detection. The expected total cost of these strategies is also presented in Fig. 5.16(a). Since the failure probability can be reduced by increasing the number of inspection/repairs, the optimum number of inspections, m_{opt}, reaches its upper bound [Eq. (5.47)] of 14 in 60 years. However, the reduction of C_T by increasing m is small for m greater than about 7. With this strategy, the mean strength of the component can be kept near its original state throughout its service life [cf. Fig. 5.16(b)].

In the third example [Table 5.7 and Figs. 5.17(a) and 5.17(b)], the damage growth rate and the relative cost are the same as that of example 1, but $x_{th_{min}}$ is increased to 0.05. The optimization problem is solved for $m \geq 3$ because the constraint on failure probability cannot be met when $m \leq 2$. The optimum times of inspection shift toward the later stage of service life because an NDE method with relatively large detection threshold does not work effectively if the damage intensity is small. As a result, the intensities of detected damages are relatively large, that leads to expensive repair. The optimum number of inspections/repairs is three; however, C_T is insensitive to m for $3 \leq m \leq 5$.

The intervals between inspections/repairs are nearly uniform in these examples regardless of damage accumulation. Results presented elsewhere[298] suggest that if the damage growth rate is nonlinear, inspection intervals remain nearly uniform, but the optimal number of inspections may not be the same. It is assumed that damage initiation is stationary and that the threshold value of damage detection is the same in all cases. Since most rapidly growing damages that are overlooked at an inspection initiate after the previous inspection, the cdf of damage remaining immediately following an inspection is nearly identical to the cdf of damage remaining immediately following the previous inspection/repair. These factors cause the maintenance intervals to be nearly uniform.

5.3.2 Partial Inspection Strategy

In the optimization involving a partial inspection strategy, there exists a number of decision variables, such as the number of partial inspections to be scheduled, m_P, critical value of damage intensity at the ith partial inspection below which damage may not be repaired, x_{cr_i}, times at which the ith partial inspection is carried out, t_{P_i}, the number of full inspections after a stage III (see Sec. 5.2.2.2) inspection is performed at t_{P_i} (branch i), m_i, the threshold value of the jth inspection/repair in branch i, $t_{R_{ij}}$. Because extensive computational effort is required to optimize all decision variables, the following relations among variables are assumed:

$$x_{cr_i} = x_{cr}, \tag{5.49}$$

$$x_{th_{ij}} = x_{th}, \tag{5.50}$$

$$x_{th} = x_{cr}, \tag{5.51}$$

$$t_{P_i} = t_{P_1} + (i-1)\Delta t_P, \tag{5.52}$$

$$t_{R_{ij}} = t_{R_{i1}} + (j-1)\Delta t_{R_i}, \tag{5.53}$$

$$\Delta t_{R_i} = \alpha_R \cdot t_{P_i}, \tag{5.54}$$

$$m_i = m_P - i + 1; \qquad \text{for } j = 1, \ldots, m_i \text{ and } i = 1, \ldots, m_P \tag{5.55}$$

in which Δt_P and Δt_{R_i} = respectively, the intervals between partial inspections and those between full inspections/repairs after a stage III inspection is carried out. It was observed in the analysis of optimum full inspection/repair strategies that the optimum value of x_{th} converges toward the minimum value when inspection is inexpensive relative to repair and failure costs. Since this usually is the case in practice, it is assumed that $x_{th} = x_{th_{min}}$. Thus, the problem is reduced to one having three decision variables, t_{P_1}, Δt_P, and α_R.

The optimum partial inspection strategies for example 1 (see Table 5.4) are presented in Table 5.8. Because of the large number of branches, extensive computational effort was required to obtain the optimum solution by MINOS for $m_P = 4$, and the optimum solutions were obtained by solving with several combinations of $\left(t_{P_1}, \Delta t_P, \alpha_R\right)$. The optimum full inspection/repair strategy is also presented in the last line of Table 5.8 for comparison. The optimum partial inspection strategy ($m_P = 4$) is slightly more cost-efficient than the optimum full inspection/repair strategy. The benefit of partial inspection/repair would increase further if the t_{P_i} and $t_{R_{ij}}$ in Eq. (5.52) and Eq. (5.53) were optimized rather than t_{P_1}, Δt_P, and α_R.

5.3.3 Example Illustrating Development of Optimum Inspection/Repair Strategy

The methodology is illustrated below through its application to a simply-supported reinforced concrete slab subjected to corrosion as a result of carbonation of the cover concrete.

5.3.3.1 Description of slab

The dimensions of the slab and the nominal material strengths are shown in Table 5.9. Assume that

1. The slab is subjected to time invariant dead load, D, and intermittent live load L, both of which are distributed uniformly on the beam. The load effect at location u, $S(u)$ is given by

$$S(u) = q(u) \cdot (D + L), \tag{5.56}$$

   in which $q(u) = 4(ul - u^2)/l^2$; and $\max\{S(u)\} = S(l^2) = D + L$.

2. The slab is underreinforced and only fails due to flexure if all reinforcing bars within a given cross section yield.

3. Compressive strength of the concrete changes with time due to continuous hydration of cement, while the section area of the reinforcement decreases with time due to corrosion. Other engineering properties of the beam are time-invariant.

The slab is designed based on the current design requirement expressed by Eq. (5.11) with $L_n = D_n$. The nominal load effects of dead and live load are both 87.6 kN·m. The nominal flexural capacity[16] is M_n = 301.6 kN·m. The initial resistance of the slab, R_0, is described by a lognormal distribution with mean value, $\mu_R = 1.11M_n$, and coefficient of variation 0.13.[287] The time-dependent stochastic models are presented in Table 5.1.

5.3.3.2 Time-dependent strength in flexure

Reinforcing bars are generally ductile, and so assumption 1 implies that the slab can be modeled as a strictly parallel system composed of ductile identical components. The reinforcing bars are assumed to be supplied by the same manufacturer and the initial yield strengths and the section areas of reinforcing bars are assumed to be perfectly correlated. The initial flexural strength, R_0, of the slab is

$$R_0 = \alpha_0(1 - 0.59\alpha_0)bd^2 f_{c_{28}}, \tag{5.57}$$

in which

$$\alpha_0 = \frac{\rho f_y}{f_{c_{28}}}, \tag{5.58}$$

$$\rho = \frac{n_s A_s}{bd} = \text{reinforcement ratio}, \tag{5.59}$$

$f_{c_{28}}$ = compressive strength of concrete at 28 days; n_s = number of reinforcing bars; and f_y and A_s = yield strength and section area of a reinforcing bar, respectively. Assume for the moment that $N(t) = n$ corroded areas exist at locations $\mathbf{U} = \mathbf{u} = \{u_1, \ldots, u_n)$ at time t. Let us define the intensity of damage at location u_j at time t as

$$X_j(t) = 1 - \frac{A_s(t, u_j)}{A_s}, \tag{5.60}$$

in which $A_s(t, u_j)$ = residual cross section area of a reinforcing bar at u_j at time t. The flexural strength at u_j at time t, $R(t, u_j)$, is expressed as

$$R(t, u_j) = \alpha(t, u_j)\left[1 - 0.59\alpha(t, u_j)\right]bd^2 f_c(t), \tag{5.61}$$

in which

$$\alpha(t, u_j) = \frac{\rho(t, u_j) f_y}{f_c(t)}, \tag{5.62}$$

$$\rho(t, u_j) = \frac{n_s A_s(t, u_j)}{bd}, \tag{5.63a}$$

$$\rho\left(t, u_j\right) = \frac{n_s\left[1 - X_j(t)\right] A_s}{bd}, \tag{5.63b}$$

in which $f_c(t)$ = compressive strength of concrete at time t. In the absence of environmentally assisted degradation, $f_c(t)$ generally is larger than $f_{c_{28}}$ due to continued hydration of the concrete. The degradation function for the slab is defined by

$$G(t) = \min\left\{ \min_{j=1}^{n}\left[\frac{R\left(t, u_j\right)}{q\left(u_j\right)R_0}\right], G_{gain}(t)\right\}, \tag{5.64}$$

in which $G_{gain}(t)$ = ratio of the strength of the component without any damage at time t to the initial strength, given by

$$G_{gain}(t) = \frac{1 - 0.59\dfrac{n_s A_s f_y}{bdf_c(t)}}{1 - 0.59\dfrac{n_s A_s f_y}{bdf_{c_{28}}}}. \tag{5.65}$$

Then the cdf of $G(t)$ can be expressed as[281]

$$F_G(z) = 1 - \exp\left[-\int_0^t \nu(\tau)\,d\tau\left\{1 - \frac{1}{V}\int_v P\left[\frac{R(t,u)}{R_0 q(u)} > z\right]du\right\}\right];$$
$$0 \le z < E\left[G_{gain}(t)\right], \tag{5.66a}$$

$$F_G(z) = 1; \qquad z \ge E\left[G_{gain}(t)\right]. \tag{5.66b}$$

5.3.3.3 Determination of optimum strategy

The initiation of corrosion following carbonation of the outer portion of concrete is described by a Poisson process with parameter $\nu(t)$ expressed as

$$\nu(w) = \begin{cases} 0, & \text{for } w < t^* \\ \nu, & \text{for } w \ge t^* \end{cases}, \tag{5.67}$$

for purposes of the example, assume that

- t^* is deterministic and equal to 10 years;
- At a given cross section, corrosion initiates in the reinforcing bars at the same time; however, the point along the span at which corrosion initiates is random;
- The mean initiation rate of corrosion, ν, is 0.2/year;
- The intensity of a damage due to corrosion is described by

$$X(t) \approx \frac{4R_c}{d_s}(t - T_I), \tag{5.68}$$

in which R_c = corrosion rate; d_s = diameter of a reinforcing bar; and T_I = initiation time of the corrosion; Eq. (5.68) is valid if the depth of corrosion is small relative to the diameter of the reinforcing bar;

- R_c is a time-invariant random variable described by a lognormal distribution with mean value, μ_{R_c}, of 50 μm/year and coefficient of variation, V_{R_c}, of 0.5.[308] Then the damage growth rate, C, in Eq. (5.17) defined by

$$C = \frac{4R_c}{d_s}, \tag{5.69}$$

is also a time-invariant random variable described by a lognormal distribution with $V_C = 0.5$. The mean of the damage growth rate, μ_C, equals 156 μm/year for D = 32 mm;

- The 28-day specified compressive strength of concrete equals 27.6 MPa. The corresponding mean of the compressive strength at 28 days is 28.7 MPa.[287] The specified yield strength of the reinforcement is 414 MPa and the mean is 465 MPa; and

- Concrete matures with time; the compressive strength increases during the first 10 years and does not change thereafter. The mean compressive strength (in megapascals) at time t is evaluated by Eq. (5.16).

The mean degradation functions of the slab subjected to reinforcement corrosion are presented in Fig. 5.18. Three cases are illustrated — the slab is fully inspected/repaired: (1) At t_R = 35 years with x_{th} = 0.05; (2) at t_R = 20, 30, 40, and 50 years with x_{th} = 0.15; and (3) not inspected/repaired. For comparison, the mean degradation function of a slab with continuous hydration of cement but without corrosion is also presented. Without corrosion, the strength of the slab increases about 5% over 60 years, while it decreases 22% at 60 years due to reinforcement corrosion if repairs are not made.

The failure probabilities of the slab with mean strength degradation illustrated in Fig. 5.18 are presented in Fig. 5.19 along with the failure probability with $g(t)$ = 1. Since the latter probability can be considered as that intended for design, it would be the target failure probability for developing an inspection/repair strategy if degradation occurs. Since $F(40)$ without repair is smaller than $F(40)$ with $g(t)$ = 1, inspection/repair would not be necessary if the intended service life is 40 years or less. If the intended service life is 60 years, some inspections/repairs are required. Both strategies described in the figure are feasible and compatible in terms of failure probability.

The optimum inspection/repair strategies for this slab for t_L = 60 years are presented in Table 5.10 and Fig. 5.20 for full inspection/repair strategies. The total cost is evaluated by Eqs. (5.43–45) with α_{ins}; α_{rep}; C_f = 1:500:10^4 and with $x_{th_{min}}$ = 0.05. In this example, the optimum strategy is to perform three inspection/repair operations at 19.6, 24.8, and 30.7 years using an NDE method with a capability of detecting defects of a size causing 5% loss of flexural capacity.

5.4 COMMENTARY ON RELIABILITY-BASED CONDITION ASSESSMENTS

A methodology has been developed that can be used in assessments of current and future structural reliability and performance of concrete structures in NPPs. The effects of aging and environmental stressors that might diminish the ability of these structures to withstand future operating, extreme environmental, or accident conditions have been taken into account. The overall condition assessment methodology is summarized in Fig. 5.21.

Basic concepts for time-dependent reliability assessment of a single component or system of components were developed. Time-dependent strength models were convolved with the stochastic load models to determine limit state probabilities and hazard functions. The sensitivity of the reliability to various parameters describing load occurrence and strength degradation was evaluated using conceptual models. Parametric analyses showed that the time-dependent system reliability is sensitive to the choice of mean initial component strength, mean strength degradation rate, and general stochastic characteristics of the time-varying load. Less sensitivity was shown to dependence in component strengths within a system. Limit state probability and hazard functions developed for the conceptual models both show a relatively sharp increase in magnitude shortly after 40 years. This indicates that under aggressive environments, evidence of past acceptable performance may not be sufficient to ensure future acceptable performance. Therefore, periodic in-service inspections may be required to ensure future acceptable performance.

The role of in-service inspection and repair on the time-dependent reliability was investigated. The quality of an inspection was described by a detectability function, d(x). As a result of an in-service inspection/repair activity, the distribution function of damage intensity can be updated using a Bayesian analysis. The resulting posterior distribution can be used to update the time-dependent strength. This information, when combined with the time-dependent reliability analysis, can be used to evaluate the effect of inspection intervals and NDE detection capabilities on the failure probabilities of a component.

The degradation in strength of a structure with random damage can be evaluated based on a growth model for individual damages that can be obtained from experimental data. This method is combined with the time-dependent reliability analysis to design optimum strategies for inspection and maintenance that minimize the expected future cost, while maintaining the limit state probability of the structure at or below an established target failure probability. Optimum inspection/repair strategies are sensitive to the relative cost of inspection, repair, and failure as well as to the threshold value of damage detection. If the consequences of failure are high, a policy requiring inspection and maintenance at nearly uniform intervals over the service life of the structure results in near-minimum total cost. Additional data on damage initiation, NDE capabilities and the cost of inspection, repair and failure are required to complete the development of an optimum inspection/repair strategy.

Several areas for future work were identified. They include:

1. Supporting Data — Time-dependent reliability analysis provides a quantitative framework for condition assessment of concrete structures. The methodology is data-intensive in comparison to the subjective rating procedures that often are used in the in-service evaluation of ordinary civil structures. Implementation and extension of the method to realistic condition assessment is difficult due to a lack of

supporting quantitative data on strength degradation models, including initiation and rate of damage growth, the mean occurrence rate of load events and cdf of the intensity of time-varying loads. The threshold value of defect detection associated with each NDE method must be identified to evaluate the effect of inspection/repair on structural failure probability. Information about relative cost of inspection, repair, and losses due to failure is required to design an optimum inspection strategy.

2. Interactions between Degradation Mechanisms — A structural component may degrade due to more than one environmental factor. In such a case, a damage model reflecting the interactions between degradation mechanisms must be developed. With such a model combined with time-dependent reliability analysis developed in this study, the failure probability of a component can be evaluated.

3. Verification — The reliability models established for condition assessment should be validated through application to laboratory and prototypical structures. In laboratory tests, the effects of environmental factors can be evaluated under controlled conditions. Selecting an existing structure with available design documentation as a prototype, the analysis should be run forward to the present time and the current predicted condition should be compared with the estimate of actual present condition. The results can be utilized to modify the theoretical models.

4. User-Oriented Guideline — The condition assessment and life prediction methodology may not be suitable for routine use in practice without simplification. User-oriented requirements and guidelines for in-service evaluation of concrete structures should be developed.

Table 5.1. Statistical properties of NPP loads.

Load	Rate of Occurrence, λ	Duration, τ	Mean^{+}	cov	pdf
Dead	n/a	n/a	$1.0D_n$	0.07, 0.10	Normal
Live	0.5/yr	3 mo	$0.4L_n$	0.50	Type I
Earthquake	0.05/yr	30 sec	$0.078\,E_{ss}^{*}$	0.85*	Type II**
Equipment	n/a	40 yr	$1.0Q_n$	0.04	Normal
Acc. Pressure	1/10000 yr	20 min	$0.8P_a$	0.20	Type I
Acc. Temp.	1/10000 yr	20 min	$0.9T_a$	0.08	Type I

$^{+}D_n$, L_n, Q_n, P_a, and T_a are nominal loads;

* E_{ss} is safe-shutdown earthquake load.

**For the pdf of annual maximum values, $F_{E_{ann}}(x)$.

Table 5.2. Statistical data for reinforced concrete.

Parameter	Mean	cov	pdf
Material Strength			
f_c, psi	$960 + 0.8\,F_c'$	0.12	Normal
f_t, psi	$6.4\,\sqrt{f_c}$	0.18	Normal
f_y (ASTM A615/Gr.60)	67 ksi	0.11	Lognormal
f_u (ASTM A416/A421)	270 ksi	0.04	Lognormal
Dimensions			
Overall dimensions (in.)	Nominal, h	$0.4/h$	Normal
Placement of reinf. (in.)	Nominal, d	$0.6/d$	Normal

Table 5.3. Probabilistic descriptions of resistance of concrete components.

Limit State	Mean*	cov	pdf
Beam, flexure	$1.11M_n$	0.13	Lognormal
Beam, shear	$1.22V_n$	0.18	Lognormal
Wall, shear	$1.35V_n$	0.18	Lognormal
Short wall, shear	$1.70V_n$	0.18	Lognormal
Slabs, flexure	$1.12M_n$	0.14	Lognormal
Short column, compression	$1.13P_n$	0.14	Lognormal
Short column, tension	$1.12P_n$	0.13	Lognormal

*The nominal values M_n, V_n, and P_n are the capacities that would be computed from the ACI Standards 318[16] or 349.[17]

Table 5.4. Models analyzed in examples.

Example	$x_{th_{min}}$	Relative cost $\alpha_{ins}:\alpha_{rep}:C_f$
1	0.005	$1:5000:10^4$
2	0.005	$1:1000:10^7$
3	0.05	$1:5000:10^4$

Table 5.5. Optimum solutions for full-inspection strategies for Example 1 in Table 5.4.

m	$F(60)$ (10^{-4})	x_{th}	t_{R_i} 1	2	3	4	5
1	4.00	0.005	21.8	—	—	—	—
2	4.00	0.005	10.8	20.6	—	—	—
3	4.00	0.005	7.9	13.8	20.3	—	—
4	4.00	0.005	5.8	10.3	15.0	20.3	—
5	4.00	0.005	4.1	8.1	12.1	16.1	22.2

Table 5.6. Optimum solutions for full-inspection strategies for Example 2 in Table 5.4.

m	$F(60)$ (10^{-4})	x_{th}	t_{R_i} 1	2	3	4	5
1	3.43	0.005	30.7	—	—	—	—
2	2.52	0.005	20.3	39.6	—	—	—
3	2.21	0.005	15.2	29.2	44.0	—	—
4	2.05	0.005	12.5	23.7	35.3	47.3	—
5	1.97	0.005	10.0	18.6	27.5	37.4	48.2

Table 5.7. Optimum solutions for full-inspection strategies for Example 3 in Table 5.4.

m	$F(60)$ (10^{-4})	x_{th}	t_{R_i} 1	2	3	4	5
3	4.00	0.005	22.0	31.1	41.0	—	—
4	4.00	0.005	4.0	22.3	31.6	40.9	—
5	4.00	0.005	4.0	8.0	22.1	31.1	41.0

Table 5.8. Optimum solutions for partial inspection strategies for Example 1 in Table 5.4.

m	$F(60)$ (10^{-4})	x_{cr}	t_{P_1}	Δt_P	α_R	C_T	C_{ins}	C_{rep}	$C_f F(60)$
2	4.00	0.005	13.2	4.0	0.7	13.2	1.2	7.9	4.0
3	4.00	0.005	9.1	4.0	0.6	11.5	1.9	5.5	4.0
4[a]	4.00	0.005	5.8	4.0	0.5	11.4	2.2	5.2	4.0
Full	4.00	—	—	—	—	11.5	2.7	4.8	4.0

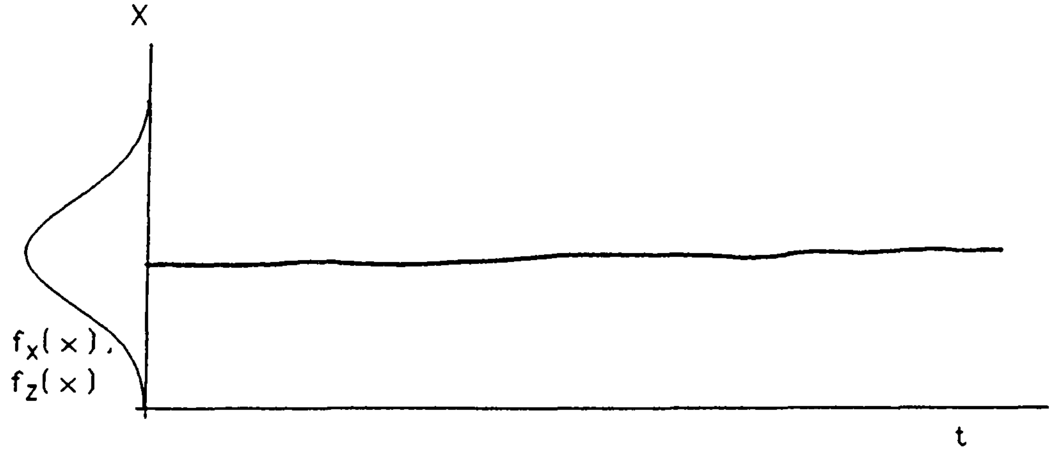
[a]Optimum partial-inspection policy.

Table 5.9. Engineering properties of slab.

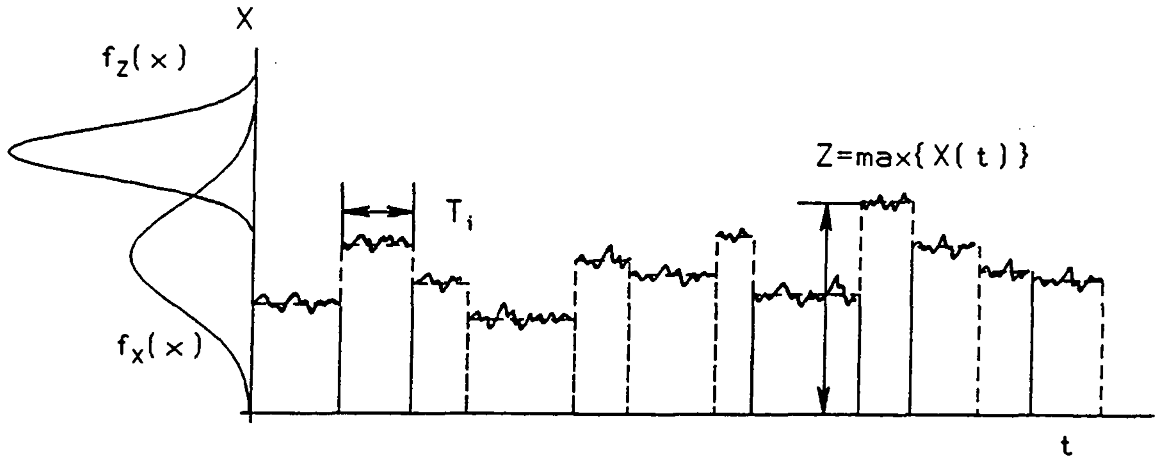
Parameter	Value
Width, b	300 mm
Span, l	7 m
Effective depth, d	500 mm
Height, h	550 mm
Concrete cover	50 mm
Diameter of reinforcing bar, d_s	32 mm
Distance between reinforcing bars, s	100 mm
Number of reinforcing bars, n_s	2
f_c' (specified at 28 days)	27.6 MPa
f_y (minimum specified)	414 MPa

Table 5.10. Optimum solutions for full-inspection strategies for slab.

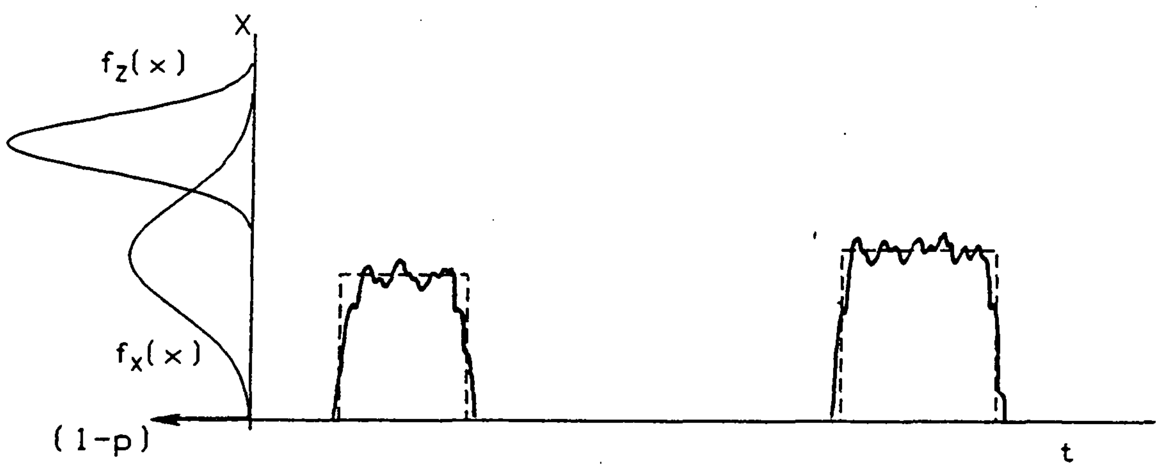
m	$F(60)$ (10^{-4})	x_{th}	t_{R_i} 1	2	3	4	5
1	1.20	0.05	31.2	—	—	—	—
2	1.20	0.05	22.8	30.8	—	—	—
3	1.20	0.05	19.6	24.8	30.7	—	—
4	1.20	0.05	17.0	21.0	25.7	30.7	—
5	1.04	0.05	16.6	20.6	24.6	28.6	32.6



(a) Permanent loads.



(b) Continuous variable loads.



(c) Intermittent variable loads.

Fig. 5.1 Typical sample functions of structural loads.

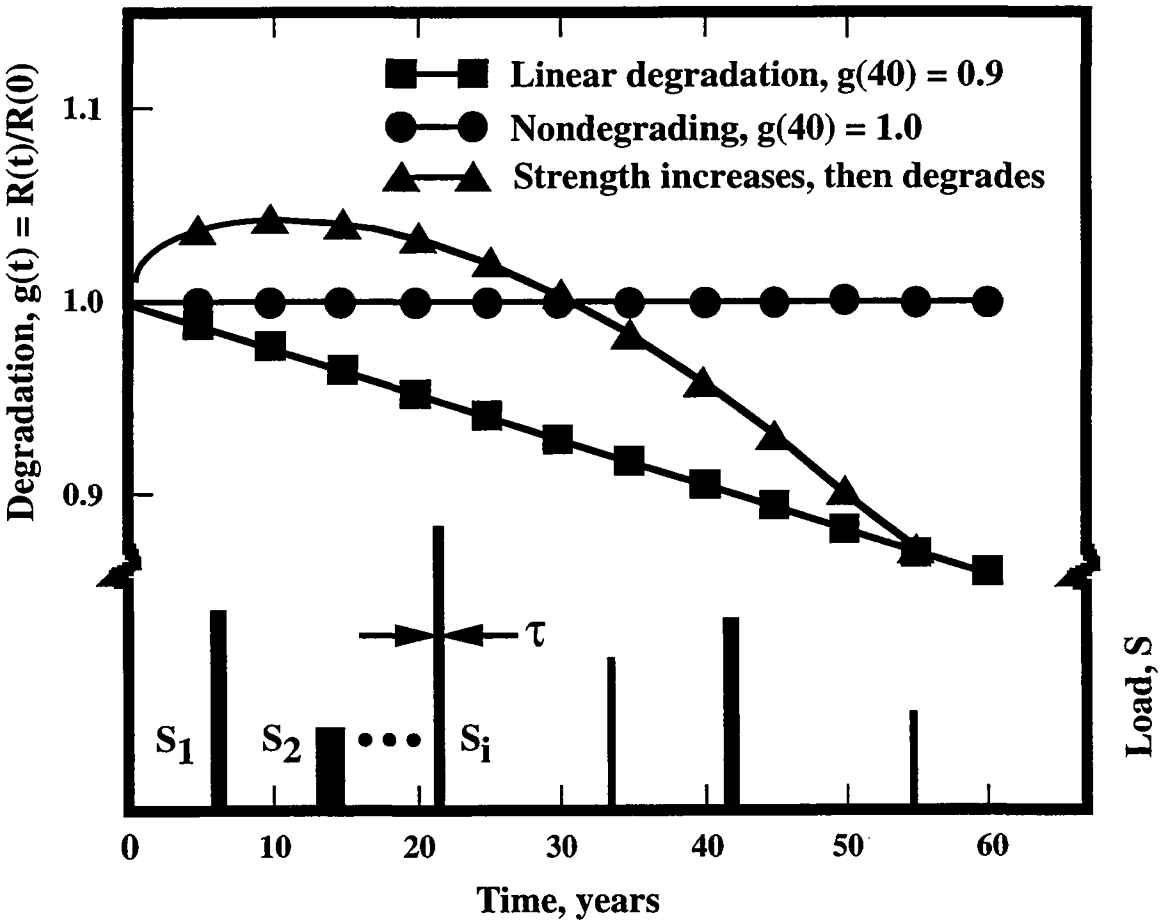


Fig. 5.2 Mean degradation functions of one-way slab.

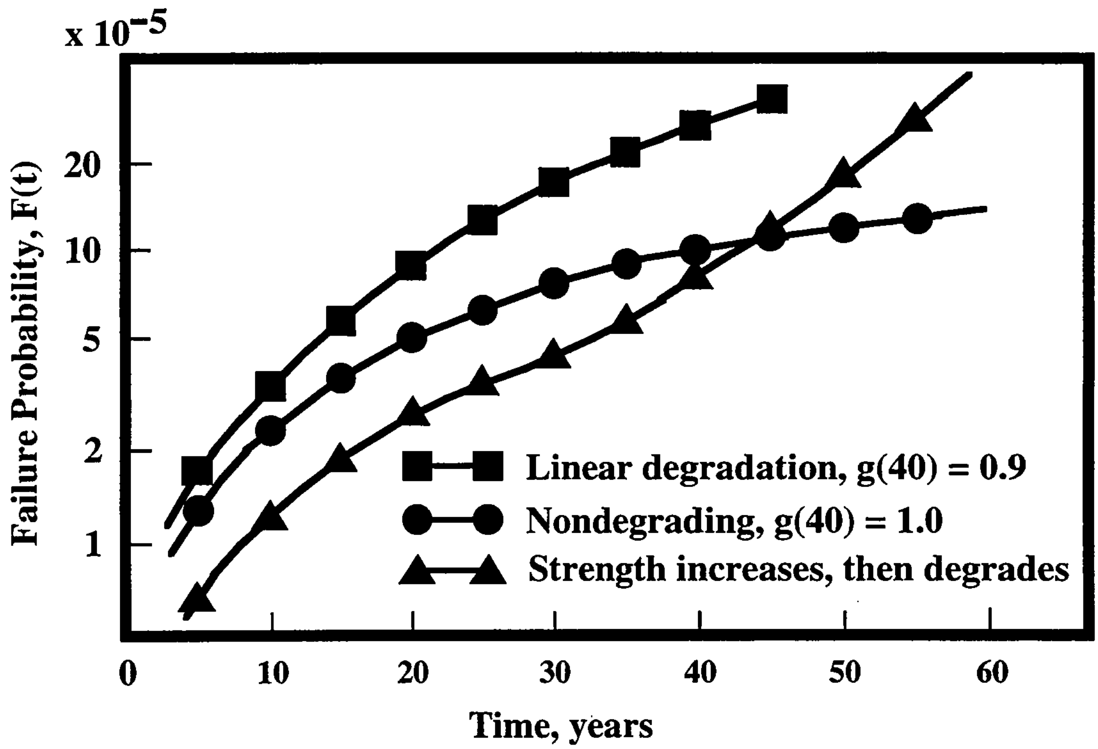


Fig. 5.3 Failure probability of a one-way slab.

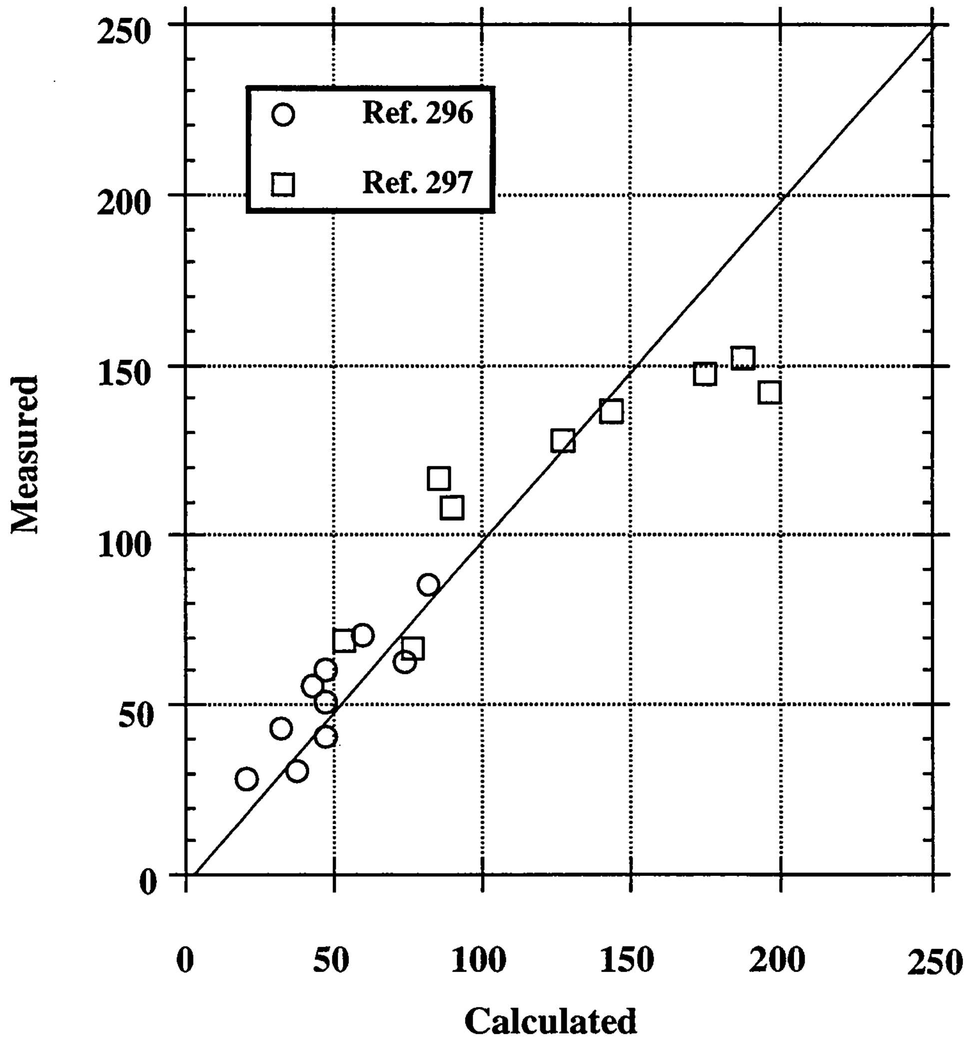


Fig. 5.4 Comparison of measured and calculated shear strength of low-rise reinforced concrete wall.

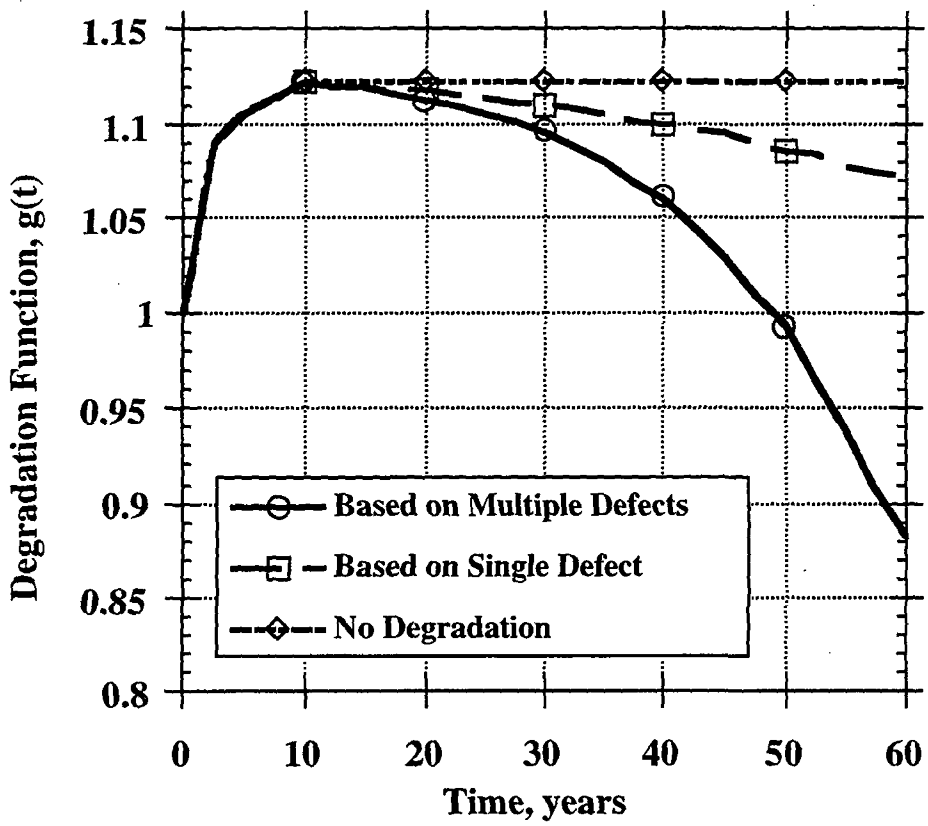


Fig. 5.5 Mean degradation function of wall in shear without repair.

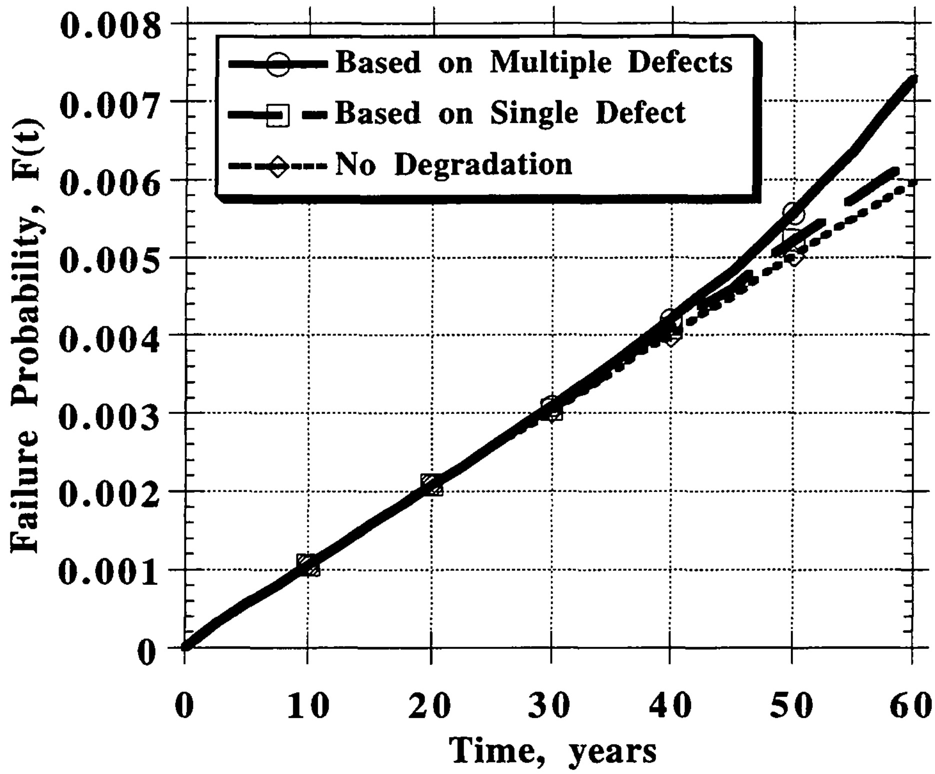


Fig. 5.6 Failure probability of wall in shear without repair.

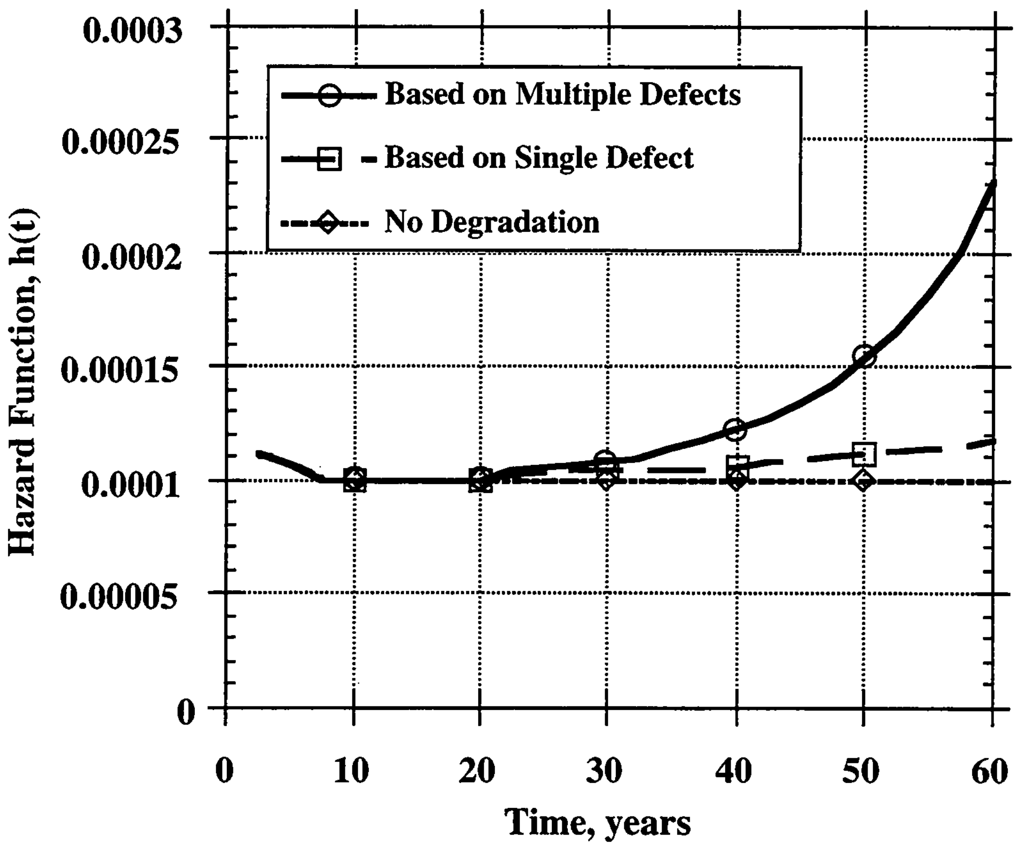


Fig. 5.7 Hazard function of wall in shear without repair.

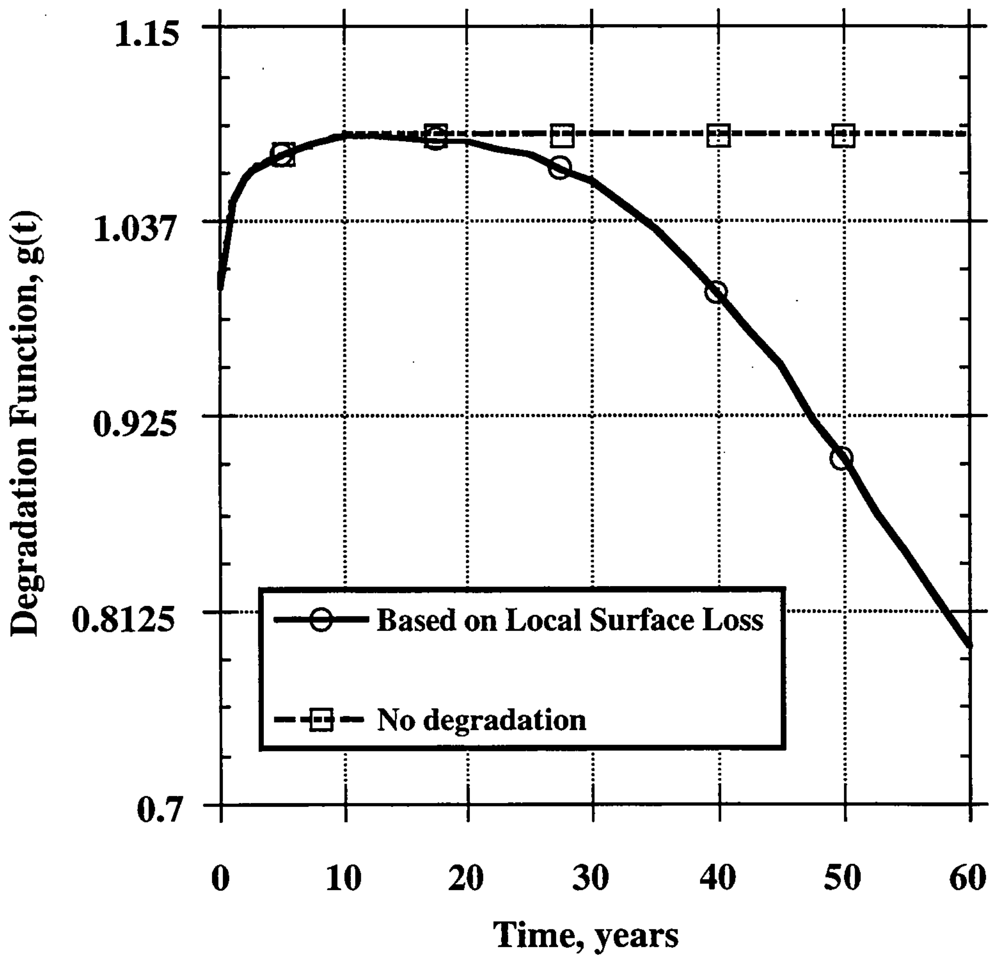


Fig. 5.8 Mean degradation function of wall in flexure/compression.

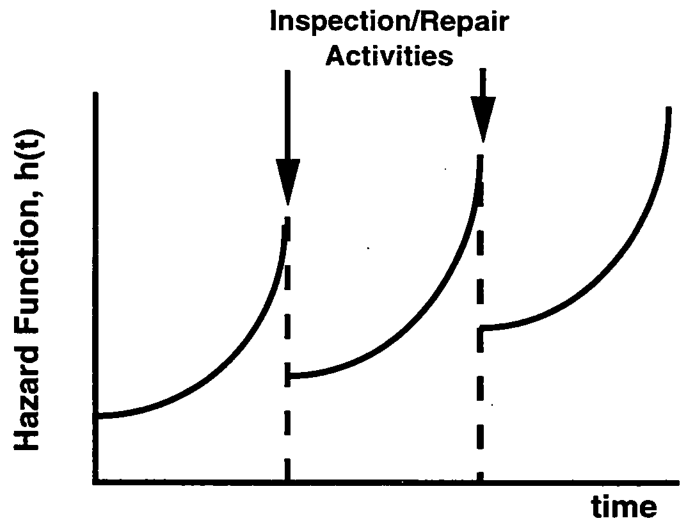


Fig. 5.9 Role of inspection/repair in controlling failure rate.

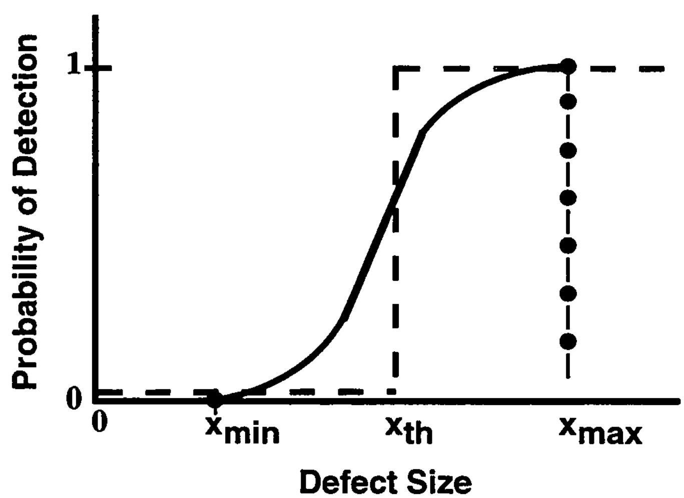


Fig. 5.10 Probability of detection of a defect of size x.

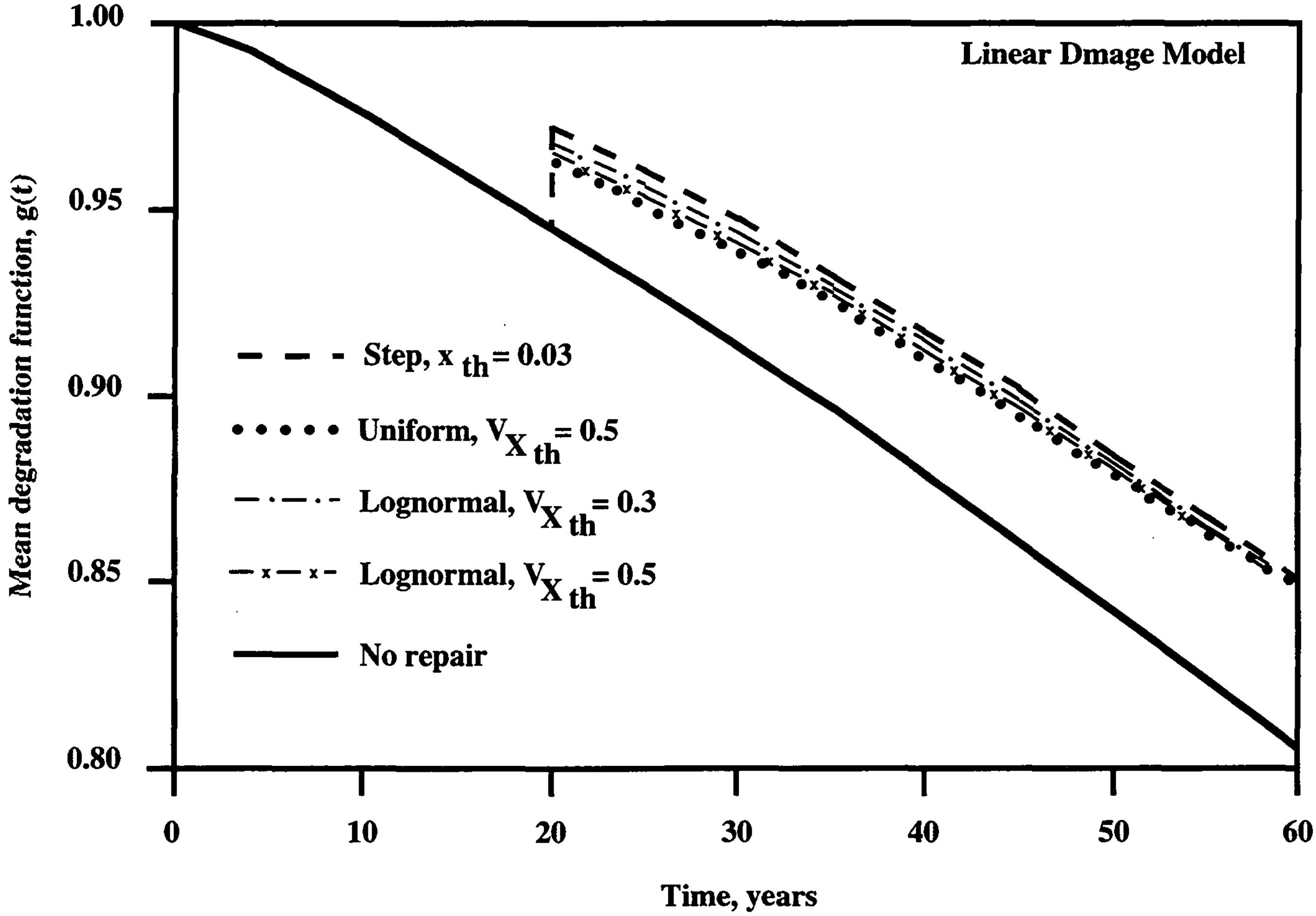


Fig. 5.11 Effect of several detectability functions on mean degradation function of inspection/repair.

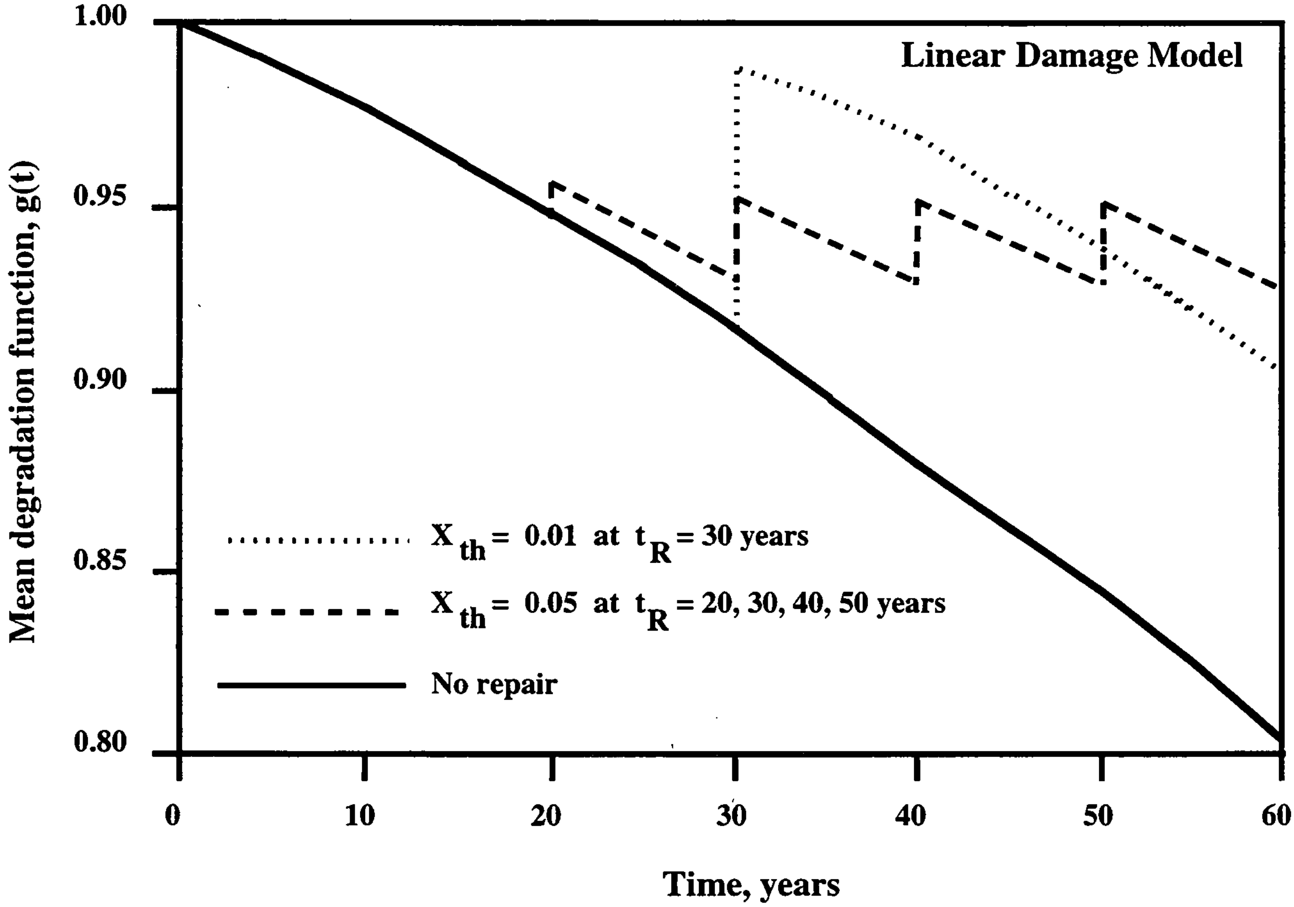


Fig. 5.12 Effect of multiple inspection/repairs on mean degradation function.

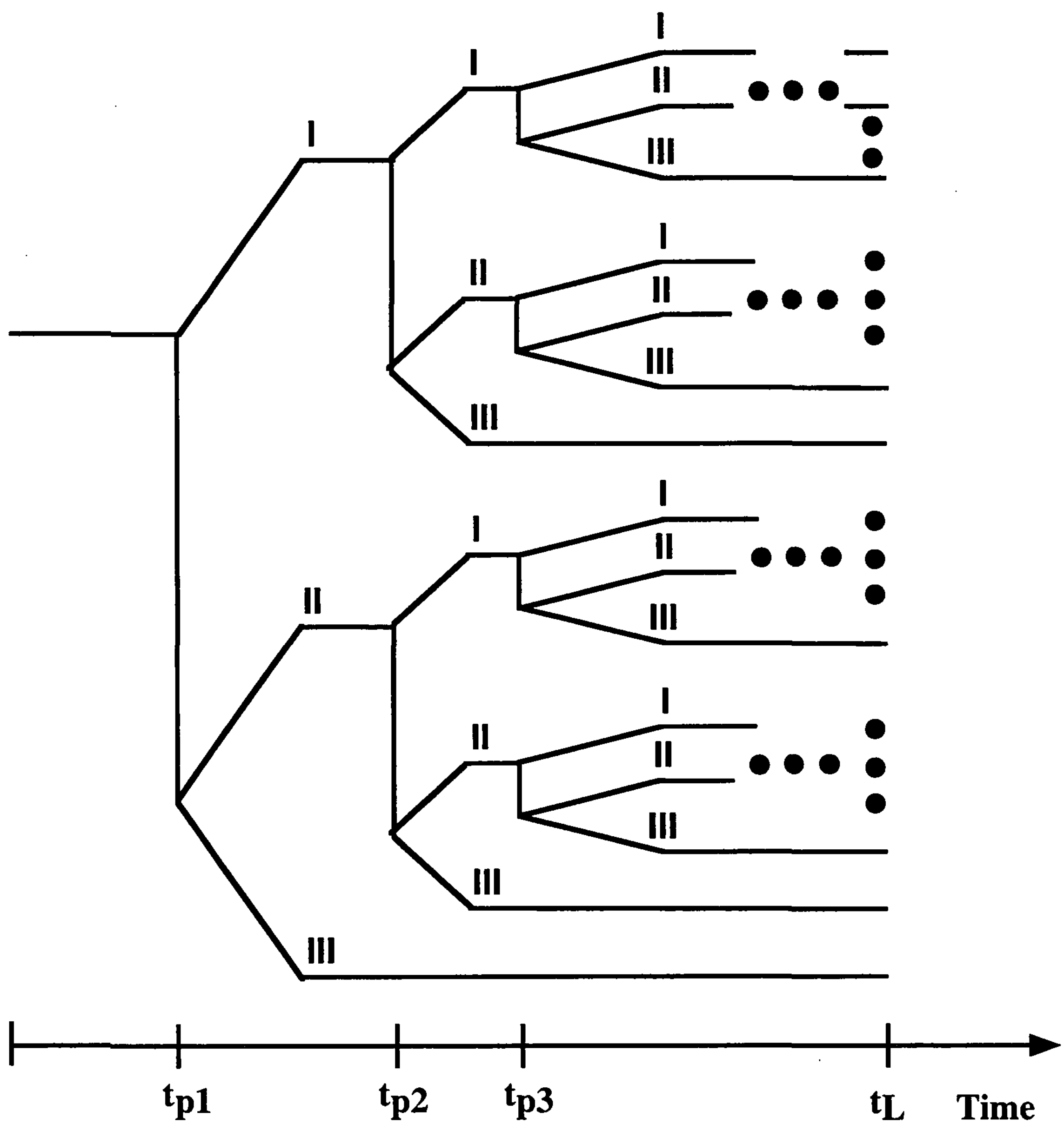
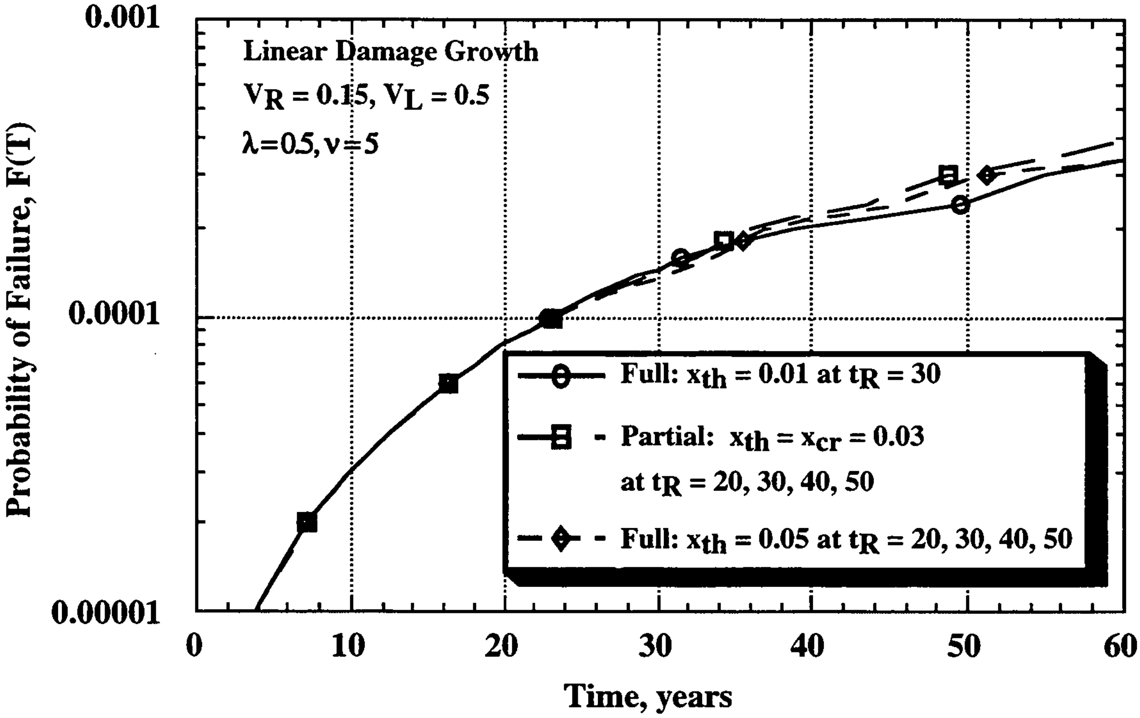
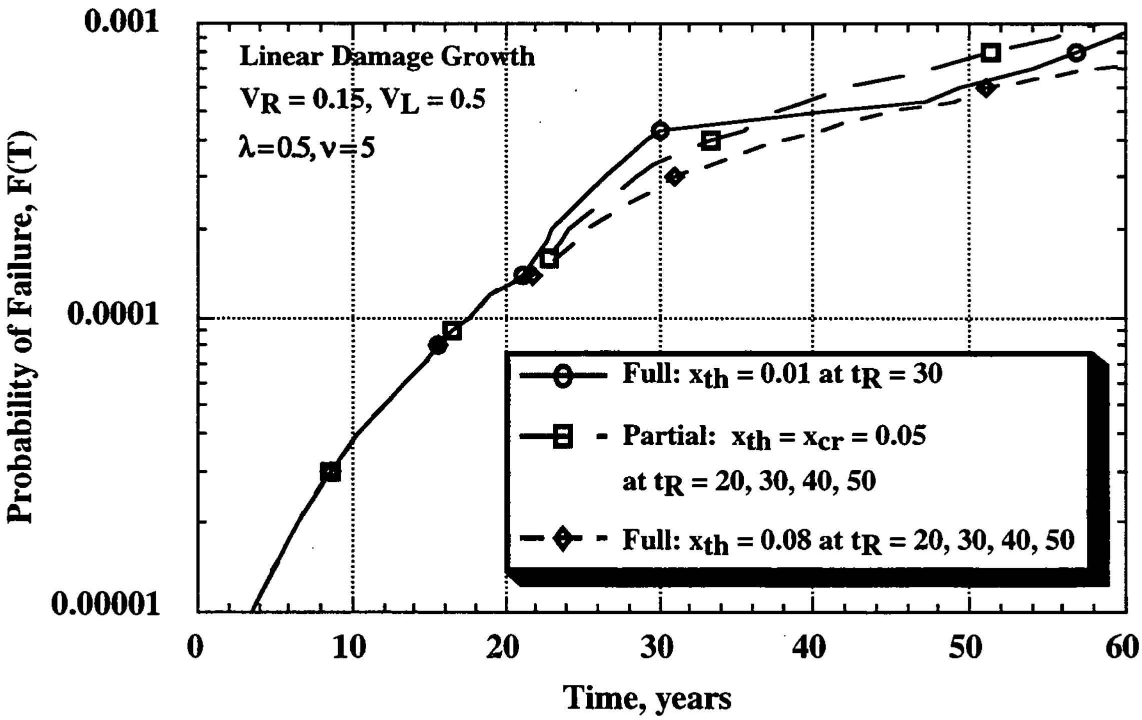


Fig. 5.13 Partial inspection/repair strategies.

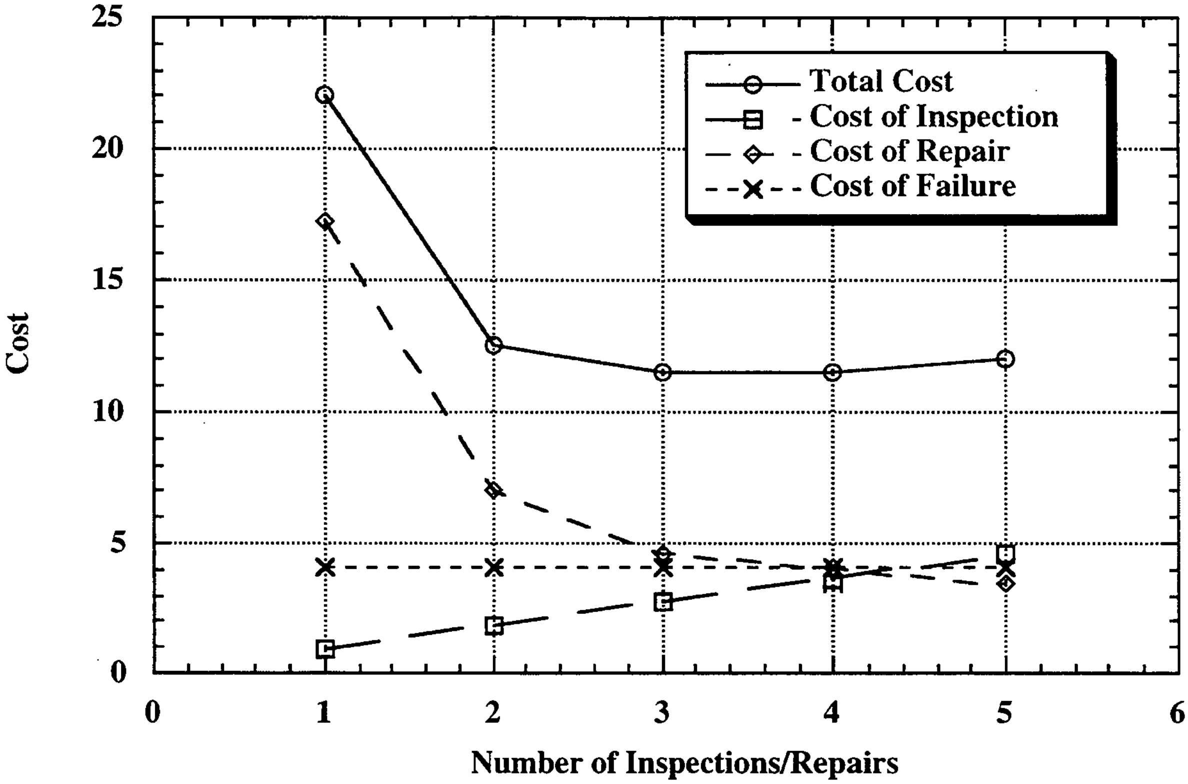


(a). E [X(40) I TI = 0] = 0.05

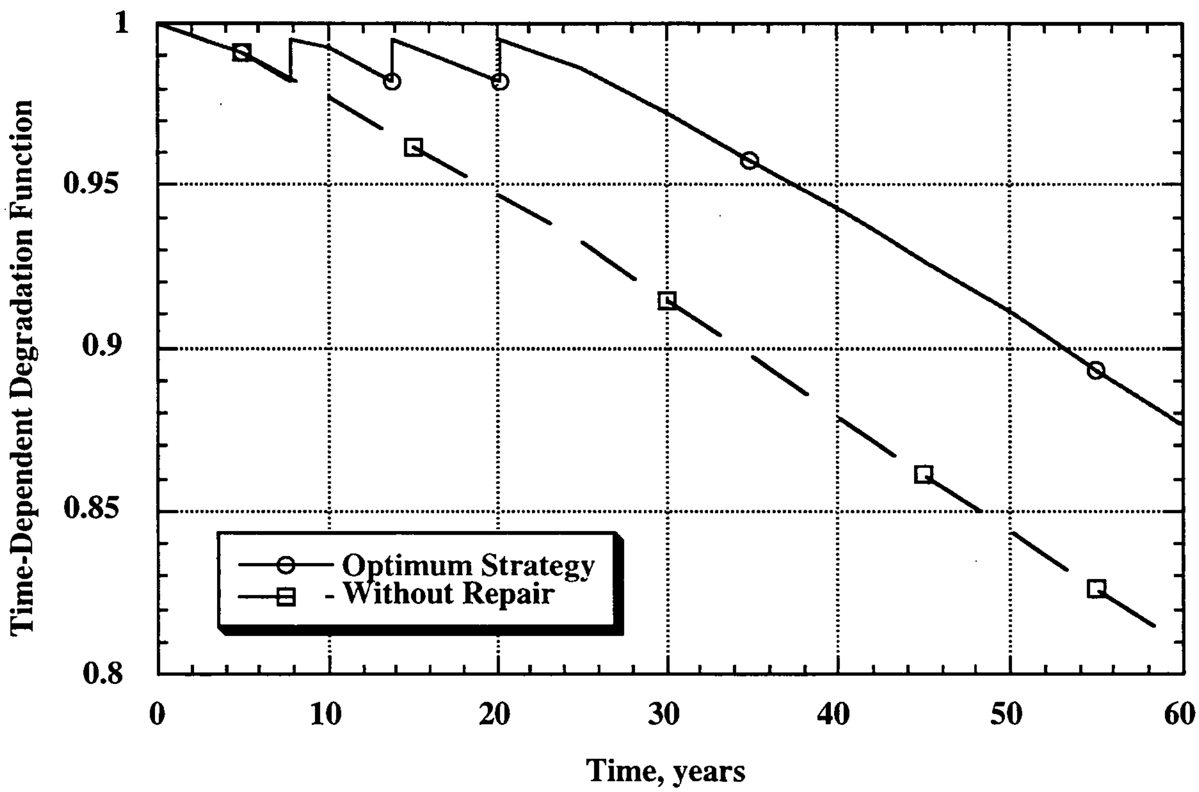


(b). E [X(40) I TI = 0] = 0.10

Fig. 5.14 Failure probability of a component with partial inspection.

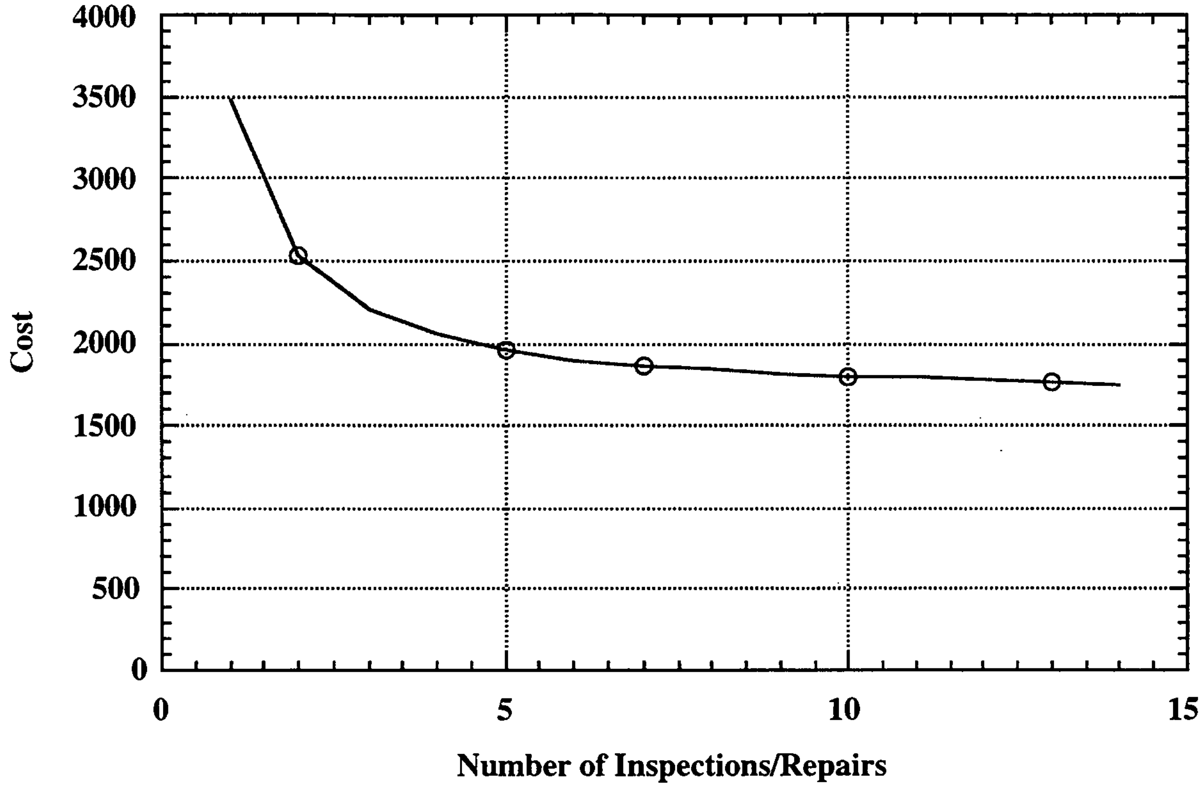


(a) Cost Function at Optimum Solution.

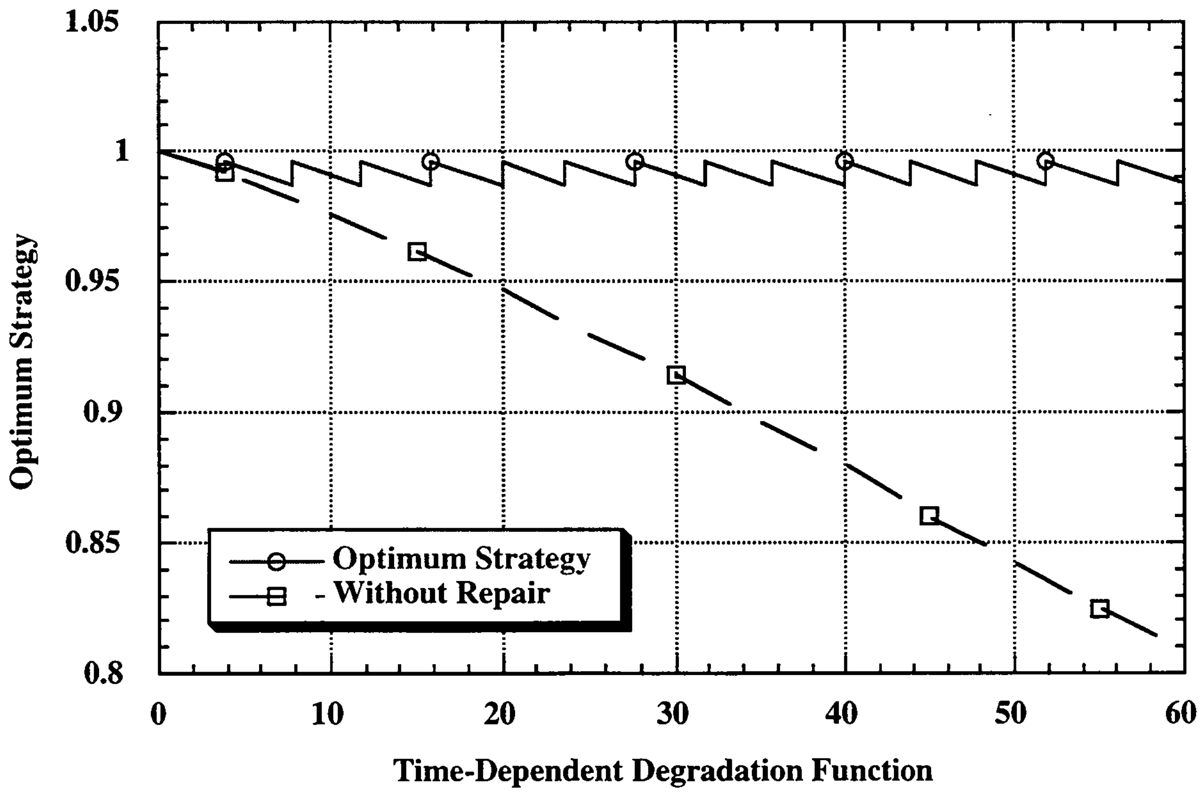


(b) Mean Degradation Function with Optimum Policy.

Fig. 5.15 Optimum solutions for full-inspection strategies for example 1 in Table 5.4.

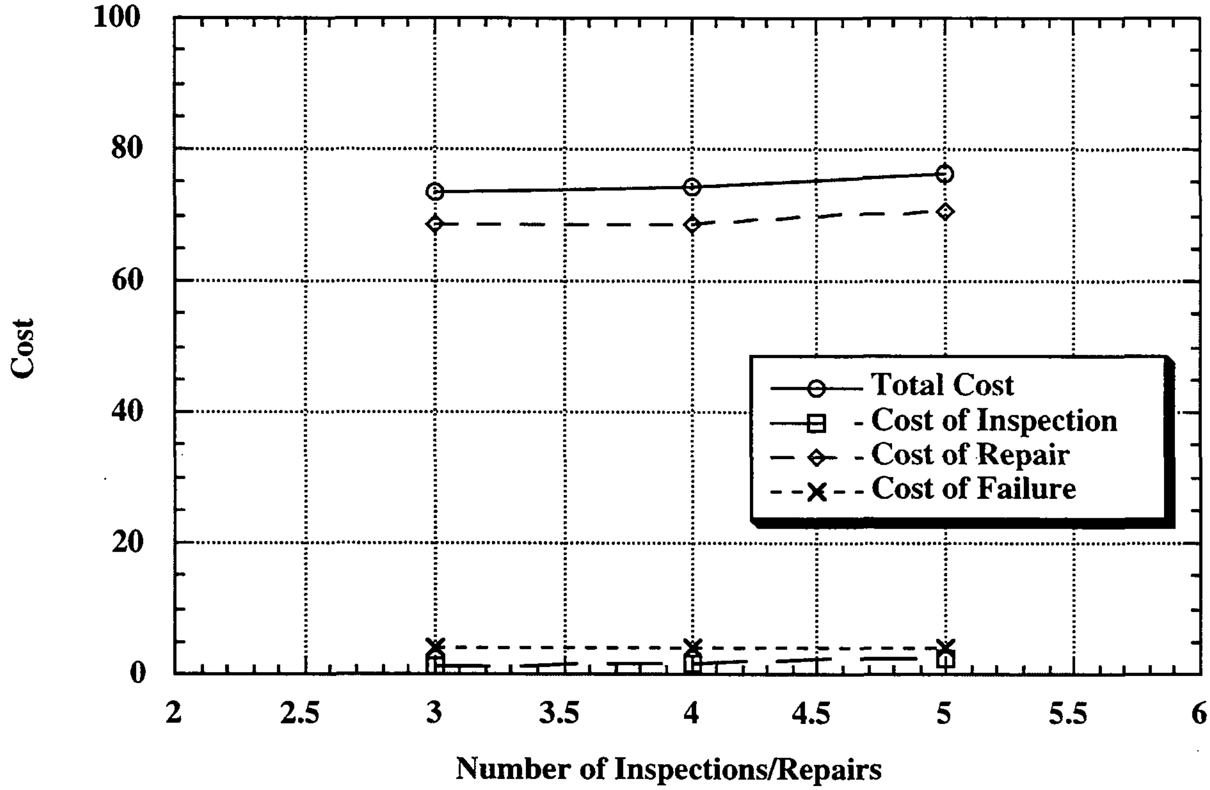


(a) Cost function at optimum solution.

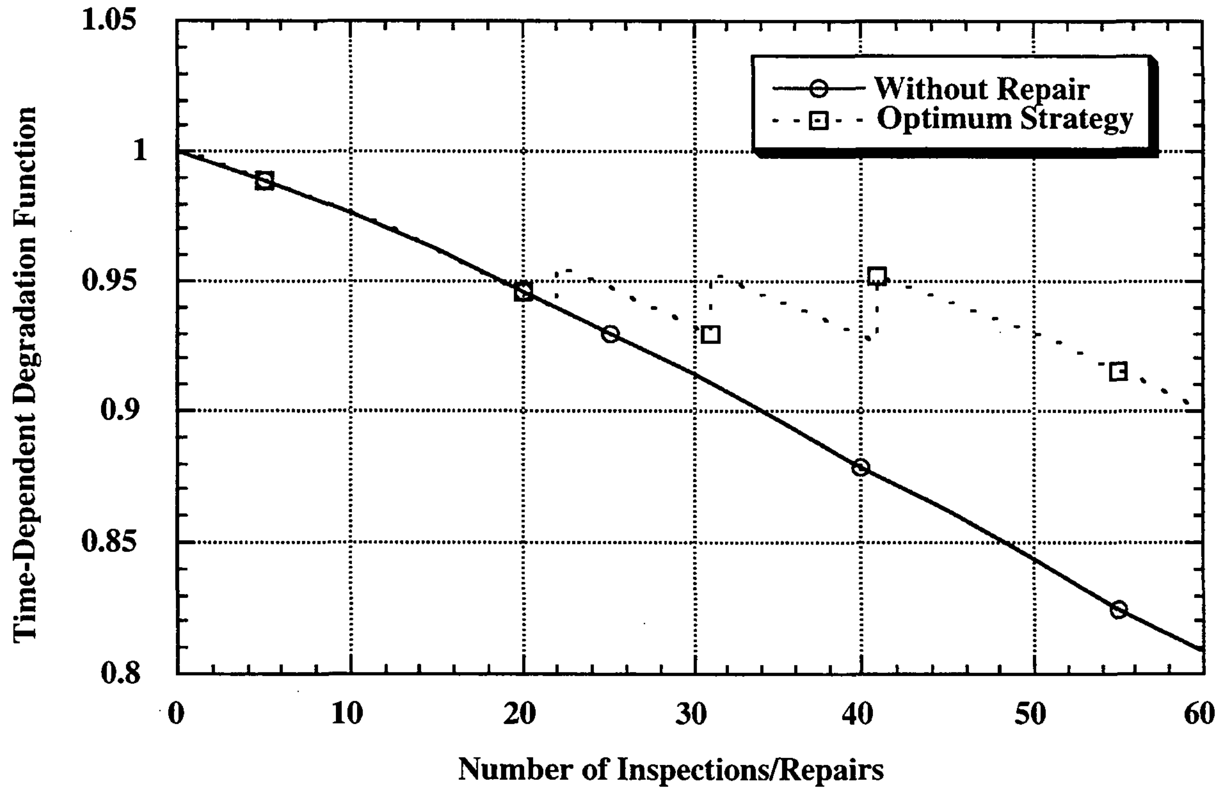


(b) Mean degradation function for with optimum policy.

Fig. 5.16 Optimum solutions for full-inspection strategies for example 2 in Table 5.4.



(a) Cost function at optimum solution.



(b) Mean degradation function with optimum policy.

Fig. 5.17 Optimum solutions for full-inspection strategies for example 3 in Table 5.4.

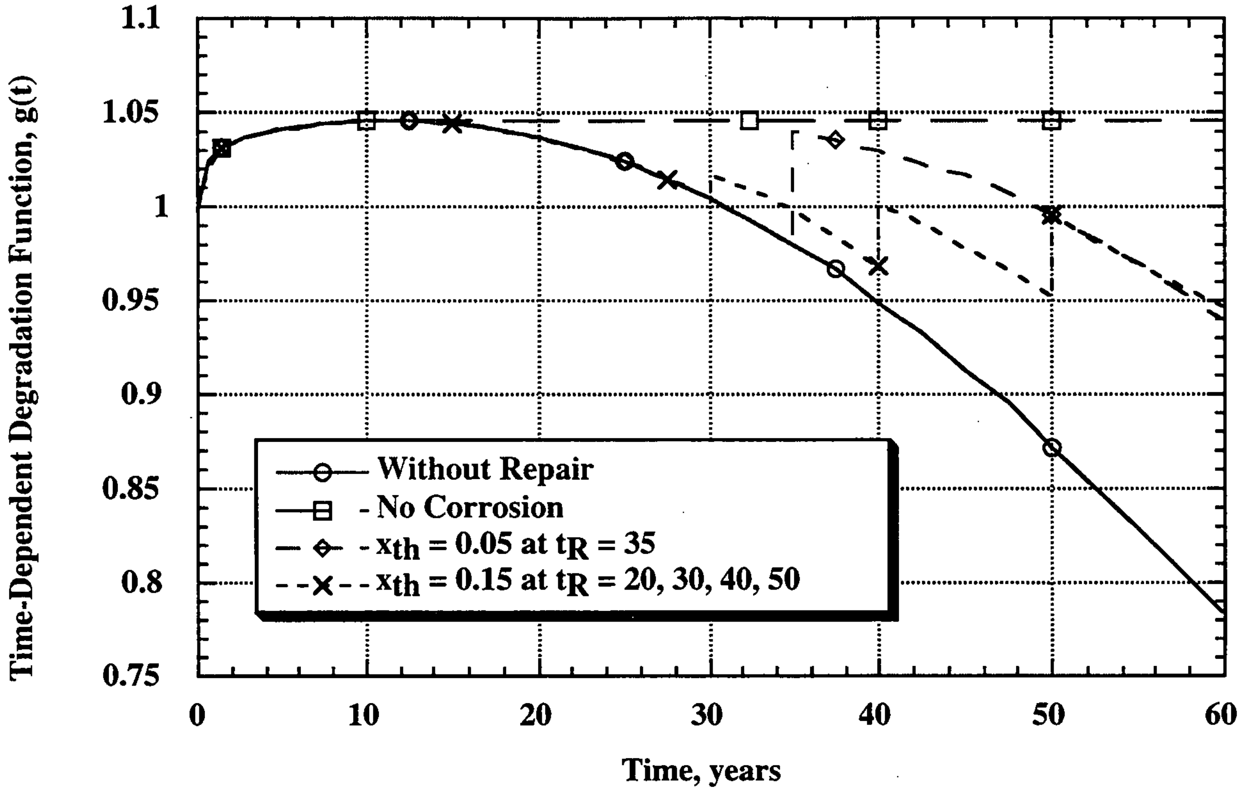


Fig. 5.18 Mean degradation function of slab subject to rebar corrosion.

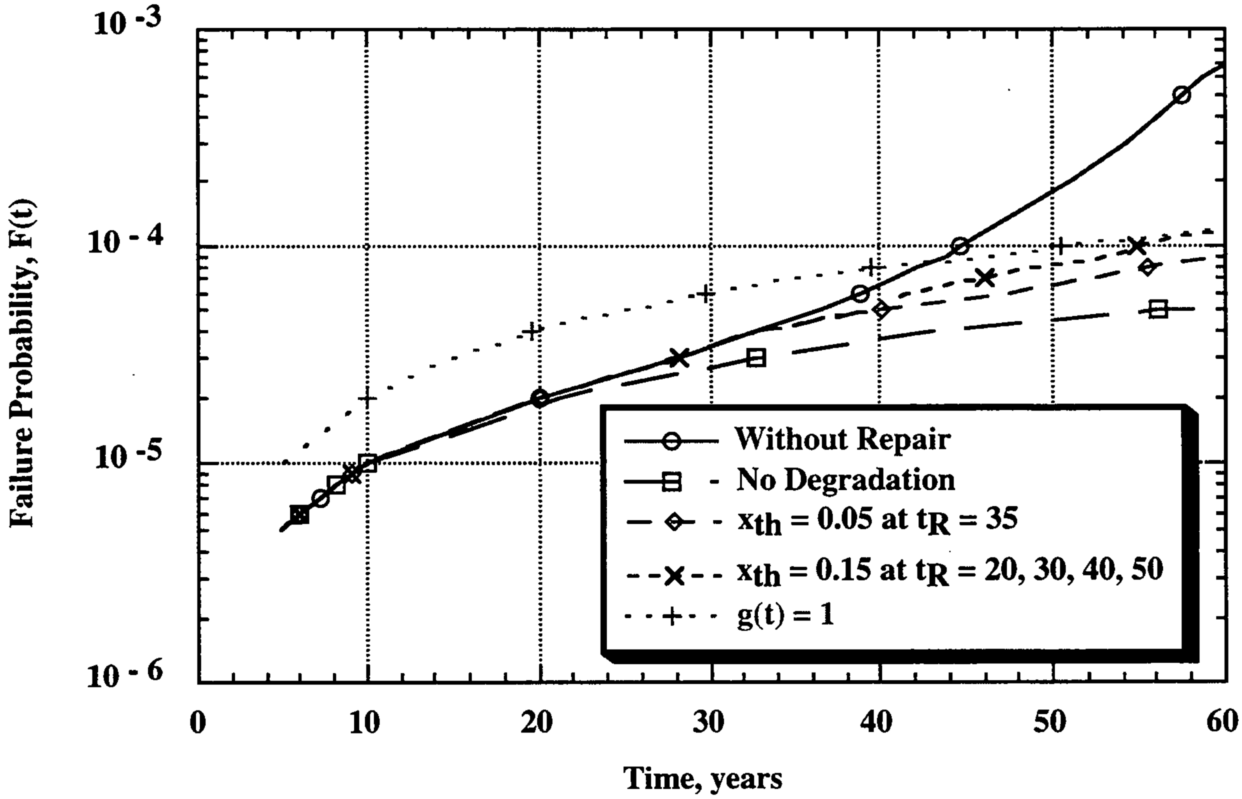
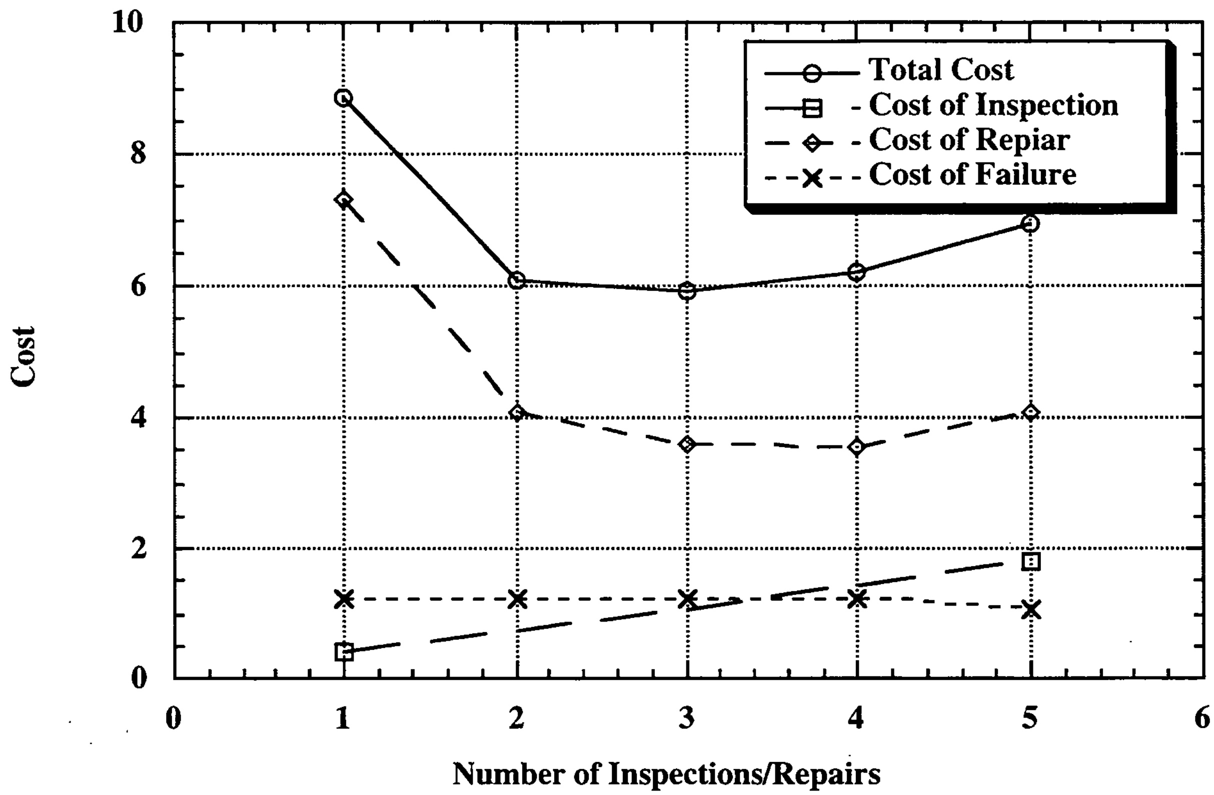
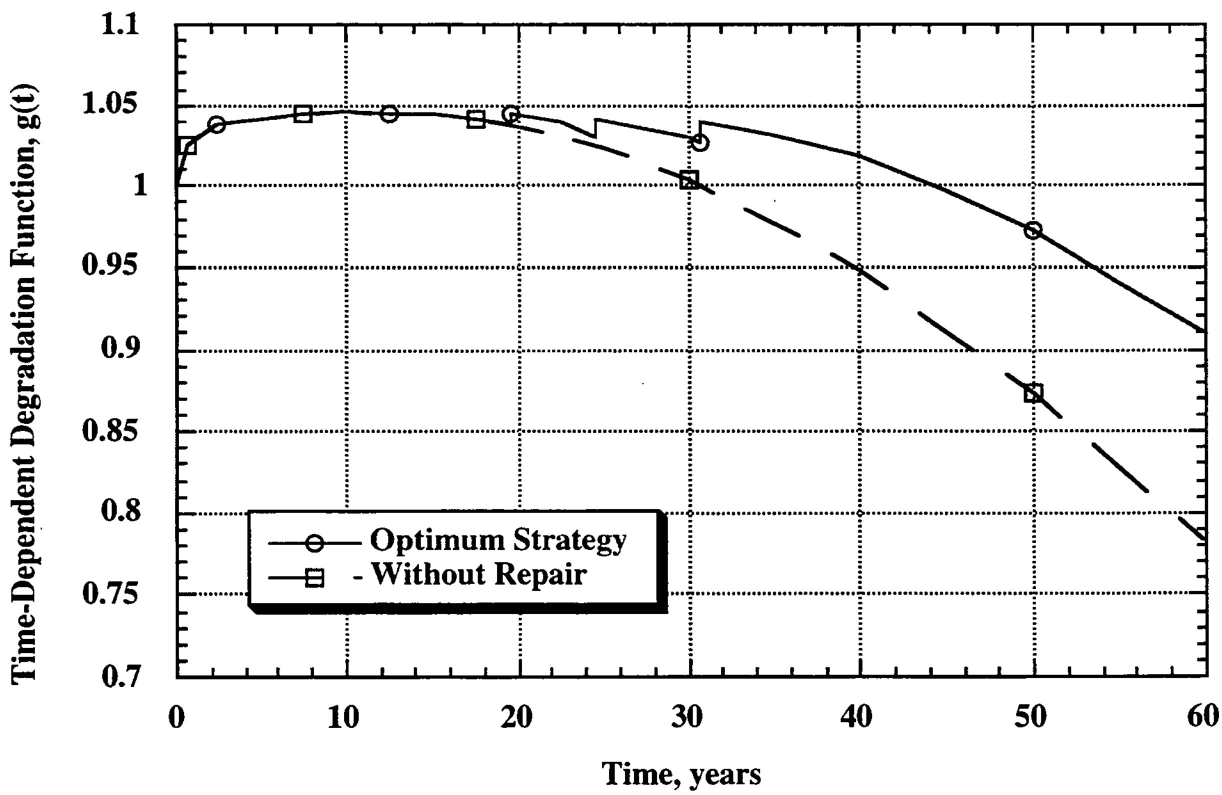


Fig. 5.19 Failure probability of slab with mean strength degradation in Fig. 5.18.



(a) Cost function at optimum solution.



(b) Mean degradation function with optimum policy.

Fig. 5.20 Optimum solutions for full-inspection strategies for slab.

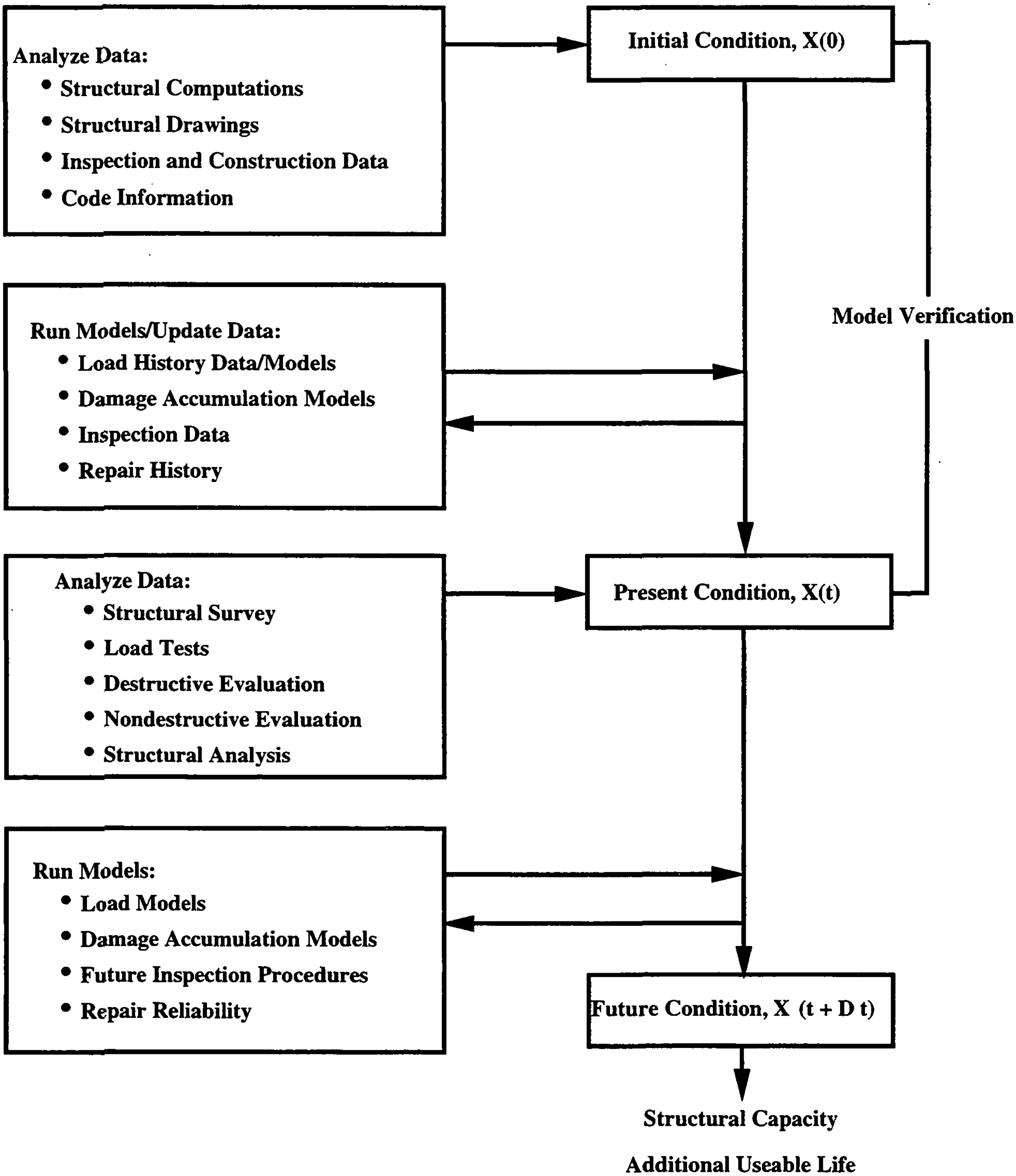


Fig. 5.21 System reliability approach to condition assessment.

6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY

The main objective of the Structural Aging (SAG) Program was to prepare a document that provides the U. S. Nuclear Regulatory Commission (USNRC) license reviewers with (1) identification and evaluation of the structural degradation processes; (2) issues to be addressed under NPP continued service reviews, as well as criteria, and their bases, for resolution of these issues; (3) identification and evaluation of relevant ISI or structural assessment programs in use, or needed; and (4) quantitative methodologies for assessing current, or estimating future structural safety margins. Results developed under the SAG Program and summarized in this document provide an improved basis for the USNRC staff to evaluate NPPs for continued service. 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 limited available inspection and surveillance data through development of a methodology that will enable the integrity of structures to be assessed, and (4) potential for improvements in damage inspection methodology through incorporation of program results into national standards [e.g., American Concrete Institute (ACI) 349][230] that could be referenced by Standard Review Plans.

Activities under the SAG Program were conducted under three technical task areas: (1) materials property data base, (2) structural component assessment and repair technologies, and (3) quantitative methodology for continued service determinations. Results provided in this report meet the objectives of the SAG Program noted above as well as provide a compendium of knowledge for use in developing a program for nuclear power plant (NPP) reinforced concrete structures to demonstrate that the effects of aging are being effectively managed. Where feasible, examples have been utilized to demonstrate pertinent aspects in several key areas of a life management program (e.g., condition assessments of NPP reinforced concrete structures, utilization of results contained in the Structural Materials Information Center (SMIC), and development of optimum ISI/repair strategies).

Primary SAG Program developments included the following

- Structural Materials Information Center containing data and information on the time variation of material properties under the influence of pertinent environmental stressors and aging factors for 144 materials;
- Aging assessment methodology that uses ranking criteria to identify structural components and degradation factors of primary importance to aging management;
- Guidelines and evaluation criteria for use in condition assessments of NPP reinforced concrete structures; and
- Reliability-based methodology for current condition assessments and estimations of future performance of NPP reinforced concrete structures.

In addition, the SAG Program conducted in-depth evaluations of several technologies. The primary purpose of these evaluations was to develop guidance on their applicability to NPP reinforced concrete structures. Reviews in the form of state-of-the-art reports, were provided on

- Data bases for concrete and concrete-related materials;

- In-service inspection (ISI) (destructive and nondestructive) and condition assessment techniques, and methodologies for their application;

- Corrosion of metals embedded in concrete, including criteria for use of cathodic protection systems and an assessment of the potential for occurrence of stray electrical current-induced corrosion; and

- Remedial measures strategies, repair materials and techniques, and performance characteristics (durability).

6.2 CONCLUSIONS

Throughout this report, conclusions are provided in the form of commentary. This commentary is provided at the end of each section that addresses a pertinent aspect related to aging management of NPP reinforced concrete structures. Summarized below are major conclusions that resulted from this program.

1. The performance of the reinforced concrete structures in NPPs has been good, with the majority of the identified problems initiating during construction and being corrected at that time. However, as these structures age, incidences of degradation due to environmental stressor effects are likely to increase to potentially threaten their durability. Items of note would be corrosion of steel reinforcement due to carbonation of the concrete or presence of chloride ions, excessive loss of prestressing force, leaching of concrete, and leakage of post-tensioning system corrosion inhibitor through cracks in the concrete.

2. Techniques for detecting the effects of environmental stressors are sufficiently developed to provide qualitative data. Areas of concern include massive members that contain large quantities of steel reinforcement such as the basemat and members that are inaccessible, such as portions of the steel pressure boundary that are embedded in concrete. Also, detectability functions that relate flaw characteristics to a probability of detection require development. Despite the limitations associated with many of the techniques, their proper use and application provides vital input for assessing the structural condition of reinforced concrete members. Frequently, increased confidence in results can be provided by using a combination of methods in tandem.

3. Methods for conducting condition assessments of reinforced concrete structures are fairly well established and generally start with a visual examination of the structure's surfaces. Condition assessments provide an effective aging management tool in that when a discontinuity is detected, a maintenance activity can be implemented to prevent the discontinuity from becoming a defect that requires a major repair. To be of most use, the condition assessments should be conducted at regular intervals. Established condition assessment methods, however, have been application specific (e.g., parking structure decks). Few standards or criteria are available for interpreting the results obtained from the condition assessments. Current inspection requirements for NPP reinforced concrete structures are fairly limited in that they only address the unbonded tendon systems in post-tensioned concrete containments and a general visual inspection in conjunction with the leak-rate tests. With the adoption of American Society of Mechanical Engineers (ASME) Section XI Subsection IWL, requirements will increase somewhat. Also, ACI Committee 349[230] has developed guidelines for inspection of safety-related

concrete structures other than containments. Due to the importance of condition assessments in effectively managing aging of structures and the likelihood for incidences of degradation to increase as the NPP structures age, it seems prudent that condition assessments of these structures should be conducted periodically. Structures identified to be of high safety significance and potentially at risk should receive the most detailed and frequent inspections. The inspection interval for future inspections could be increased based on a proven performance history. Structures having accessibility constraints for visual examinations or conduction of other non-destructive evaluations could start with an indirect approach such as monitoring the structure's ambient environment to determine if it is potentially aggressive. More detailed examinations would be required if the environment is found to be potentially aggressive.

4. Techniques for repair of concrete structures are well established and when properly selected and applied are effective. At present no codes or standards are available for repair of reinforced concrete structures, although some are being developed. Criteria that may be used to determine when a repair action should be implemented are not available (e.g., parameters that relate damage state such as crack width to environmental exposure). Data on the long-term effectiveness or durability of remedial measures is required. Effective implementation of a repair strategy requires knowledge of the degradation mechanisms, the environment of the structure at the macro and micro level, proper preconditioning of the structure to be repaired, correct choice of repair technique and material, and quality workmanship.

5. A reliability-based methodology has been developed that can be used to facilitate quantitative assessments of current and future structural reliability and performance of reinforced concrete structures in NPPs. The methodology is able to take into account the nature of past and future loads, and randomness in strength and in degradation resulting from environmental factors. The methodology can be used as a basis for selecting appropriate periods for continued service and/or determining optimum intervals and extent of inspection and repair activities. Inspection/repair strategies can be developed to minimize expected future cost while keeping the failure probability of the structure at or below an established target failure probability during its anticipated service period. Implementation and extension of the method to realistic condition assessments is difficult due to a lack of supporting quantitative data on strength degradation models, including initiation and rate of damage growth, the mean occurrence rate of local events and cumulative density function (cdf) of the intensity of time-varying loads. The reliability models established for condition assessments have not been validated through application to laboratory or prototypical structures.

6.3 RECOMMENDATIONS

A number of recommendations have been made throughout this report with respect to such things as use of existing data and service life models to estimate longevity of NPP concrete structures, capabilities of ISI techniques, methods for conducting condition assessments and criteria for interpretation of results, selection of repair materials and techniques, and development of ISI/repair strategies. Presented below is a summary of major recommendations for additional activities.

1. Durability assessments of reinforced concrete structures require an improved understanding of the degradation mechanisms, improved characterization of service environments, the development of advanced service life models, and the

development of guidelines and standards for acceptance of service life estimations. Information and data on the estimation of service life of in-service concrete should be investigated in more detail, particularly where two or more degradation factors may be occurring simultaneously.

2. The SMIC is the most comprehensive data base that has been developed to date for concrete and concrete-related materials. As more data and information become available as a result of activities related to continued service assessments of NPPs, it is recommended that this data continue to be incorporated into SMIC. Also, as SMIC has the capability to add other structural materials of importance to aging of NPPs, it is recommended that SMIC be expanded to include data and information on these materials. Furthermore, 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, it is recommended that the data base be completely redesigned to address the shortcomings of the current data base management system.

3. Nondestructive evaluation techniques were found in large measure to be more qualitative than quantitative and require the development of correlation curves for interpretation of results. Detectability functions for these techniques are not presently available. Developments in non-destructive evaluation techniques are required with respect to two specific areas related to inspection of NPP reinforced concrete structures. No technique was found to be capable of providing reliable information when inspecting massive, heavily-reinforced concrete structures such as basemats. Non-destructive evaluation techniques capable of inspecting inaccessible regions of the pressure boundary such as where it is embedded in concrete also require development.

4. Post-tensioning systems should be investigated in more detail to investigate the cause(s) of larger than estimated losses of prestressing force that are being experienced by several of the older prestressed concrete containments. The significance of leakage of corrosion inhibitor that is occurring through cracks in the concrete of several of the prestressed concrete containments should be investigated with respect to its potential effects on the properties of the concrete and structural performance of the containment. Results of evaluation of a limited data set indicate that the use of lift-off loads to indicate prestressing forces in the concrete containments may overestimate the actual prestressing forces. This should be investigated in more detail to assess the significance of this difference, particularly with respect to long-term aging considerations.

5. Although basic approaches are available for conducting condition assessments of reinforced concrete structures, information is required for use in interpretation of results. Criteria for interpretation of crack parameters (e.g., width, depth, and length) with respect to environmental factors and the impact of observed degradation on structural performance in large measure, require development. Additional guidance should be developed that provides acceptance criteria for use in conjunction with condition assessments of NPP reinforced concrete structures. In association with the condition assessments, more definitive guidelines are required for application of repair strategies that cover the entire repair spectrum (i.e., repair requirements, selection of repair materials and techniques, preparation of the

structure, application of repair methods, and curing). Additional information is required with respect to methods for use in evaluation of the effectiveness of a remedial measure, and durability characteristics of the various methods.

6. Several areas should be pursued in more detail with respect to the time-dependent reliability analysis. Additional data should be developed so that the method can be extended to make more realistic condition assessments (e.g., strength degradation models; mean occurrence rate of load event and cumulative distribution function of the intensity of time-varying loads; threshold level of defect detection for nondestructive evaluation methods; and costs of inspection, repair, and loss due to structural failure). A damage model reflecting interactions between degradation mechanisms is required for use with the time-dependent reliability analysis to evaluate the failure probability of a component. The reliability models developed should be evaluated through application to laboratory and prototypical structures. Also, user-oriented guidelines for use in the condition assessment and life estimation methodology should be developed.

7. Finally, the effect of aging on the structural margins of safety-related reinforced concrete structures such as containments should be evaluated.

7. REFERENCES

1. *Nuclear Regulatory Commission Information Digest*, NUREG-1350, **Vol. 7**, Division of Budget and Analysis, Office of the Controller, United States Nuclear Regulatory Commission, Washington, D.C., March 1995.

2. H. J. C. Kouts, "Aging Nuclear Plants," pp. 39–41 in *Nuclear Plant Journal*, **13(1)**, January–February 1995.

3. D. G. Eisenhut and F. T. Stetson, "Regulatory Implications of Plant Life Extension," pp. 253–258 in *Transactions of the 9th International Conference on Structural Mechanics in Reactor Technology*, **Vol. D**, A. A. Balkema (publisher), Boston, Massachusetts, 1987.

4. *Code of Federal Regulations*, Title 10–Energy, Office of the Federal Register, National Archives and Records Administration, Washington, D.C., January 1, 1995.

5. *PWR Pilot Plant Life Extension Study at Surry Unit 1: Phase 2*, EPRI NP-6232M, Electric Power Research Institute, Palo Alto, California, 1989.

6. *BWR Pilot Plant Life Extension Study at the Monticello Plant: Interim Phase 2*, EPRI NP-5836M, Electric Power Research Institute, Palo Alto, California, 1988.

7. American Nuclear Society, *Proceedings of the Topical Meeting on Plant License Renewal held June 4–6, 1991 in Orlando, Florida*, American Nuclear Society, La Grange Park, Illinois, 1991.

8. American Society of Mechanical Engineers, "Plant Systems/Components Aging Management-1993," *Proceedings of the Pressure Vessels and Piping Conference held July 25–29, 1993 in Denver, Colorado*, 1993.

9. J. O. Thoma, "Regulatory Guidance for License Renewal," *Proceedings of Topical Meeting on Plant License Renewal held June 4–6, 1991 in Orlando, Florida*, American Nuclear Society, La Grange Park, Illinois, 1991.

10. B. M. Morris and J. P. Vora, *Nuclear Plant Aging Research (NPAR) Program Plan*, NUREG-1144, U.S. Nuclear Regulatory Commission, Washington, D.C., 1985.

11. J. P. Vora, *NRC Research Program on Plant Aging: Listing and Summaries of Reports Issued Through September 1993*, NUREG-1377, Rev. 4, U.S. Nuclear Regulatory Commission, Washington, D.C., December 1993.

12. *Nuclear Plant Aging Research (NPAR) Program Plan – Status and Accomplishments*, NUREG-1144, Rev. 2, U.S. Nuclear Regulatory Commission, Washington, D.C., June 1991.

13. D. J. Naus, *Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants*, NUREG/CR-4652 (ORNL/TM-10059), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1986.

14. *Seismic Design Classification*, Regulatory Guide 1.29, U.S. Atomic Energy Commission, Washington, D.C., June 1972.

15. "Concrete and Steel Internal Structures of Steel and Concrete Containments," Sect. 3.8.3 in *Regulatory Standard Review Plan*, NUREG-0800, Directorate of Licensing, U.S. Nuclear Regulatory Commission, Washington, D.C., June 1981.

16. ACI Committee 318, *Building Code Requirements for Reinforced Concrete*, ACI Standard 318-71, American Concrete Institute, Detroit, Michigan, November 1971.

17. ACI Committee 349, *Code Requirements for Nuclear Safety Related Concrete Structures*, ACI Standard 349-85, American Concrete Institute, Detroit, Michigan, June 1985.

18. *Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments) (for comment issue)*, Regulatory Guide 1.142, U.S. Nuclear Regulatory Commission, Washington, D.C., April 1978.

19. *Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)*, Regulatory Guide 1.70 (Rev. 3), Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., November 1978.

20. Joint Technical Committee of ACI and ASME, "Code for Concrete Reactor Vessels and Containments," Sect. III, Division 2 of the *ASME Boiler and Pressure Vessel Code* (ACI Standard 359-77), American Society of Mechanical Engineers, New York, New York, 1977.

21. "Concrete Containment," Sect. 3.8.1 in *Regulatory Standard Review Plan*, NUREG-0800, Directorate of Licensing, U.S. Nuclear Regulatory Commission, Washington, D.C., June 1981.

22. T. Lo et al., "Containment Integrity of SEP Plants Under Combined Loads," *Proceedings of the ASCE Conference on Structural Engineering in Nuclear Facilities*, J. Ucciferro (ed.), American Society of Civil Engineers, New York, New York, September 1984.

23. *Class I Structures License Renewal Industry Report: Revision 1*, EPRI TR-103842 (NUMARC 90-06), Electric Power Research Institute, Palo Alto, California, July 1994.

24. C. J. Hookham and V. N. Shah, *Insights for Aging Management of Major Light Water Reactor Components, Vol. 2 – Reinforced and Prestressed Concrete Containments*, NUREG/CR-5314 (EGG-2562), Idaho National Engineering Laboratory, Idaho Falls, Idaho, July 1994 (draft).

25. *PWR Containment Structures License Renewal Industry Report: Revision 1*, EPRI TR-103835 (NUMARC 90-01), Electric Power Research Institute, Palo Alto, California, July 1994.

26. V. N. Shah et al., "Boiling Water Reactor Containments," Chapter 9 in *Residual Life Assessment of Major Light Water Reactor Components-Overview Volume 2*, NUREG/CR-4731 (EGG-2469, **Vol. 2**), Idaho National Engineering Laboratory, Idaho Falls, Idaho, November 1989.

27. F. E. Gregor and C. J. Hookham, "Remant Life Preservation of LWR Plant Structures," pp. 453–459 in *Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology held August 15–20, 1993, in Stuttgart, Germany*, Paper DH06/2, Elsevier Science Publishers, Amsterdam, The Netherlands, 1993.

28. "Standard Specification for Portland Cement," *Annual Book of ASTM Standards*, ASTM C150, Part 14, American Society of Testing and Materials, Philadelphia, Pennsylvania, 1979.

29. G. Troxell et al., *Composition and Properties of Concrete*, McGraw-Hill, New York, New York, 1968.

30. "Standard Specification for Structural Steel," *Annual Book of ASTM Standards*, ASTM A36/A 36M, **Vol. 01.04**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1989.

31. "Deformed and Plain Billet-Steel Bars for Concrete Reinforcement," *Annual Book of ASTM Standards*, ASTM A615, Ibid.

32. "Low-Alloy Steel Deformed Bars for Concrete Reinforcements," *Annual Book of ASTM Standards*, ASTM A706, Ibid.

33. "Deformed Billet Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Strength (superseded)," ASTM A432, Ibid.

34. "Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement (superseded)," ASTM A 305, *1968 Book of ASTM Standards, Part 4: Structural Steel; Concrete Reinforcing Steel; Boiler and Pressure Vessel Plate; Steel Rails, Wheels, and Tires; Bearing Steel; Steel Forgings; and Ferrous Filler Metal*, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1968.

35. "Steel Welded Wire Fabric, Plain, for Concrete Reinforcement," *Annual Book of ASTM Standards*, ANSI/ASTM A 185, **Vol. 01.04**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1989.

36. "Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement," *Annual Book of ASTM Standards*, ASTM A 497, Ibid.

37. "Steel Wire, Deformed, for Concrete Reinforcements," *Annual Book of ASTM Standards*, ASTM A 496, Ibid.

38. "Fabricated Deformed Steel Bar Mats for Concrete Reinforcement," *Annual Book of ASTM Standards*, ASTM A 184, Ibid.

39. "Uncoated Stress-Relieved Steel Wire for Prestressed concrete," *Annual Book of ASTM Standards*, ASTM A 421, Ibid.

40. "Steel Strand, Uncoated Seven Wire Stress-Relieved for Prestressed Concrete," *Annual Book of ASTM Standards*, ASTM A 416, Ibid.

41. "Uncoated High-Strength Steel Bar for Prestressing Concrete," *Annual Book of ASTM Standards*, ASTM A 722, Ibid.

42. "Steel Bars, Cold Finished, Standard Quality, Grades 1010 through 1020," *Annual Book of ASTM Standards*, ASTM A 108, **Vol. 01.05**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1989.

43. "Pressure Vessel Plates, Carbon Steel, for Moderate- and Lower-Temperature Service," *Annual Book of ASTM Standards*, ASTM A 516, **Vol. 01.04**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1989.

44. "Standard Specification for Stainless and Heat-Resisting Steel Bars and Shapes," *Annual Book of ASTM Standards*, ASTM A 276, **Vol. 01.05**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1986.

45. "Specification for Alloy Steel Bars Subject to End-Quench Hardenability Requirements," *Annual Book of ASTM Standards*, ASTM A 304, Ibid.

46. "Standard Specification for Pressure Vessel Plates, Carbon Steel, Low- and Intermediate-Tensile Strength," *Annual Book of ASTM Standards*, ASTM A 285, **Vol. 01.04**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1986.

47. "Standard Specification for Pressure Vessel Plates, Heat-Treated, Carbon-Managanese-Silicon Steel," *Annual Book of ASTM Standards*, ASTM A 537, Ibid.

48. "Standard Specification for Carbon Steel Externally Threaded Standard Fasteners," *Annual Book of ASTM Standards*, ASTM A 307, **Vol. 01.01**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1985.

49. "Standard Specification for High Strength Bolts for Structural Joints," *Annual Book of ASTM Standards*, ASTM A 325, **Vol. 15.08**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1986.

50. "Standard Specification for Heat-Treated, Steel Bolts, 150 ksi (1035 MPa) Tensile Strength," *Annual Book of ASTM Standards*, ASTM A 490, Ibid.

51. B. Mather, "Concrete Need Not Deteriorate," pp. 32–37 in *Journal of American Concrete Institute*, **Vol. 1(9)**, Detroit, Michigan, September 1979.

52. *Durable Concrete Structures — Design Guide*, Comite Euro-International de Beton, Thomas Telford Service Publisher, London, England, 1992.

53. J. H. Willenbrock et al., *Final Summary Report : A Comparative Analysis of Structural Concrete Quality Assurance Practices of Nine Nuclear and Three Fossil Fuel Plant Construction Projects*, Report No. C00/4120-3, Department of Civil Engineering, The Pennsylvania State University, University Park, Pennsylvania, December 1978.

54. J. Fraczek, "ACI Survey of Concrete Structure Errors," pp. 14–20 in *Journal of American Concrete Institute*, **Vol. 1(12)**, Detroit, Michigan, December 1979.

55. R. Hauser, "Lessons Learned from European Failures," pp. 21–25 in *Journal of American Concrete Institute*, **Vol. 1(12)**, Detroit, Michigan, December 1979.

56. P. D. Krauss, *Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants*, ORNL/NRC/LTR-93/28, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, Mary 1994.

57. *Non-Structural Cracks in Concrete*, Technical Report 22, Concrete Society, London, England, 1982.

58. RILEM Committee 104-DDC, "Draft Recommendations for Damage Classification of Concrete Structures," pp. 362–369 in *Materials and Structures*, **Vol. 27**, International Union of Testing and Research Laboratories for Materials and Structures, Cachan, France, 1994.

59. P. K. Mehta, *Concrete-Structure, Properties, and Materials*, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1986.

60. P. K. Mehta and B. C. Gerwick, Jr., "Cracking-Corrosion Interaction in Concrete Exposed to Marine Environment," pp. 45–51 in *Concrete International*, **4**(10), October 1982.

61. "Structural Use of Concrete: Code of Practice for Design and Construction," *British Standard 8110*, Part 1, Table 14.1, London, England, 1985.

62. ACI Committee 515, "A Guide for the Use of Waterproofing Dampproofing, Protective, and Decorative Systems for Concrete," pp. 515.1R-6 to 515.1R-11 in *ACI Manual of Concrete Practice–Masonry, Precast Concrete, Special Processes*, Table 2.5.2 of Part 5, American Concrete Institute, Detroit, Michigan, 1990.

63. *Alkalies in Concrete*, ASTM STP 930, V. H. Dodson (ed.), American Society for Testing and Materials, Philadelphia, Pennsylvania, 1986.

64. S. Mindess and J. F. Young, *Concrete*, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1981.

65. *Proposed Test Method for Accelerated Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction*, ASTM P 214, **Vol. 04.02**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1991.

66. *Standard Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)*, ASTM C 586, Ibid.

67. ACI Committee 201, "Guide to Durable Concrete," pp. 573–609 in *Journal of American Concrete Institute*, **74**(12), Detroit, Michigan, December 1977.

68. *Standard Test Method for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilatation Procedures*, ASTM C 682, **Vol. 04.02**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1991.

69. *Standard Specification for Concrete Aggregates*, ASTM C 33, Ibid.

70. ACI Committee 210, *Erosion of Concrete Hydraulic Structures*, ACI 210R-89, American Concrete Institute, Detroit, Michigan, 1989.

71. Z. P. Bazant et al., *Normal and Refractory Concretes for LMFRB Applications — Vol. 1: Review of Literature on High-Temperature Behavior of Portland Cement and Refractory Concrete*, EPRI NP-2437, Electric Power Research Institute, Palo Alto, California, June 1982.

72. G. N. Freskakis et al., "Strength Properties of Concrete at Elevated Temperature," *Civil Engineering Nuclear Power*, **Vol. 1**, ASCE National Convention, Boston, April 1979.

73. T. Uddin and C. G. Culver, "Effects of Elevated Temperature on Structural Materials," *J. Str. Div.*, **101(7)**, American Society of Civil Engineers, New York, New York, 1975.

74. H. L. Malhotra, "The Effect of Temperature on Compressive Strength of Concrete," pp. 85–94 in *Mag. of Concrete Res.*, **8(23)**, London, England, August 1956.

75. N. Zoldners, "The Effect of High Temperature on Concrete Incorporating Different Aggregates," pp. 1087–1108 in *Proc. ASTM 60*, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1960.

76. M. S. Abrams, "Compressive Strength of Concrete at Temperatures to 1600°F," pp. 33–58 in *Temperature and Concrete*, Paper SP 25-2 in SP-25, American Concrete Institute, Detroit, Michigan, 1971.

77. C. R. Cruz, "Elastic Properties of Concrete at High Temperature," pp. 37–45 in *J. PCA – Research and Development Laboratories*, **8(1)**, Portland Cement Association, Skokie, Illinois, January 1966.

78. ACI Committee 349, *Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Concrete Structures*, ACI 349.1R-80, American Concrete Institute, Detroit, Michigan, 1980.

79. ACI Committee 216, *Guide for Determining the Fire Resistance of Concrete Elements*, ACI 216R-81, American Concrete Institute, Detroit, Michigan, 1981.

80. *Fire Safety of Concrete Structures*, SP-80, American Concrete Institute, Detroit, Michigan, 1983.

81. American Nuclear Society, *Guidelines on the Nuclear Analysis and Design of Concrete Radiation Shielding for Nuclear Power Plants*, American National Standard, ANSI/ANS-6.4-1985, La Grange Park, Illlinois, 1985.

82. H. K. Hilsdorf et al., *The Effects of Nuclear Radiation on the Mechanical Properties of Concrete*, ACI SP-55, Douglas McHenry International Symposium on Concrete and Concrete Structures, American Concrete Institute, Detroit, Michigan, 1978.

83. Science Applications, Inc., *Study of Radiation Dosage to Structural Components in Nuclear Reactors*, EPRI NP-152, Electric Power Research Institute, Palo Alto, California, 1977.

84. M. F. Kaplan, *Concrete Radiation Shielding,* Concrete Design & Construction Series, Longman Scientific & Technical, Essex, England, 1989.

85. ACI Committee 215, *Considerations for Design of Concrete Structures Subjected to Fatigue*, ACI 215R-74, American Concrete Institute, Detroit, Michigan, 1974.

86. S. P. Shah (ed.), *Fatigue of Concrete Structures*, SP-75, American Concrete Institute, Detroit, Michigan, 1982.

87. ACI Committee 222, *Corrosion of Metals in Concrete*, ACI 222R-89, American Concrete Institute, Detroit, Michigan, 1989.

88. W. Swiat et al., *State-of-the-Art Report — Corrosion of Steel in Concrete*, ORNL/NRC/LTR-93/2, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, May 1993.

89. U. Schneider et al., "Effect of Temperature on Steel and Concrete for PCPV's," pp. 245–258 in *Nuclear Engineering and Design*, **67**, 1981.

90. P. Smith, "Resistance to High Temperatures," STP 169B, *Significance of Tests and Properties of Concrete and Concrete-Making Materials*, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1978.

91. M. S. Abrams and C. R. Cruz, *The Behavior at High Temperature of Steel Strand for Prestressed Concrete*, Research Department Bulletin No. 134, Portland Cement Association, Skokie, Illinois, 1961.

92. A. Cowen and R. W. Nichols, "Effect of Irradiation on Steels Used in Pressure Vessels," pp. 229–235 in *Prestressed Concrete Pressure Vessels*, Group D, Paper 20, Marilyn S. Udell (ed.), The Institute of Civil Engineers, London, England, 1968.

93. W. H. Hartt, "A Critical Evaluation of Cathodic Protection for Prestressing Steel in Concrete," pp. 515–524 in *Proceedings of The Third International Symposium on Corrosion of Reinforcement in Concrete Construction* held at Belfry Hotel in Wishaw, Warwickshire, United Kingdom, on May 21–24, 1990, Elsevier Applied Science, New York, New York, 1990.

94. J. Wagner, Jr., et al., *Cathodic Protection Developments for Prestressed Concrete Components*, Report No. FHWA-RD-92-056, Federal Highway Administration, Washington, D.C., March 1992.

95. R. F. Stratfull, "Criteria for the Cathodic Protection of Bridge Decks," Chapter 18 in *Corrosion of Reinforcement in Concrete Construction*, Society of Chemical Industry, London, England, 1983.

96. W. Podolny, Jr. and T. Melville, "Understanding the Relaxation in Prestressing," *Prestressed Concrete Institute Journal*, **14(4)**, Chicago, Illinois, August 1969.

97. T. Z. Harmathy and W. W. Stanzack, *Elevated Temperature Tensile and Creep Properties for Some Structural and Prestressing Steels*, STP 464, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1970.

98. T. Cahill and G. D. Branch, "Long-Term Relaxation Behavior of Stabilized Prestressing Wires and Strands," Group D, Paper 19, pp. 219–228 in *Prestressed Concrete Pressure Vessels*, Marilyn S. Udell (ed.), The Institute of Civil Engineers, London, England, 1968.

99. S. Freedman, "Properties of Materials for Reinforced Concrete," *Handbook of Concrete Engineering*, M. Fintel (ed.), Von Nostrand Reinhold Company, New York, New York, 1974.

100. J. R. Libby, *Modern Prestressed Concrete-Design Principles and Construction Methods*, Von Nostrand Reinhold Company, New York, New York, 1971.

101. J. A. Hanson, *Prestress Loss as Affected by Type of Curing*, Bulletin D75, Portland Cement Association, Skokie, Illinois, 1964.

102. W. Podolny, Jr. and T. Melville, "Understanding the Relaxation in Prestressing," *Journal of the Prestressed Concrete Institute*, **14**(4), Chicago, Illinois, 1969.

103. W. Podolny, Jr., "Understanding the Losses in Prestressing," *Journal of the Prestressed Concrete Institute*, **14**(5), Chicago, Illinois, 1969.

104. PCI Committee on Prestress Losses, "Recommendations for Estimating Prestress Losses," *Prestressed Concrete Institute Journal*, **20**(5), Chicago, Illinois, 1975.

105. "Inservice Inspections of Ungrouted Tendons in Prestressed Concrete Containment Structures," *Regulatory Guide 1.35*, Rev. 3, Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., July 1990.

106. "Determining Prestressing Forces for Inspection of Prestressed Concrete Containments," *Regulatory Guide 1.35.1*, Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., July 1990.

107. *Metals Handbook*, ASM International, Ninth Edition, **Vol. 13**, Metals Park, Ohio, 1987.

108. M. Romanoff, *Corrosion of Steel Pilings in Soils*, Monograph 58, National Bureau of Standards, Gaithersburg, Maryland, October 1962.

109. R. Peck et al., *Foundation Engineering*, Wiley and Sons, New York, New York, 1974.

110. D. F. Meinheit and F. D. Heidbrink, "Behavior of Drilled-In Expansion Anchors," pp. 62–66 in *Concrete Interational*, **7(4)**, April 1985.

111. "Standard Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials," *Annual Book of ASTM Standards*, ASTM E 632, **Vol. 04.07**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1990.

112. J. R. Clifton, "Methods for Predicting Service Lives of Concrete," pp. 361–373 in *Durability of Building Materials and Components*, Proceedings of Fifth International Conference, E. & F. N. Spon, London, England, 1990.

113. J. R. Clifton, *Predicting the Remaining Service Life of Concrete*, NISTIR 4712, U.S. Department of Commerce, National Institute of Standards and Technology, Gaithersburg, Maryland, November 1991 (report prepared under Structural Aging Program).

114. E. Vesikari, "Service Life Design of Concrete Structures with Regard to Frost Resistance of Concrete," pp. 215–228 in *Nordic Concrete Research*, Publication No. 5, Norske Betongforcning, Oslo, Norway, 1986.

115. G. L. Kalousek et al., "Concrete for Long-Term Service in Sulfate Environment," pp. 79–90 in *Cement and Concrete Research*, **2**(1), 1972.

116. E. Vesikari, *Service Life of Concrete Structures with Regard to Corrosion of Reinforcement,* Research Report 553, Technical Research Centre of Finland, Espoo, 1988.

117. J. Martin, "Service Life Predictions from Accelerated Aging Tests Using Reliability Theory and Life Testing Analysis," pp. 191–212 in *Problems in Service Life Prediction of Building and Construction Materials*, L. W. Master (ed.), Martinus Nijhoff Publishers, Boston, Massachusetts, 1985.

118. L. Sentler, "Stochastic Characterization of Carbonation of Concrete," pp. 569–580 in *3rd International Conference on the Durability of Building Materials and Components*, Technical Research Centre of Finland, Espoo, 1984.

119. J. R. Clifton and L. I. Knab, *Service Life of Concrete*, NISTIR 89-4086, Department of Commerce, National Institute of Standards and Technology, Gaithersburg, Maryland, 1989.

120. J. Pommersheim and J. R. Clifton, "Prediction of Concrete Service Life," pp. 21–30 in *Materials and Construction*, **18**, 1985.

121. J. C. Walton et al., *Models for Estimation of Service Life of Concrete Barriers in Low-Level Radioactive Waste Disposal*, NUREG/CR-5542, U.S. Nuclear Regulatory Commission, Washington, D.C., 1990.

122. J. Pommersheim and J. R. Clifton, *Models of Transport Processes in Concrete*, NISTIR 4405, Department of Commerce, National Institute of Standards and Technology, Gaithersburg, Maryland, 1990.

123. J. Hartley and P. B. Bamforth, *Collation of Survey Data and Review of the Durability Assessment of Reinforced Concrete Structures at Nuclear Power Stations in the U.K.*, Report 1303/92/6163, Taywood Engineering Ltd., R. & D. Division, London, England, April 1993 1991 (report prepared under Structural Aging Program).

124. *Deterioration Prevention in Reinforced Concrete Structures Subject to Hostile Environments*, Report 1303/41/5781 (BRITE Project 1552), Taywood Engineering Ltd., R. & D. Division, London, England, 1991.

125. D. C. Pocock and P. B. Bamforth, "Cost Effective Cures for Reinforcement Corrosion — Collaborative Research Brings New Developments," pp. 7–10 in *Construction Maintenance and Repair*, Nov./Dec. 1991.

126. E. Hognestad, "Design of Concrete for Service Life," *Concrete International*, **8**(6), American Concrete Institute, Detroit, Michigan, June 1986.

127. T. P. Tassios, "Report of Discussion Group on Inorganic Materials," in *Problems in Service Life Prediction of Building and Construction Materials*, L. Masters (ed.), Martinus Nihoff Publishers, Boston, Massachusetts, 1985.

128. G. Frohnsdorff and L. W. Masters, "Suggestions for a Logically Consistent Structure for Service Life Standards," pp. 113–123 in *Proceedings of the Fifth International Conference on Building Materials and Components* held in Brighton, England, on November 7–9, 1990.

129. I. Biczok, *Concrete Corrosion – Concrete Protection*, Akademiai Kiado Publishing House of the Hungarian Academy of Sciences, Budapest, Hungary, 1972.

130. G. Fagerlund, "Predicting the Service Life of Concrete Structures," presented at the *Second Engineering Foundation Conference Related to Cement and Concrete*, published by United Engineering Trustees, Inc., New York, New York, 1983.

131. S. Pihlajavaara, "A Review of Some of the Main Results of a Research on Aging Phenomena in Concrete: Effect of Moisture Content on Strength, Shrinkage, and Creep of Mature Concrete," pp. 761–777 in *Cement and Concrete Research*, **4**(5), September 1974.

132. P. K. Mehta (ed.), *International Experience with Durability of Concrete in Marine Environment*, Department of Civil Engineering, University of California, Berkeley, January 1989.

133. L. G. Mallinson and I. Ll. Davies, *A Historical Examination of Concrete*, EUR 10937 EN, Commission of European Communities, Luxembourg, 1987.

134. G. W. Washa, J. C. Seamann, and S. M. Cramer, "Fifty Year Properties of Concrete," pp. 367–371 in *Materials Journal*, **86**(4), American Concrete Institute, Detroit, Michigan, July–August 1989.

135. R. E. Weyers et al., *Concrete Deterioration Inspection System for Extending the Operating Life of Nuclear Power Plants Applied to Surry Unit One*, Charles E. Via, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, Virginia, February 1988.

136. P. Zwicky and D. Kluge, "Aging Management for Safety Related Concrete Structures in Switzerland," Paper DH06/1, pp. 447–452 in *Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology*, North-Holland Publishers, Amsterdam, The Netherlands, 1993.

137. M. F. Marchbanks, *A Review and Assessment of Materials Property Databases with Particular Reference to Concrete Material Systems*, ORNL/NRC/LTR-89/3, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 1989.

138. C. B. Oland et al., *Plan for Use in Development of the Structural Materials Information Center*, ORNL/NRC/LTR-89/8, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1989.

139. C. B. Oland and D. J. Naus, *Structural Materials Information Center for Presentation of the Time Variation of Material Properties*, ORNL/NRC/LTR-90/22, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 1990.

140. C. B. Oland, *The Structural Materials Information Center and Its Potential Applications*, ORNL/NRC/LTR-92/8, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, May 1992.

141. C. B. Oland and D. J. Naus, *Summary of Materials Contained in the Structural Materials Information Center*, ORNL/NRC/LTR-94/22, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 1994.

142. *Mat.DB, Version 1.22*, American Society for Metals (ASM) International, ASM/Center for Materials Data, Materials Park, Ohio, 1990.

143. *EnPlot, Version 2.0*, ASM International, ASM/Center for Materials Data, Materials Park, Ohio, 1990.

144. M. G. Van Geem and D. J. Naus, *Summary of Test Results for Portland Cement Association Study on Long-Term Properties of Concrete*, ORNL/NRC/LTR-91/26 (Subcontract Report 11X-SF710V from Construction Technology Laboratories, Inc., Skokie, Illinois), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, January 1992.

145. M. G. Van Geem, *Forty-Two-Year Results for PCA Test Series 374 on Long-Term Properties of Concrete*, ORNL/NRC/LTR-92/26 (Subcontract Report 37X-SK412V from Construction Technologies Laboratories, Inc., Skokie, Illinois), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1992.

146. S. Widdows, *Testing of Aged Concretes from U.K. Nuclear Power Stations*, Report 1303/92/6037, Taywood Engineering Ltd., R & D Division, London, England, April 1993 (report prepared under Structural Aging Program).

147. ACI Committee 437, *Strength Evaluation of Existing Buildings*, ACI 437D-67(82), American Concrete Institute, Detroit, Michigan, 1982.

148. M. F. Marchbanks and C. B. Oland, *Considerations in the Formulation of a Customized Data Base for Structural Materials*, ORNL/NRC/LTR-94/20, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 1994.

149. *Microsoft Windows*, Microsoft Corporation, One Microsoft Way, Redmond, Washington.

150. *Microsoft Access, Version 2.0*, Microsoft Corporation, One Microsoft Way, Redmond, Washington.

151. H. Ashar, D. J. Naus, and C. P. Tan, "Prestressed Concrete in U.S. Nuclear Power Plants (Part 1)," pp. 30–34 in *Concrete International*, **16**(5), American Concrete Institute, Detroit, Michigan, May 1994.

152. H. Ashar, C. P. Tan, and D. J. Naus, "Prestressed Concrete in U.S. Nuclear Power Plants (Part 2)," pp. 58–61 in *Concrete International*, **16**(6), American Concrete Institute, Detroit, Michigan, June 1994.

153. H. Ashar and D. Jeng, "Effectiveness of In-Service Inspection Requirements of Prestressed Concrete Containments — U.S. Experience," *Proceedings of Second International Conference on Containment Design and Operation*, Toronto, Ontario, Canada, October 1990.

154. H. Ashar and G. Bagchi, *Assessment of Inservice Conditions of Safety-Related Nuclear Plant Structures*, NUREG 1522, U.S. Nuclear Regulatory Commission, Washington, D.C., July 1995.

155. H. T. Hill, *Concrete Containment Posttensioning System Aging Study*, ORNL/NRC/LTR-95/13, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, August 1995.

156. C. W. Novak, "Evaluation of Existing Nuclear Safety-Related Concrete Structures Post Tensioning – Tendon Sheathing Filler," Viscosity Oil (Willowbrook, Illinois), letter to C. J. Hookham of Black & Veatch, Ann Arbor, Michigan, April 26, 1995.

157. "Requirements for Class CC Concrete Components of Light-Water Cooled Plants," Section XI, Subsection IWL in *American Society of Mechanical Engineers Boiler and Pressure Vessel Code*, 1992 edition with Addenda, New York, New York, 1995.

158. G. H. Lewis, *Degradation of Building Materials Over a Lifespan of 30–100 Years*, EUR 1002 EN, Commission of European Communities, Luxembourg, 1985.

159. D. C. Pocock et al., *Long-Term Performance of Structures Comprising Nuclear Power Plants*, EUR 12758 EN, Commission of European Communities, Luxembourg, 1990.

160. "Containment Liners Found Corroded at Some EDF 900-MW Reactors," pp. 5–6 in *Nucleonics Week*, McGraw-Hill Publishers, New York, New York, May 7, 1992.

161. P. Dawson and M. J. M. Wilson, *Surveillance Data for PCPVs at Wylfa, Hartlepool, and Heysham 1 Power Stations*, Report No. 1302/92/5957, Taywood Engineering Ltd., R & D Division, London, England, January (report prepared under Structural Aging Program)..

162. ACI Committee 364, "Guide for Evaluation of Concrete Structures Prior to Rehabilitation," pp. 479–498 in *ACI Materials Journal*, **90**(5), American Concrete Institute, Detroit, Michigan, September–October 1993.

163. *Guidelines for Structural Condition Assessment of Existing Buildings*, ANSI/ASCE 11-90, American Society of Civil Engineers, New York, New York, August 1, 1991.

164. V. M. Malhotra and N. J. Carino (eds.), *Handbook of Nondestructive Testing of Concrete*, CRC Press, Boca Raton, Florida, 1991.

165. R. G. Mathey and J. R. Clifton, *Review of Nondestructive Evaluation Methods Applicable to Construction Materials and Structures*, NBS Technical Note 1247, National Bureau of Standards, U.S. Department of Commerce, Gaithersburg, Maryland, 1988.

166. V. M. Malhotra (ed.), *In Situ/Nondestructive Testing of Concrete*, SP-82, American Concrete Institute, Detroit, Michigan, 1984.

167. H. S. Lew (ed.), *Nondestructive Testing*, SP-112, American Concrete Institute, Detroit, Michigan, 1988.

168. A. E. Komendant, *Contemporary Concrete Structures*, McGraw-Hill Book Company, New York, New York, 1972.

169. ACI Committee 224, *Control of Cracking in Concrete Structures*, Report 224R-80, American Concrete Institute, Detroit, Michigan, 1980.

170. L. I. Knab et al., "Laboratory Evaluation of Ultrasonics for Crack Detection in Concrete Structures," pp. 17–27 in *Proc. J. American Concrete Institute*, **80**(1), January–February 1983.

171. J. R. Leslie and W. J. Cheesman, "An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures," pp. 17–36 in *Proc. J. American Concrete Institute*, **46**(1), September 1949.

172. M. Sansalone and N. J. Carino, "Stress Wave Propagation Methods," Chapter 12 in *CRC Handbook on Nondestructive Testing of Concrete*, CRC Press, Boca Raton, Florida, 1991.

173. N. J. Carino et al., "Flaw Detection in Concrete by Frequency Spectrum Analysis of Impact Echo Waveforms," pg. 117 in *Int'l Adv. in Nondestructive Testing*, 12th Edition, W. J. McGonnagle (ed.), Gordon & Breech, New York, New York, 1986.

174. K. M. Lozen, "Detecting Invisible Corrosion in Concrete," *Concrete Construction*, **33**(8), The Aberdeen Group, Addison, Illinois, August 1988.

175. J. A. Forrester, "Applications of Gamma Radiography to Concrete," *The Engineer 205*, London, England, February 28, 1958.

176. G. J. Weil, "Infrared Thermographic Techniques," Chapter 13 in *CRC Handbook on Nondestructive Testing of Concrete*, CRC Press, Boca Raton, Florida, 1991.

177. *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*, ASTM C 42, **Vol. 04.02**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1988.

178. W. E. Murphy, "The Interpretation of Tests on the Strength of Concrete in Structures," pp. 377–392 in *In Situ/Nondestructive Testing of Concrete*, SP-82, American Concrete Institute, Detroit, Michigan, 1984.

179. *Concrete Core Testing for Strength*, Publication 51.071, Technical Report No. 11, Concrete Society, London, England, 1976.

180. R. D. Browne and R. Blundell, "The Behavior of Concrete Prestressed Concrete Pressure Vessels," Paper H1/1, *First International Conference on Structural Mechanics in Structural Technology*, Berlin, Germany, 1971.

181. K. Hindo and W. R. Bergstrom, "Statistical Evaluation of In-Place Compressive Strength of Concrete," pp. 44–48 in *Concrete International*, 7(2), American Concrete Institute, Detroit, Michigan, February 1985.

182. "Standard Recommended Practice for Petrographic Examination of Hardened Concrete," ASTM C 856, Park 14, pp. 514–523 in *Annual Book of ASTM Standards*, American Society of Testing and Materials, Philadelphia, Pennsylvania, 1979.

183. G. L. Munday and R. Dhir, "Assessment of In Situ Concrete Quality by Core Testing," pp. 393–410 in *In Situ/Nondestructive Testing of Concrete*, SP-82, American Concrete Institute, Detroit, Michigan, 1984.

184. J. Bungey, "Determination of Concrete Strength by Using Small Diameter Cores," pp. 91–98 in *Mag. Concr. Res.*, **31**(107), June 1979.

185. "Standard Test Method for Pulse Velocity through Concrete," ASTM C 597, Part 14, pp. 353–356 in *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1979.

186. V. M. Malhotra, "Surface Hardness Materials," Chapter 1 in *CRC Handbook on Nondestructive Testing of Concrete*, CRC Press, Boca Raton, Florida, 1991.

187. "Test Method for Penetration Resistance of Hardened Concrete," ASTM C 803, **Vol 04.03**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1988.

188. N. J. Carino, "Pullout Test," Chapter 3 in *CRC Handbook on Nondestructive Testing of Concrete*, CRC Press, Boca Raton, Florida, 1991.

189. K. Mather, "Preservation Technology: Evaluating Concrete in Structures," pp. 33–41 in *Concrete International*, 7(10), American Concrete Institute, Detroit, Michigan, October 1985.

190. "Standard Recommended Practice for Microscopical Determination of Air-Void System," ASTM C 457, Part 14, pp. 465–469 in *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1979.

191. "Standard Test Method for Cement Content of Hardened Portland Cement Concrete," ANSI/ASTM C 85, Part 14, pp. 41–44 in *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1979.

192. *Methods of Testing Concrete*, BS 1881, Part 6, British Standards Institution, London, England, 1983.

193. K. R. Lauer, "Magnetic/Electrical Methods," Chapter 9 in *CRC Handbook on Nondestructive Testing of Concrete*, CRC Press, Boca Raton, Florida, 1991.

194. R. J. Currie, "The Implication of Reinforcement Corrosion for Safety and Serviceability of Structures," pp. 11–18 in *Proceedings of a Meeting on Corrosion of Reinforcement in Concrete Construction*, A. P. Crane (ed.), Ellis Harwood Ltd. Publishers, Chichester, England, 1983.

195. R. D. Browne, "Durability of Reinforced Concrete Structures," *New Zealand Concrete Construction*, Parts 1 and 2, September and October 1989.

196. "Detecting Defects and Deterioration in Highway Structures," National Cooperative Highway Research Program No. 118, Transportation Research Board, Washington, D.C., July 1985.

197. *Standard Test Method for Half Cell Potentials of Reinforcing Steel in Concrete*, ASTM C 876, **Vol. 04.02**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1988.

198. ACI Committee 222, "Corrosion of Metals in Concrete," ACI 222R-85, pp. 3–32 in *Proc. American Concrete Institute*, **82**(1), Detroit, Michigan, January–February 1985.

199. "Codes and Standards for Nuclear Power Plants: Subsection IWE and Subsection IWL," *Federal Register*, **59**(5), United States Nuclear Regulatory Commission, 10 CFR Part 50, Proposed Rules, January 17, 1994.

200. "Inspection and Maintenance of Reinforced and Prestressed Concrete Structures," FIP Guide to Good Practice, Fédération Internationale de la Précontrainte, Thomas Telford Publisher, London, England, 1986.

201. "Requirements for Class MC and Metalic Liners of Class CC Components of Light-Water Cooled Power Plants," Section XI, Subsection IWE in *American Society of Mechanical Engineers Boiler and Pressure Vessel Code*, 1992 edition with Addenda, New York, New York, 1992.

202. G. B. Hasselwander (ed.), *Anchorage to Concrete*, SP-103, American Concrete Institute, Detroit, Michigan, 1987.

203. E. N. Kusenberger and J. R. Barton, *Detection of Flaws in Reinforcing Steel in Concrete Bridge Members*, Rep. No. FHWA/RD-83/081, Federal Highway Administration, Washington, D.C., November 1983.

204. G. A. Matzkanin et al., *Determination of Moisture Levels in Structural Concrete Using Pulsed NMR (Final Report)*, Rep. No. FHWA/RD-82/008, Southwest Research Institute, San Antonio Texas, Federal Highway Administration, Washington, D.C., April 1982.

205. E. Hammond and T. D. Robson, "Comparison of Electrical Properties of Various Cements and Concretes," *The Engineer*, 199, 78, and 115, London, England, 1955.

206. J. R. Bell et al., "Determination of Moisture Content of Hardened Concrete and Its Dielectric Properties," *Proc. ASTM*, **63**, 996, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1963.

207. R. Jones, "A Preview of the Non-Destructive Testing of Concrete," *Proc. Sym. on Non-Destructive Testing of Concrete and Timber*, Institution of Civil Engineers, 1, London, England, June 1969.

208. L. Knab et al., *Laboratory Evaluation of Nondestructive Methods to Measure Moisture in Built-Up Roofing Systems*, National Bureau of Standards Building Science Series 1311, Gaithersburg, Maryland, 1981.

209. E. Esalante et al., *Measuring the Corrosion Rate of Reinforcing Steel in Concrete*, NBSIR 84-2853, National Bureau of Standards, Gaithersburg, Maryland, 1984.

210. E. Esalante et al., *Measuring the Rate of Corrosion of Reinforcing Steel in Concrete (Final Report)*, NBSIR 86-3456, National Bureau of Standards, Gaithersburg, Maryland, 1986.

211. S. Ito et al., *Diagnostic Analysis of Corrosion Deterioration of Steel Structures*, Tech. Rept. No. 32, Nippon Steel, Tokyo, Japan, January 1987.

212. D. D. MacDonald, *Evaluation of Electrochemical Impedance Techniques for Detecting Corrosion of Rebar in Reinforced Concrete*, SHRP-ID/UFR-91-524, **Vol. 1**, Strategic Highway Research Program, National Research Council, Washington, D.C., 1991.

213. "Measuring Corrosion Rate of Reinforced Concrete," *SHRP Product-2001*, Strategic Highway Research Program, National Research Council, Washington, D.C., June 1, 1993.

214. T. M. Refai and M. K. Lim, *In-Service Inspection and Structural Integrity Assessment Methods for Nuclear Power Plant Concrete Structures*, ORNL/NRC/LTR-90/29, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1991.

215. F. K. Brocklehurst et al., *Evaluation of the Use of Laser Ultrasonics for the Rapid, Noncontact Inspection of Concrete and Asphalt*, SHRP-ID/UFR-92-604, Strategic Highway Research Program, National Research Council, Washington, D.C., 1992.

216. R. H. Atkinson and M. P. Schuller, *Characterization of Concrete Conditions Using Acoustic Tomographic Imaging*, Atkinson-Noland & Associates, Inc., Boulder, Colorado, Phase 1 SBIR Technical Report under Contract No. NRC-04-93-094 with U.S. Nuclear Regulatory Commission, April 11, 1994.

217. J. M. Milne, "Thermodynamic Inspection of Concrete Using a Controlled Heat Source," paper presented at *Infrared Technology and Applications Conference* held June 25–28, 1990, in London, England.

218. D. King, Private Communication from Dave King, Raytheon Engineers & Constructors, Inc., Lyndhurst, New Jersey, to D. J. Naus, Oak Ridge National Laboratory, Oak Ridge, Tennessee, May 5, 1995.

219. *Identification of Alkali-Silica Reactivity*, SHRP Product 20-10, Strategic Highway Research Program, National Research Council, Washington, D.C., June 1, 1993.

220. E. J. Gannon and P. D. Cady, *Condition Evaluation of Concrete Bridges Relative to Reinforcement Corrosion: State-of-the-Art of Existing Methods*, SHRP-S-323, **Vol. 1**, Strategic Highway Research Program, National Research Council, Washington, D.C., 1993.

221. K. A. Snyder, J. R. Clifton, and N. J. Carino, *Nondestructive Evaluation of the In-Place Compressive Strength of Concrete Based Upon Limited Destructive Testing*, NISTIR 4874, U.S. Department of Commerce, National Institute of Standards and Technology, Gaithersburg, Maryland, July 1992 (report prepared under Structural Aging Program).

222. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division*, ORNL/FTR-4924, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 31, 1994.

223. ACI Committee 201, "Guide for Making a Condition Survey of Concrete In-Service," pp. 905–918 in *Proceedings of American Concrete Institute*, **65**(11), Detroit, Michigan, November 1968

224. ACI Committee 224, "Causes, Evaluation, and Repair of Cracks in Concrete Structures," Report No. 224.R-84, pp. 211–230 in *Journal of American Concrete Institute*, **81**(3), Detroit, Michigan, May–June 1984.

225. ACI Committee 207, "Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions," pp. 47–61 in *Concrete International*, **1**(3), American Concrete Institute, Detroit, Michigan, March 1979.

226. ACI Committee 437, "Strength Evaluation of Existing Concrete Structures," pp. 707–710 in *Journal of American Concrete Institute*, **64**(11), Detroit, Michigan, November 1968.

227. ACI Committee 311, "Guide for Concrete Inspection," pp. 81–85 in *Concrete International*, **2**(10), Detroit, Michigan, October 1980.

228. W. F. Perenchio, "The Condition Survey," pp. 59–62 in *Concrete International*, **11**(1), American Concrete Institute, Detroit, Michigan, January 1989.

229. "Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete," *Annual Book of ASTM Standards*, ASTM C1152, **Vol. 04.02**, American Society of Testing and Materials, Philadelphia, Pennsylvania, 1990.

230. ACI Committee 349, *Evaluation of Existing Nuclear Safety-Related Concrete Structures*, ACI 349.3R-95 (Draft), American Concrete Institute, Detroit, Michigan, 1995.

231. J. J. Waddell, "Basic Steps of a Concrete Repair Program," pp. 30–33 in *Concrete International*, **2**(9), American Concrete Institute, Detroit, Michigan, September 1980.

232. Technical Committee 124-SRC, "Draft Recommendation for Repair Strategies for Concrete Structures Damaged by Reinforcement Corrosion," pp. 415–436 in *Materials and Structures*, **27**(171), International Union of Testing and Research Laboratories for Materials and Structures (RILEM), Cachan, France, 1994.

233. W. F. Price et al., *Review of European Repair Practice for Corrosion Damaged Reinforced Concrete*, Report No. 1303/91/5823, Taywood Engineering Ltd., R & D Division, London, England, April 1993. (report prepared under Structural Aging Program).

234. J. McDonald and R. Campbell, Sr., "The Condition of the Corps of Engineers Civil Works Concrete Structures," U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi, 1985.

235. *Summary Results of the Survey on Concrete Containment Ageing*, IAEA/NENS, International Atomic Energy Agency, Vienna, Austria, 1995.

236. D. Darwin, "Debate: Crack Width, Cover, and Corrosion," pp. 29–35 in *Concrete International*, **7**(5), American Concrete Institute, Detroit, Michigan, May 1985.

237. D. Campbell-Allen and B. Lau, *Cracks in Concrete Bridge Decks*, Civil Engineering Research Report R313, University of Sydney, Australia, January 1978.

238. A. W. Beeby, "The Prediction of Crack Width in Hardened Concrete," pp. 9–17 in *The Structural Engineer*, **57A**(1), January 1979.

239. *Evaluation and Repair of Concrete Structures*, EM-1110-2-2003, U.S. Army Corps of Engineers, Washington, D.C., 1986.

240. *Chemical Grouting*, Engineer Manual EM 1110-2-3504, U.S. Army Corps of Engineers, Office of Chief of Engineers, Washington, D.C., May 1973.

241. *Standard Specification for the Repair of Concrete*, M-47, M0470000.390, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, March 1, 1990.

242. ACI Committee 546, "Guide for Repair of Concrete Bridge Superstructures," ACI 546.1R-80 in *Manuarl of Concrete Practice*, Part 2, American Concrete Institute, Detroit, Michigan, 1988.

243. H. Jerzak, "Unpublished Test Data," Transportation Laboratory, California Department of Transportation, Sacramento, California, 1988.

244. ACI Committee 116, *Cement and Concrete Terminology*, ACI 116R-85, American Concrete Institute, Detroit, Michigan, 1985.

245. D. W. Pfeifer and M. J. Scali, *Concrete Sealers for Protection of Bridge Structures*, NCHRP Report No. 244, Transportation Research Board, National Cooperative Highway Research Program, Washington, D.C., December 1981.

246. "Method of Measuring Adhesion by Tape Test," *Annual Book of ASTM Standards*, ASTM D3359-87, **Vol. 06.01**, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1989.

247. P. Pullar-Strecker, *Corrosion Damaged Concrete Assessment and Repair*, Butterworths, London, England, 1987.

248. *Troubleshooting Concrete Problems – And How to Prevent Them in the Future*, SCM-17, American Concrete Institute, Detroit, Michigan, 1987.

249. *Concrete Repair Basics*, SCM 24-91, CT92, American Concrete Institute, Detroit, Michigan, 1991.

250. "The REMR Notebook," U.S. Army Corps of Engineers, Department of Army, Washington, D.C., April 1992.

251. German Committee on Reinforced Concrete, "Richtlinie für Schutz und Instandesetzung von Betonbauteilen, Part 2: Bauplanung und Bauausführung," Deutscher für Stahlbeton, Berlin, Germany, August 1990.

252. "Austrian Standard ONORM B4200, Part 6," *Proceedings of Conference on Repair and Maintenance of Concrete and Reinforced Concrete Structures*, held January 14–15, 1988, at the University of Innsbruck, Austria.

253. *Repair of Concrete Damaged by Reinforcement Corrosion*, Technical Report 26, Concrete Society, London, England, 1984.

254. "Code of Practice for Routine Maintenance," DTp, Department of Transport, London, England, 1985.

255. "Materials for the Repair of Concrete Highway Structures," DTp, Department of Transport, London, England, 1986.

256. *Behavior in Service, Maintenance, and Repair*, Bulletin 163, Comite Euro International du Beton, Paris, France, 1983.

257. T. A. Meyers et al., "Application of Structural Reliability and Risk Assessment to the Management of Aging," NUREG/CP-0100 in *Proceedings of Internatinal Nuclear Power Plant Aging Symposium*, Washington, D.C., 1988.

258. J. A. Christensen, "NPAR Approach to Controlling Aging in Nuclear Power Plants," pp. 509–529 in *Proceedings of the 17th Water Reactor Safety Information Meeting*, **Vol. 3**, NUREG/CP-0105, Washington, D.C., 1990.

259. C. J. Hookham, *Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants*, ORNL/NRC/LTR-90/17, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 1991.

260. *Quality Group Classifications and Standards for Water, Steam, and Radioactive-Waste-Containing Components of Nuclear Power Plants (For Comment)*," Regulatirion Guide 1.26 (Rev. 3), Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., February 1976.

261. *PRA Procedures Guide*, NUREG/CR-2300, **Vols. 1 and 2**, U.S. Nuclear Regulatory Commission, Washington, D.C., 1983.

262. B. R. Ellingwood, *Aging Effects on Probabilistic Risk Assessment*, NUREG/CR-6425 (ORNL/TM-13149), Lockheed Martin Research Corp., Oak Ridge National Laboratory, Oak Ridge, Tennessee, January 1996.

263. P. Emilius, Jr., "Close-Range Photogrammetry Documents As-Built Structures," *Power*, December 1994.

264. C. J. Hookham, *In-Service Inspection Guidelines for Concrete Structures in Nuclear Power Plants*, ORNL/NRC/LTR-95/14, Lockheed Martin Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, December 1995.

265. J. R. Dougan, *Evaluation of Inservice Inspections of Greased-Prestressing Tendons*, NUREG/CR-2719 (ORNL/TM-8278), Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1982.

266. *An International Survey of In-Service Inspection Experience with Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors*, FIP/3/5, Fédération Internationale de la Précontrainte, Wexham Springs, Slough, England, April 1982.

267. "ASTM Protective Coatings Standards for Use in Nuclear Power Plants," American Society for Testing and Materials, Philadelphia, Pennsylvania, 1990.

268. *Manual on Maintenance of Coatings for Nuclear Power Plants*, Manual MNL 8, American Society for Testing and Materials, Philadelphia, Pennsylvania, 1990.

269. *Concrete-Performance Production, Placing, and Compliance Criteria*, (Draft Document) prEN 206, Comité European de Normalisation, 1984.

270. M. Regourd, "Physico-Chemical and Biological Processes Related to Concrete," in *Durability of Concrete Structures*, Comité Euro-International du Béton, Bulletin d'Information 152, Leusanne, Switzerland, 1984.

271. "Load Carrying Capacity of Structural Members Subjected to Alakli-Silica Reactions," Danish Ministry of Transport, Copenhagen, Denmark, October 1990.

272. E. Koehn and A. Kao, *Evaluation of Existing Condition Rating Procedures for Civil Works Structures and Facilities*, Technical Report REMR-OM-1, Department of Army, U.S. Army Corps of Engineers, Champaign, Illinois, May 1986.

273. R. Stowe and H. Thornton, Jr., *Engineering Condition Survey on Concrete in Service*, Technical Report REMR-CS-1, Department of Army, U.S. Army Corps of Engineers, Vicksburg, Mississippi, May 1989.

274. R. Bullock, *A Rating System for the Concrete in Navigation Lock Monoliths*, Technical Report REMR-0M-4, Department of Army, U.S. Army Corps of Engineers, Vicksburg, Misissppi, May 1989.

275. E. Ellingwood et al., "Probability Based Load Criteria: Load Factors and Load Combinations," pp. 978–997 in *Journal of Structural Division*, **108**(5), American Society of Civil Engineers, New York, New York, 1982.

276. T. V. Galambos et al., "Probability Based Load Criteria: Assessment of Current Design Practice," pp. 957–977, Ibid.

277. H. B. Hwang et al., "Probability-Based Design Criteria for Nuclear Plant Structures," pp. 925–942 in *Journal of Structure Design*, **113**(5), American Society of Civil Engineers, New York, New York, 1987.

278. B. Ellingwood and Y. Mori, "Probabilistic Methods for Condition Assessmentand Life Prediction of Concrete Structures in Nuclear Plants," pp. 155–160 in *Nuclear Engineering and Design*, **142**, Elsevier Science Publishers, Amsterdam, The Netherlands, 1993.

279. Y. Mori and B. Ellingwood, "Reliability-Based Service Life Assessment of Aging Concrete Structures," pp. 1600–1621 in *Journal of Structural Engineering*, **119**(5), American Society of Civil Engineers, New York, New York, 1993.

280. Y. Mori and B. Ellingwood, "Maintaining Reliability of Concrete Structures I: Role of Inspection/Repair," pp. 824–845 in *Journal of Structural Engineering*, **120**(3), American Society of Civil Engineers, New York, New York, 1994.

281. Y. Mori and B. Ellingwood, "Maintaining Reliability of Concrete Structures II: Optimum Inspection/Repair Strategies," pp. 846–862 in *Journal of Structural Engineering*, **120**(3), American Society of Civil Engineers, New York, New York, 1994.

282. G. Somerville, "The Design Life of Concrete Structures," pp. 60–71 in *The Structural Engineer*, **64A**(2), with "Discussion" in pp. 233–241 of *The Structural Engineer*, **64A**(9), London, England, 1986.

283. B. Ellingwood and Y. Mori, *Condition Assessment and Reliability-Based Life Prediction of Concrete Structures*, ORNL/NRC/LTR-92/4, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, 1992.

284. R. Larrabee and C. A. Cornell, "Combinations of Various Load Processed," pp. 223–239 in *Journal of Structural Design*, **107**(1), American Society of Civil Engineers, New York, New York, 1981.

285. T. H. Pearce and Y. K. Wen, "Stochastic Combinations of Load Effects," pp. 1613–1629 in *Journal of Structural Engineers*, **110**(7), American Society of Civil Engineers, New York, New York, 1984.

286. B. Ellingwood and H. Hwang, "Probabilistic Descriptions of Resistance of Safety-Related Structures in Nuclear Plants," pp. 169–178 in *Nuclear Engineering and Design*, **88**(2), Elsevier Science Publishers, Amsterdam, The Netherlands, 1985.

287. J. G. MacGregor, A. Mirza, and B. Ellingwood, "Statistical Analysis of Resistance of Reinfored and Prestressed Concrete Members," pp. 167–176 in *Journal of American Concrete Institute*, **80**(3), Detroit, Michigan, May/June 1983.

288. B. Ellingwood, "Probabilistic Risk Assessment," pp. 89–116 in *Engineering Safety*, McGraw-Hill Book Co., Ltd., London, England, 1992.

289. D. Veneziano et al., "Vector Process Models for System Reliability," pp. 441–460 in *Journal of Engineering Mechanics*, **103**(3), American Society of Civil Engineers, New York, New York, 1977.

290. Y. Mori and B. Ellingwood, *Reliability Assessment of Degrading Concrete Shear Walls*, ORNL/NRC/LTR-94/6, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, April 1994.

291. F. Barda, J. M. Hanson, and W. G. Corley, "Shear Strength of Low-Rise Walls with Boundary Element," in *Reinforced Concrete Structures in Seismic Zones*, ACI SP-53, American Concrete Institute, Detroit, Michigan, 1977.

292. I. Shiraishi et al., "Macroscopic Models for R/C Shear Walls," pp. 271–280 in *Proceedings of Structural Congress, Structural Design, Analysis, and Testing*, American Society of Civil Engineers (ASCE), New York, New York, 1989.

293. R. Shohara, N. Shirai, and H. Noguchi, "Verification of Macroscopic Models for R/C Walls," pp. 281–290, Ibid.

294. F. Watanabe and T. Ichinose, "Strength and Durability Design of RC Members Subjected to Combined Bending and Shear," pp. 429–438 in *Concrete Shear in Earthquake*, T. C. C. Hus and S. T. Mau (eds.), Elsevier Applied Science, New York, New York, 1992.

295. W. F. Chen and D. J. Han, "Plasticity for Structural Engineers," Springer-Vartag, New York, New York, 1988.

296. F. P. Vecchio and M. P. Collins, "The Modified Compression-Field Theory for Reinforced Concrete Elements Subjected to Shear," pp. 219–231 in *Journal of American Concrete Institute*, **83**(2), Detroit, Michigan, March–April 1986.

297. A. E. Cardenas et al., "Design Provisions for Shear Walls," pp. 221–230 in *Journal of American Concrete Institute*, **70**(3), Detroit, Michigan, March 1973.

298. Y. Mori, "Reliability-Based Condition Assessment and Life Prediction of Concrete Structures," thesis presented to the Johns Hopkins University, Baltimore, Maryland, in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 1992.

299. *Reliability-Based Inspection-Development of Guidelines*, NUREG/GR-0005, U.S. Nuclear Regulatory Commission, Washington, D.C., 1992.

300. H. Madsen, J. D. Sorenson, and R. Olesen, "Optimal Inspection Planning for Fatigue Damage of Offshore Structures," pp. 2099–2106 in *Proceedigs of ICOSSAR '89*, **Vol. III**, ASCE, New York, New York, 1989.

301. J. Yang, "Application of Reliability Methods to Fatigue, Quality Assurance, and Maintenance," pp. 3–18 in *Proceedings of ICOSSAR '93*, **Vol. I**, A. A. Balkema, Rotterdam, The Netherlands, 1994.

302. H. M. Taylor and S. Karlin, "An Introduction to Stochastic Modeling," Academic Press, Orlando, Florida, 1984.

303. Y. Mori and B. Ellingwood, *Methodology for Reliability Based Condition Assessment-Application to Concrete Structures*, NUREG/CR-6052, U.S. Nuclear Regulatory Commission, Washington, D.C., August 1993.

304. O. J. V. Chapman, "A Statistical Approach to the Analysis of ISI Data Using Bayes Method," in *Proceedings of the 7th International Conference on Structural Mechanics in Reactor Technology*, **Vol. K**, Chicago, Illinois, 1983.

305. P. Thoft-Christensen and J. D. Sorensen, "Optimal Strategy for Inspection and Repair on Structural Stystems," pp. 94–100 in *Civil Engineering Systems*, **4**, June 1987.

306. J. R. Benhamin and C. A. Cornell, *Probability, Statistics, and Decision for Civil Engineering*, McGraw-Hill Publishers, New York, New York, 1970.

307. B. A. Urtagn and M. A. Saunders, *MINOS 5.1 Users Guide*, Technical Report SOL 83-20R, System Optimization Laboratory, Department of Operation Research, Stanford University, Stanford, California, 1987.

308. K. Tutti, *Corrosion of Steel in Concrete*, Report 4-82, Swedish Cement and Concrete Research Institute, Stockholm, Sweden, 1982.

APPENDIX A

U.S. Commercial Nuclear Power Reactor Summary[1,2†]

Nuclear Unit	Docket Number	Power MW(e), Net	Type* Plant	Type* Containment	Construction Permit	Operating License Issue	Operating License Expiration	Commercial Operation
Arkansas 1	50-313	836	P	PC-D	1968	1974	2014	1974
Arkansas 2	50-368	858	P	PC-D	1972	1978	2018	1980
Beaver Valley 1	50-334	810	P	RC-SA	1970	1976	2016	1976
Beaver Valley 2	50-412	820	P	RC-SA	1974	1987	2027	1987
Bellefonte 1	50-438	1212	P	PC-C	1974			
Bellefonte 2	50-439	1212	P	PC-C	1974			
Big Rock Point 1	50-155	69	B	S-PREMK	1960	1962	2000	1963
Braidwood 1	50-456	1120	P	PC-D	1975	1987	2026	1988
Braidwood 2	50-457	1120	P	PC-D	1975	1988	2027	1988
Browns Ferry 1	50-259	1065	B	S-MKI	1967	1973	2013	1974
Browns Ferry 2	50-260	1065	B	S-MKI	1967	1974	2014	1975
Browns Ferry 3	50-296	1065	B	S-MKI	1968	1976	2016	1977
Brunswick 1	50-325	790	B	RC-MKI	1970	1976	2016	1977
Brunswick 2	50-324	790	B	RC-MKI	1970	1974	2014	1975
Byron 1	50-454	1120	P	PC-D	1975	1985	2024	1985
Byron 2	50-455	1120	P	PC-D	1975	1987	2026	1987
Callaway 1	50-483	1180	P	PC-E	1976	1984	2024	1984
Calvert Cliffs 1	50-317	845	P	PC-B	1969	1974	2014	1975
Calvert Cliffs 2	50-318	845	P	PC-B	1969	1976	2016	1977
Catawba 1	50-413	1145	P	S-IC	1975	1985	2024	1985
Catawba 2	50-414	1145	P	S-IC	1975	1986	2026	1986
Clinton 1	50-461	950	B	RC-MKIII	1976	1987	2026	1987
Comanche Peak 1	50-445	1150	P	RC-LD	1974	1990	2030	1990
Comanche Peak 2	50-446	1150	P	RC-LD	1974	1993	2033	
Cook 1	50-315	1030	P	RC-IC	1969	1974	2014	1975
Cook 2	50-316	1100	P	RC-IC	1969	1977	2017	1978
Cooper Station	50-298	778	B	S-MKI	1968	1974	2014	1974
Crystal River 3	50-302	825	P	PC-B	1968	1977	2016	1977

†Sources: 1. *Information Digest*, U.S. Nuclear Regulatory Commission, Office of the Controller, NUREG-1350, Vol. 7, Washington, D.C., March 1995.
2. *Commercial Nuclear Power Plants*, Edition No. 19, Halliburton NUS, Clearwater, Florida, December 1991.

APPENDIX A (Cont'd)

Nuclear Unit	Docket Number	Power MW(e), Net	Type Plant	Type Containment	Construction Permit	Operating License Issue	Operating License Expiration	Commercial Operation
Davis-Besse 1	50-346	906	P	S-LD	1969	1977	2017	1978
Diablo Canyon 1	50-275	1073	P	RC-LD	1968	1984	2008	1985
Diablo Canyon 2	50-323	1087	P	RC-LD	1970	1985	2010	1986
Dresden 2	50-237	794	B	S-MKI	1966	1969	2006	1970
Dresden 3	50-249	794	B	S-MKI	1966	1971	2011	1971
Duane Arnold	50-331	538	B	S-MKI	1970	1974	2014	1975
Farley 1	50-348	829	P	PC-D	1972	1977	2017	1977
Farley 2	50-364	829	P	PC-D	1972	1981	2021	1981
Fermi-2	50-341	1093	B	S-MKI	1972	1985	2025	1988
FitzPatrick	50-333	816	B	S-MKI	1970	1974	2014	1975
Fort Calhoun 1	50-285	492	P	PC-A	1968	1973	2013	1974
Ginna	50-244	470	P	PC-A	1966	1969	2009	1970
Grand Gulf 1	50-416	1250	B	RC-MKIII	1974	1984	2022	1985
Haddam Neck	50-213	565	P	RC-LD	1964	1967	2007	1968
Hatch 1	50-321	786	B	S-MKI	1969	1974	2014	1975
Hatch 2	50-366	795	B	S-MKI	1972	1978	2018	1979
Hope Creek 1	50-354	1067	B	S-MKI	1974	1986	2026	1986
Indian Point 2	50-247	1007	P	RC-LD	1966	1973	2013	1974
Indian Point 3	50-286	965	P	RC-LD	1969	1976	2015	1976
Kewaunee	50-305	535	P	S-LD	1968	1973	2013	1974
LaSalle 1	50-373	1078	B	PC-MKII	1973	1982	2022	1984
LaSalle 2	50-374	1078	B	PC-MKII	1973	1984	2023	1984
Limerick 1	50-352	1055	B	RC-MKII	1974	1985	2024	1986
Limerick 2	50-353	1055	B	RC-MKII	1974	1989	2029	1990
Maine Yankee	50-309	840	P	RC-SA	1968	1973	2008	1972
McGuire 1	50-369	1180	P	S-IC	1973	1981	2021	1981
McGuire 2	50-370	1180	P	S-IC	1973	1983	2023	1984
Millstone 1	50-245	654	B	S-MKI	1966	1970	2010	1971
Millstone 2	50-336	863	P	PC-D	1970	1975	2015	1975
Millstone 3	50-423	1137	P	RC-SA	1974	1986	2025	1986
Monticello	50-263	536	B	S-MKI	1967	1971	2010	1971
Nine Mile Point 1	50-220	610	B	S-MKI	1965	1969	2009	1969

APPENDIX A (Cont'd)

Nuclear Unit	Docket Number	Power MW(e), Net	Type Plant	Type Containment	Construction Permit	Operating License Issue	Operating License Expiration	Commercial Operation
Nine Mile Point 2	50-410	1080	B	RC-MKII	1974	1987	2026	1988
North Anna 1	50-338	915	P	RC-SA	1971	1978	2018	1978
North Anna 2	50-339	915	P	RC-SA	1971	1980	2020	1980
Oconee 1	50-269	860	P	PC-B	1967	1973	2013	1973
Oconee 2	50-270	860	P	PC-B	1967	1973	2013	1974
Oconee 3	50-287	860	P	PC-B	1967	1974	2014	1974
Oyster Creek 1	50-219	620	B	S-MKI	1964	1969	2009	1969
Palisades	50-255	777	P	PC-B	1967	1972	2007	1971
Palo Verde 1	50-528	1270	P	PC-E	1976	1985	2024	1986
Palo Verde 2	50-529	1270	P	PC-E	1976	1986	2025	1986
Palo Verde 3	50-530	1270	P	PC-E	1976	1987	2027	1988
Peach Bottom 2	50-277	1065	B	S-MKI	1967	1973	2013	1974
Peach Bottom 3	50-278	1065	B	S-MKI	1967	1974	2014	1974
Perry 1	50-440	1205	B	S-MKIII	1977	1986	2026	1987
Pilgrim 1	50-293	670	B	S-MKI	1968	1972	2012	1972
Point Beach 1	50-266	497	P	PC-B	1967	1970	2010	1970
Point Beach 2	50-301	497	P	PC-B	1968	1971	2013	1972
Prairie Island 1	50-282	520	P	S-LD	1968	1974	2013	1973
Prairie Island 2	50-306	520	P	S-LD	1968	1974	2014	1974
Quad Cities 1	50-254	789	B	S-MKI	1967	1972	2012	1973
Quad Cities 2	50-265	789	B	S-MKI	1967	1972	2012	1973
River Bend 1	50-458	936	B	S-MKIII	1977	1985	2025	1986
Robinson 2	50-261	665	P	PC-A	1967	1970	2010	1971
St. Lucie 1	50-335	810	P	S-LD	1970	1976	2016	1976
St. Lucie 2	50-389	810	P	S-LD	1977	1983	2023	1983
Salem 1	50-272	1090	P	RC-LD	1968	1976	2016	1977
Salem 2	50-311	1115	P	RC-LD	1968	1981	2020	1981
San Onofre 2	50-361	1070	P	PC-E	1973	1982	2013	1983
San Onofre 3	50-362	1080	P	PC-E	1973	1983	2013	1984
Seabrook 1	50-443	1150	P	RC-LD	1976	1990	2026	1990
Sequoyah 1	50-327	1150	P	S-IC	1970	1980	2020	1981

APPENDIX A (Cont'd)

Nuclear Unit	Docket Number	Power MW(e), Net	Type Plant	Type Containment	Construction Permit	Operating License		Commercial Operation
						Issue	Expiration	
Sequoyah 2	50-328	1150	P	S-IC	1970	1981	2021	1982
Shearon Harris 1	50-400	860	P	RC-LD	1978	1987	2026	1987
South Texas 1	50-498	1250	P	PC-E	1975	1988	2027	1988
South Texas 2	50-499	1250	P	PC-E	1975	1989	2028	1989
Summer 1	50-395	900	P	PC-C	1973	1982	2022	1984
Surry 1	50-280	781	P	RC-SA	1968	1972	2012	1972
Surry 2	50-281	781	P	RC-SA	1968	1973	2013	1973
Susquehanna 1	50-387	1050	B	RC-MKII	1973	1982	2022	1983
Susquehanna 2	50-388	1050	B	RC-MKII	1973	1984	2024	1985
Three Mile Island 1	50-289	792	P	PC-B	1968	1974	2014	1974
Turkey Point 3	50-250	728	P	PC-B	1967	1972	2012	1972
Turkey Point 4	50-251	728	P	PC-B	1967	1973	2013	1973
Vermont Yankee 1	50-271	514	B	S-MKI	1967	1973	2012	1972
Vogtle 1	50-424	1160	P	PC-E	1974	1987	2027	1987
Vogtle 2	50-425	1160	P	PC-E	1974	1989	2029	1989
Washington Nucl. 2	50-397	1150	B	S-MKII	1973	1984	2023	1984
Waterford 3	50-382	1165	P	S-LD	1974	1985	2024	1985
Watts Bar 1	50-390	1170	P	S-IC	1971			
Watts Bar 2	50-391	1170	P	S-IC	1971			
Wolf Creek	50-482	1150	P	PC-E	1977	1985	2025	1985
Zion 1	50-295	1040	P	PC-B	1968	1973	2013	1973
Zion 2	50-304	1040	P	PC-B	1968	1973	2013	1974

* B = Boiling Water Reactor
P = Pressurized-Water Reactor
S-LD = Steel, Large Dry
S-PREMK = Steel, Pre-Mark
S-MKI = Steel, Mark I
S-MKII = Steel, Mark II
S-MKIII = Steel, Mark III
S-IC = Steel, Ice Condenser

PC-A = Prestr. Concrete, No Buttresses
PC-B = Prestr. Concrete, 6 Buttresses/Shallow Dome
PC-C = Prestr. Concrete, 4 Buttresses/Shallow Dome
PC-D = Prestr. Concrete, 3 Buttresses/Shallow Dome
PC-E = Prestr. Concrete, 3 Buttresses/Hemispherical Dome
PC-MKII = Prestr. Concrete, Mark II

RC-LD = Rein. Concrete, Large Dry
RC-MKI = Rein. Concrete, Mark I
RC-MKII = Rein. Concrete, Mark II
RC-MKIII = Rein. Concrete, Mark III
RC-SA = Rein. Concrete, Subatmospheric
RC-IC = Rein. Concrete, Ice Condenser

APPENDIX B
REPORTS AND PAPERS PREPARED UNDER THE SAG PROGRAM

1. D. J. Naus, *Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants*, NUREG/CR-4652 (ORNL/TM-10059), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1986.*

2. D. J. Naus, "Aging of Concrete Components and Its Significance Relative to Life Extension of Nuclear Power Plants," pp. 229–234, in *Trans. of the 9th Int'l. Conf. on Str. Mech. in Reactor Tech.*, Paper D5/1, Lausanne, Switzerland, A. A. Belkema (Publisher), August 1987.*

3. D. J. Naus and C. E. Pugh, *Report of Foreign Travel of C. E. Pugh and D. J. Naus, Engineering Technology Division,* ORNL/FTR-2694, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, August 8–26, 1987 (September 21, 1987).*

4. D. J. Naus, M. F. Marchbanks, and E. G. Arndt, "Evaluation of Aged Concrete Structures for Continued Service in Nuclear Power Plants," pp. 57–67, in *Proc. of Topical Meeting on Nuclear Power Plant Life Extension*, July 31–August 3, 1988, Snowbird, Utah, Session 2, Paper 1, American Nuclear Society, 1988.

5. D. J. Naus et al., *Structural Aging Program Five-Year Plan: FY 1988–1992*, ORNL/NRC/LTR-89/1, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 1989.

6. M. F. Marchbanks, *A Review and Assessment of Materials Property Databases with Particular Reference to Concrete Material Systems*, ORNL/NRC/LTR-89/3, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 1989.

7. D. J. Naus, M. F. Marchbanks, and E. G. Arndt, "Evaluation of Aged Concrete Structures for Continued Service in Nuclear Power Plants," *Proceedings of the United States Nuclear Regulatory Commission Sixteenth Water Reactor Safety Information Meeting* held at National Institute of Standards and Technology, Gaithersburg, Maryland, NUREG/CP-0097, **Vol. 3**, *Structural and Seismic Engineering*, March 1989.

8. D. J. Naus et al., "Structural Aging Program to Assess Adequacy of Critical Concrete Components in Nuclear Power Plants," pp. 109–118 in *Transactions of the 10th International Conference on Structural Mechanics in Reactor Technology*, Session D, Paper 122, Anaheim, California, August 1989.

9. C. B. Oland, M. F. Marchbanks, and D. J. Naus, *Plan for Use in Development of the Structural Materials Information Center*, ORNL/NRC/LTR-89/8, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1989.

---

*Prepared under NPAR Program.

10. D. J. Naus and C. B. Oland, *Five-Year Compressive Strength Test Results for Moist-Cured and Sealed High-Strength Variable Fly Ash Content Concretes*, ORNL/NRC/LTR-89/12, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 1989.

11. D. J. Naus et al., "Considerations in the Evaluation of Concrete Structures for Continued Service in Aged Nuclear Power Plants," pp. 827–32 in *Proceedings of the American Power Conference,* **Vol. 51**, Illinois Institute of Technology, Chicago, Illinois, 1989.

12. D. J. Naus et al., *Structural Aging (SAG) Program Five-Year Plan: FY 1989 - 1993,* ORNL/NRC/LTR-89/15, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, December 1989.

13. D. J. Naus, C. B. Oland, and M. F. Marchbanks, *Structural Aging Program Annual Technical Progress Report for Period October 1, 1988, to September 30, 1989 (FY 1989),* ORNL/NRC/LTR-90/1, Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, January 1990.

14. C. B. Oland, *Report of Canadian Travel of C. B. Oland, Pressure Vessel Technology Section, Engineering Technology Division,* Letter Report to E. G. Arndt, U. S. Nuclear Regulatory Commission, Rockville, Maryland, from C. B. Oland, Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, April 10, 1990.

15. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, June 9–23, 1990,* ORNL/FTR-3641, Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, July 9, 1990.

16. C. B. Oland and D. J. Naus, *Structural Materials Information Center for Presentation of the Time Variation of Materials Properties,* ORNL/NRC/LTR-90/22, Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 1990.

17. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, November 3–15, 1990,* ORNL/FTR-3827, Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 27, 1990.

18. C. J. Hookham, *Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants,* ORNL/NRC/LTR-90/17 (Subcontract Report 11X-SD343V from Multiple Dynamics Corporation, Southfield, Michigan), Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 1991.

19. D. J. Naus and C. B. Oland, *Structural Aging Program Technical Progress Report for Period October 1, 1989, to December 31, 1990,* ORNL/NRC/LTR-91/2, Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 1991.

20. D. J. Naus, C. B. Oland, and E. G. Arndt, "Management of the Aging of Critical Safety-Related Concrete Structures in Light-Water Reactor Plants," pp. 527–552 in *Proceedings of United States Nuclear Regulatory Commission Eighteenth Water Reactor Safety Information Meeting* held at Holiday Inn–Crowne Plaza, Rockville, Maryland, NUREG/CP-0114, *Vol. 1 – Structural and Seismic Engineering,* April 1991.

21. C. B. Oland and D. J. Naus, "Development of the Structural Materials Information Center," pp. 579–596 in *Proceedings of United States Nuclear Regulatory Commission Eighteenth Water Reactor Safety Information Meeting* held at Holiday Inn– Crowne Plaza, Rockville, Maryland, NUREG/CP-0114, *Vol. 1– Structural and Seismic Engineering,* April 1991.

22. B. R. Ellingwood and Y. Mori, "Probabilistic Methods for Condition Assessment and Life Prediction of Concrete Structures in Nuclear Power Plants, pp. 553–577 in *Proceedings of United States Nuclear Regulatory Commission Eighteenth Water Reactor Safety Information Meeting* held at Holiday Inn–Crowne Plaza, Rockville, Maryland, NUREG/CP-0114, *Vol. 1–Structural and Seismic Engineering,* April 1991.

23. D. J. Naus, C. B. Oland, and E. G. Arndt, "Aging Management of Safety-Related Concrete Structures in Nuclear Power Plants," pp. 1–7 in *Technical Sessions on Nuclear Plant Systems/Components Aging Management and Life Extension,* ASME Pressure Vessel and Piping Conference 1991, ASME PVP-**Vol. 208**, San Diego, California, June 23–27, 1991.

24. D. J. Naus, C. B. Oland, and E. G. Arndt, "Aging Management of Safety-Related Concrete Structures to Provide Improved Bases for Continuing the Service of Nuclear Power Plants," pp. 308–316 in *Journal. of Mat'ls and Struct.,* **24**(142), The International Union of Testing and Research Laboratories for Materials and Structures, Chapman and Hall, New York, July 1991.

25. B. R. Ellingwood and Y. Mori, "Probabilistic Methods for Life Prediction of Concrete Structures in Nuclear Power Plants," pp. 291–296 in *Transactions of the 11th International Conference on Structural Mechanics in Reactor Technology,* **Vol. D**, Tokyo, Japan, August 18–23, 1991.

26. C. B. Oland, *Report of Foreign Travel of C. B. Oland, Engineering Technology Division, August 4–16, 1991,* ORNL/FTR-4004, Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, August 30, 1991.

27. D. J. Naus, C. B. Oland, and E. G. Arndt, "Aging of Concrete Structures in Nuclear Power Plants," M91-50997, *Minutes, RILEM TC-104 Damage Classification of Concrete Structures*, Technical University, Kosice, Czechoslovakia, August 30, 1991.

28. D. J. Naus, C. B. Oland, and E. G. Arndt, "Aging of Concrete Structures in Nuclear Power Plants," *Proceedings of the RILEM-IMEKO International Conference on Diagnosis of Concrete Structures,* EXPERTCENTRUM CZECHOSLOVAKIA, September 3–6, 1991, Slovak Technical University, Bratislava, Czechoslovakia, 1991.

29. T. M. Refai and M. K. Lim, *Inservice Inspection and Structural Integrity Assessment Methods for Nuclear Power Plant Concrete Structures,* ORNL/NRC/LTR-90/29 (Subcontract Report 17X-SE611V from Construction Technology Laboratories, Inc., Skokie, Illinois), Martin Marietta Energy Systems, Inc., Oak Ridge Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1991.

30. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, August 26–September 7, 1991* , ORNL/FTR-4029, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 23, 1991.

31. J. R. Clifton, *Predicting the Remaining Life of In-Service Concrete,* NISTIR 4712, National Institute of Standards and Technology, U.S. Department of Commerce, Gaithersburg, Maryland, November 1991.

32. B. R. Ellingwood and Y. Mori, "Condition Assessment and Reliability-Based Life Prediction of Concrete Structures in Nuclear Power Plants," ORNL/NRC/LTR-92/4 (Subcontract Report 19X-SD684V from The Johns Hopkins University, Baltimore, Maryland), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, January 1992.

33. M. G. Van Geem and D. J. Naus, *Summary of Test Results for Portland Cement Association Study on Long-Term Properties of Concrete,* ORNL/NRC/LTR-91/26 (Subcontract Report 11X-SF710V from Construction Technology Laboratories, Inc., Skokie, Illinois), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, January 1992.

34. D. J. Naus and C. B. Oland, *Structural Aging Program Technical Progress Report for Period January 1, 1991, to December 31, 1991*, ORNL/NRC/LTR-92/3, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, February 1992.

35. D. J. Naus et al., "Progress in Research on Aging of Structures," pp. 91–114 in *Proceedings of United States Nuclear Regulatory Commission Nineteenth Water Reactor Safety Information Meeting* held at Marriott Hotel, Bethesda, Maryland, NUREG/CP-0119, **Vol. 3**, April 1992.

36. C. B. Oland, *The Structural Materials Information Center and Its Potential Applications*, ORNL/NRC/LTR-92/8, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, May 1992.

37. G. M. Sabnis and A. T. Kemngang, *Development of a Damage Rating and Repair Prioritization Methodology for Reinforced Concrete Components in Nuclear Power Plants*, Phase I Report, Subcontract No. 11X-SD343V, Howard University, Washington, D. C., June 10, 1992.

38. D. J. Naus et al., "An Improved Basis for Evaluating Continued Service of Category I Concrete Structures in Nuclear Power Plants (NPPs)," pp. 31–39 *in Proceedings of ASME Pressure Vessel and Piping Conference*, PVP-**Vol. 228**, Technical Session 16 on Nuclear Plant Systems/Components Aging Management and Life Extension, New Orleans, Louisiana, June 21–25, 1992.

39. D. J. Naus et al., "Aging of Concrete Containment Structures in Nuclear Power Plants," *Proceedings of Fifth Workshop on Containment Integrity* held at Washington Marriott, Washington, D.C., NUREG/CP-0120 (SAND92-0173), July 1992.

40. K. A. Snyder, J. R. Clifton, and N. J. Carino, *Nondestructive Evaluation of the In-Place Compressive Strength of Concrete Based Upon Limited Destructive Testing,* NISTIR 4874, National Institute of Standards and Technology, U.S. Department of Commerce, Gaithersburg, Maryland, July 1992.

41. D. J. Naus and E. G. Arndt, "An Overview of the Structural Aging Program," pp. 253–275 in *Proceedings of NRC Aging Research Information Conference* held at Holiday Inn–Crowne Plaza, Rockville, Maryland, NUREG/CP-0122, **Vol. 2**, September 1992.

42. C. B. Oland, "Data Base on Structural Materials Aging Properties," pp. 276–298 in *Proceedings of NRC Aging Research Information Conference* held at Holiday Inn–Crowne Plaza, Rockville, Maryland, NUREG/CP-0122, **Vol. 2**, September 1992.

43. B. R. Ellingwood and Y. Mori, "Reliability-Based Condition Assessment of Concrete Structures in Nuclear Power Plants," pp. 299–323 in *Proceedings of NRC Aging Research Information Conference* held at Holiday Inn–Crowne Plaza, Rockville, Maryland, NUREG/CP-0122, **Vol. 2**, September 1992.

44. M. G. Van Geem, *Forty-Two-Year Test Results for PCA Test Series 374 on Long-Term Properties of Concrete*, ORNL/NRC/LTR-92/26 (Subcontract Report 37X-SK412V from Construction Technology Laboratories, Inc., Skokie, Illinois), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 1992.

45. Y. Mori, "Reliability-Based Condition Assessment and Life Prediction of Concrete Structures," Dissertation submitted to The Johns Hopkins University in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Baltimore, Maryland, September 1992.

46. C. B. Oland, *Report of Foreign Travel of C. B. Oland, Engineer, Structural Aging Program, Engineering Technology Division, September 24–October 2, 1992,* ORNL/FTR-4416, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, October 16, 1992.

47. D. J. Naus et al., "An Update on the Structural Aging Program," *Proceedings of United States Nuclear Regulatory Commission Twentieth Water Reactor Safety Information Meeting* held at Marriott Hotel, Bethesda, Maryland, October 28–30, 1992.

48. D. J. Naus and C. B. Oland, "An Update on the Structural Aging Program," *Proceedings of the 6th Meeting: RILEM Technical Committee 104-DDC, Damage Classification of Concrete Structures* held 18-20 May 1992 Haifa, Israel, Published by The National Building Research Institute, Technion-Israel Institute of Technology, I. Soroka (ed.), Technion City Haifa, Israel, 1992.

49. P. Dawson and M. J. M. Wilson, *Surveillance Data for PCPVs at Wylfa, Hartlepool and Heysham I Power Stations*, Report No. 1302/92/5957 (Subcontract No. 31X-SH469V), Taywood Engineering Ltd., London, England, January 1993.

50. S. J. Widdows, *Testing of Aged Concretes from UK Nuclear Power Stations*, Report No. 1303/92/6037 (Subcontract No. 31X-SH469V), Taywood Engineering Ltd., London, England, April 1993.

51. J. Hartley and P. B. Bamforth, *Collation of Survey Data and Review of the Durability Assessment of Reinforced Concrete Structures at Nuclear Power Stations in the UK*, Report No. 1303/92/6163 (Subcontract No. 31X-SH469V), Taywood Engineering Ltd., London, England, April 1993.

52. W. F. Price,. P. B. Bamforth, and G. K. Glass, *Review of European Repair Practice for Corrosion Damaged Reinforced Concrete*, Report No. 1303/91/5823 (Subcontract No. 31X-SH469V), Taywood Engineering Ltd., London, England, April 1993.

53. W. J. Swiat et al., *State-of-the-Art Report — Corrosion of Steel in Concrete*, ORNL/NRC/LTR-93/2 (Subcontract Report 1BX-SK529V from Corrpro Companies, Inc., Medina, Ohio), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, May 1993.

54. D. J. Naus et al., "Continuing the Service of Safety-Related Concrete Structures in Nuclear Power Plants," *Proceedings of the RILEM International Conference on Failures of Concrete Structures,* EXPERTCENTRUM-BRATISLAVA, Strbske Pleso, Slovakia, June 14–18, 1993.

55. D. J. Naus, "Report of Foreign Travel of D. J. Naus, Manager, Structural Aging Program, Engineering Technology Division," Letter DJN/93-53 from D. J Naus, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, to E. G. Arndt, U.S. Nuclear Regulatory Commission, Washington, D.C., July 22, 1993.

56. D. J. Naus et al., "Towards Assuring the Continued Performance of Safety-Related Concrete Structures in Nuclear Power Plants," *Technical Sessions on Nuclear Plant Systems/Components Aging Management and Life Extension*, PVP-**Vol. 252**, ASME Pressure Vessel and Piping Conference, July 25–29, 1993, Denver, Colorado, 1993.

57. G. M. Sabnis and A. T. Kemngang, *Damage Rating and Repair Prioritization of Concrete Structural Components in Nuclear Power Plants*, Subcontract Report 19X-SJ685V from Howard University, Washington, D.C., to Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, July 1993.

58. Y. Mori and B. Ellingwood, "Reliability-Based Condition Assessment of Concrete Structures," *Proceedings of the 6th International Conference on Structural Safety and Reliability*, International Association for Structural Safety and Reliability, Innsbruck, Austria, August 9–13, 1993.

59. D. J. Naus, C. B Oland and E. G. Arndt, "Continuing the Service of Aging Concrete Structures in Nuclear Power Plants," pp. 473-478 in *Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology*, Session DH06 — Concrete and Containment Structures, **Vol. D**, Stuttgart, Germany, August 15–20, 1993.

60. C. B. Oland, D. J. Naus, and S. Jerath, "A Data Base for Aging of Structural Materials," pp. 461-466 in *Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology*, Session DH06 — Concrete and Containment Structures, **Vol. D**, Stuttgart, Germany, August 15–20, 1993.

61. B. R. Ellingwood and Y. Mori, "Role of InService Inspection and Maintenance in Reliability Assurance of Concrete Structures in Nuclear Plants," pp. 87–92 in *Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology*, Session M — Structural Reliability and Probabilistic Safety, **Vol. M**, Stuttgart, Germany, August 15–20, 1993.

62. D. J. Naus and C. B. Oland, *Structural Aging Program Technical Progress for Period January–December 31, 1992*, NUREG/CR-6015 (ORNL/TM-12342), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, July 1993.

63. Y. Mori and B. R. Ellingwood, *Methodology for Reliability-Based Condition Assessment — Application to Concrete Structures in Nuclear Plants*, NUREG/CR-6052 (ORNL/Sub/93-SD684), The Johns Hopkins University, Baltimore, Maryland, August 1993.

64. D. J. Naus, C. B. Oland, and B. R. Ellingwood, "Structural Aging Program Approach to Providing an Improved Basis for Aging Management of Safety-Related Concrete Structures," *Proceedings of United States Nuclear Regulatory Commission Twenty-First Water Reactor Safety Information Meeting* held at Marriott Hotel, Bethesda, Maryland, October 25–27, 1993.

65. Y. Mori and B. Ellingwood, "Time-Dependent System Reliability Analysis by Adaptive Importance Sampling," *Journal of Structural Safety*, 1993.

66. Y. Mori and B. Ellingwood, "Reliability-Based Service Life Assessment of Aging Concrete Structures," pp. 1600–1621, *Journal of Structural Engineering*, American Society of Civil Engineers, **119**(5), 1993.

67. D. J. Naus et al., "An Overview of the ORNL/NRC Program to Address Aging of Concrete Structures in Nuclear Power Plants, pp. 327–329 in *Nucl. Eng. and Design,* **142**, 1993.

68. C. B. Oland, D. J. Naus and E. G. Arndt, "Research and Development at the Structural Materials Information Center," pp. 179–187 in *Nucl. Eng. and Design,* **142**, 1993.

69. B. Ellingwood and Y. Mori, "Probabilistic Methods for Condition Assessment and Life Prediction of Concrete Structures in Nuclear Power Plants," pp. 155–166 in *Nucl. Eng. and Design* **142**, 1993.

70. G. M. Sabnis and A. T. Kemngang, "Damage Rating and Repair Prioritization of Concrete Structures in Nuclear Power Plants," *Proceedings of the 18th Conference on Our World in Concrete & Structures*, Singapore, August 25–27, 1993.

71. D. J. Naus, *Report of Foreign Travel of D. J. Naus March 8–March 15, 1994, Engineering Technology Division,* ORNL/FTR-4924, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 25,1994.

72. P. D. Krauss, *Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants,* ORNL/NRC/LTR-93/28 (Subcontract 90X-SJ024C with Wiss, Janney, Elstner, and Associates, Inc., Northbrook, Illinois), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, March 1994.

73. B. Ellingwood and Y. Mori, *Reliability Assessment of Degrading Concrete Shear Walls,* ORNL/NRC/LTR-94/6, (ORNL/Sub/93-SD684V), (Subcontract 19X-SD684V with The Johns Hopkins University, Baltimore, Maryland), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, April 1994.

74. J. Pachner, T. Tai, and D. J. Naus, "Concrete Aging Study," *Proceedings of the American Power Conference* held at Sheraton Chicago Hotel and Towers April 25–27, 1994, Chicago, Illinois, April 1994.

75. D. J. Naus, C. B. Oland, and B. R. Ellingwood, "Structural Aging Program to Evaluate Continued Performance of Safety-Related Concrete Structures in Nuclear Power Plants," *Proceedings of British Nuclear Energy Society 1994 International Conference on Thermal Reactor Safety Assessment,* held at Ramada Hotel, Manchester, England, May 23–26, 1994.

76. H. Ashar, D. J. Naus, and C. P. Tan, "Prestressed Concrete in U. S. Nuclear Power Plants: Part 1," pp. 58-61 in *Concrete International,* **16**(5), American Concrete Institute, Detroit, Michigan, May 1994.

77. H. Ashar, C. P. Tan and D. J. Naus "Prestressed Concrete in U. S. Nuclear Power Plants: Part 2," pp. 30-34 in *Concrete International*, **16**(6), American Concrete Institute, Detroit, Michigan, June 1994.

78. D. J. Naus, *Report of Foreign Travel of D. J. Naus, May 23–28, 1994, Engineering Technology Division*, ORNL/FTR-4987, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, June 8, 1994.

79. B. Ellingwood and Y. Mori, "Maintaining Reliability of Concrete Structures I: Role of Inspection/Repair, pp. 824–345 in *Journal of Structural Engineering*, **120**(3), American Society of Civil Engineers, New York, 1994.

80. B. Ellingwood and Y. Mori, "Maintaining Reliability of Concrete Structures II: Optimum Inspection/Repair Strategies, pp. 846–862 in *Journal of Structural Engineering*, **120**(3), American Society of Civil Engineers, New York, 1994.

81. D. J. Naus et al., "Aging Management of Containment Structures in Nuclear Power Plants," *Proceedings of Third International Conference on Containment Design and Operation* held at Toronto Hilton October 19–21, 1994, The Canadian Nuclear Society, Toronto, Ontario, Canada, 1994.

82. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, October 6–October 11, 1994*, ORNL/FTR-5167, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, October 31, 1994.

83. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, October 13–October 21, 1994,* ORNL/FTR-5168, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 4, 1994.

84. M. F. Marchbanks and C. B. Oland, *Considerations in the Formulation of a Customized Data Base for Structural Materials*, ORNL/NRC/LTR-94/20, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 1994.

85. C. B. Oland and D. J. Naus, *Summary of Materials Contained in the Structural Materials Information Center*, ORNL/NRC/LTR-94/22, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 1994.

86. D. J. Naus and C. B. Oland, *Structural Aging Program Technical Progress Report for Period January 1, 1993, to June 30, 1994*, ORNL/NRC/LTR-94/21, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 1994.

87. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, February 20, 1995*, ORNL/FTR-5305, Lockheet Martin Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, February 18, 1995.

88. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, April 4–6, 1995,* ORNL/FTR-5335, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, April 16, 1995.

89. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, April 19–21, 1995,* ORNL/FTR-5356, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, May 1, 1995.

90. B. R. Ellingwood, "Reliability-Based Service Life Assessment of Concrete Structures in Nuclear Power Plants: Optimum Inspection and Repair," *Transactions of the 13th International Conference on Structural Mechaincs in Reactor Technology,* Session M – Reliability, Paper 81, Porto Alegre, RS, Brazil, August 1995.

91. D. J. Naus, B. R. Ellingwood, and C. B. Oland, "Extending the Lifespan of Nuclear Power Plant Concrete Structures," *Symposium – Extending the Lifespan of Structures*, International Association for Bridge and Structural Engineers, San Francisco, California, August 23–25, 1995.

92. H. T. Hill, *Concrete Containment Posttensioning System Aging Study*, ORNL/NRC/LTR-95/13, Lockheed Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, August 1995.

93. D. J. Naus, C. B. Oland, and B. R. Ellingwood, "Management of Age-Related Degradation in Nuclear Power Plant Concrete Structures," *Proceedings of Specialists Meeting on the Effectiveness of Methods for Detection and Monitoring of Age Related Degradation in Nuclear Power Plants*, Comisíon Nacional de Energía Atomica of Argentina, Bariloche, Argentina, October 1995.

94. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, September 25–29, 1995*, ORNL/FTR-5584, Lockheed Martin Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, October 13, 1995.

95. D. J. Naus, *Report of Foreign Travel of D. J. Naus, Engineering Technology Division, October 14–21, 1995*, ORNL/FTR-5607, Lockheed Martin Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, October 27, 1995.

96. C. J. Hookham, *In-Service Inspection Guidelines for Concrete Structures in Nuclear Power Plants*, ORNL/NRC/LTR-95/14, Lockheed Martin Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, December 1995.

97. B. R. Ellingwood, *Aging Effects on Probabilistic Risk Assessment*, NUREG/CR-6425 (ORNL/TM-13149), Lockheed Martin Research Corp., Oak Ridge National Laboratory, Oak Ridge, Tennessee, January 1996.

98. D. J. Naus, C. B. Oland, and B. R. Ellingwood, *Final Report on Aging of Reinforced Concrete Structures*, NUREG/CR-6424 (ORNL/TM-13148), Lockheed Martin Energy Research Corp., Oak Ridge National Laboratory, Oak Ridge, Tennessee, January 1996.

INTERNAL DISTRIBUTION

1.	W. G. Craddick	21.	C. E. Pugh
2.	R. G. Gilliland	22.	J. O. Stiegler
3.	F. J. Homan	23.	ORNL Patent Section
4.	H. Jones	24.	Central Research Library
5–12.	D. J. Naus	25.	Document Reference Section
13.	D. G. O'Connor	26–27.	Laboratory Records
14–20.	C. B. Oland	28.	Laboratory Records, ORNL-RC

EXTERNAL DISTRIBUTION

29. H. G. Ashar, Division of Engineering, Office of Nuclear Reactor Regulation, Nuclear Regulatory Commission, Washington, DC 20555-0001
30. G. Bagchi, Division of Engineering, Office of Nuclear Reactor Regulation, Nuclear Regulatory Commission, Washington, DC 20555-0001
31. P. B. Bamforth, Taywood Engineering Limited, Taywood House, 345 Ruislip Road, Southall, Middlesex UB1 2QX, England
32. F. Best, Singleton Laboratory, Louisville, TN 37777
33. R. G. Burg, Construction Technology Laboratories, Inc., 5420 Old Orchard Road, Skokie, IL 60077-1030
34. J. Carey, Electric Power Research Institute, 3412 Hillview Avenue, Palo Alto, CA 94304
35. K. N. Chao, Consumers Power Co., 1945 West Parnell Road, Jackson, MI 49201
36. J. R. Clifton, National Institute of Standards of Technology, Building 226, Room B-348, Gaithersburg, MD 20899
37. Commander and Director, USAE Waterways Experiment Station, ATTN: CEWES-IM-MI-R, Alfreda S. Clark, CD Dept. 1 #1072, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199
38. J. F. Costello, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
39. K. Cozens, NUMARC, 1776 I Street, NW, Washington, DC 20006
40. C. Cragg, Ontario Hydro, 700 University Avenue, Toronto, Ontario MSG 1X6, Canada
41. J. W. Craig, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
42. P. Dawson, Taywood Engineering Limited, Taywood House, 345 Ruislip Road, Southall, Middlesex UB1 2QX, England
43. G. W. DePuy, U.S. Department of Interior, Bureau of Reclamation, P.O. Box 25007, Denver Federal Center, Denver, CO 80225-5989
44. D. R. Eggett, Commonwealth Edison, 1400 Opus Place, Suite 500, Downers Grove, IL 60515

45–55. B. Ellingwood, Professor of Civil Engineering, The Johns Hopkins University, 3400 North Charles St., Baltimore, MD 21218-2699

56. G. Fordyce, HM Principal Inspector, Nuclear Installations Inspectorate, St. Peters House, Balliol Road, Bootle, Merseyside L20 3LZ, England
57. G. Frohnsdorff, National Institute of Standards of Technology, Building 226, Room B-368, Gaithersburg, MD 20899
58. W. L. Gamble, Professor of Civil Engineering, University of Illinois, 3122 Newmark Civil Engineering Laboratory, 205 N. Mathews, Urbana, IL 61801
59. B. N. Grainger, Nuclear Electric Plc., Bedminster Down, Bridgewater Road, Bristol BS123 8AN, England
60–61. H. Graves, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
62. H. K. Hilsdorf, Institute Massivbau and Baustofftechnologie, Universität Karlsruhe (TH), Arbeitlung Baustofftechnologie, Postfach 6380, 7500 Karlsruhe, Germany
63. C. J. Hookham, 2902 Eisenhower, Ann Arbor, MI 48108
64. T. Jávor, Research Institute of Civil Engineering-VUIS, Lamacská Cesta 8, Bratislava, Czechoslovakia
65. D. C. Jeng, Division of Engineering, Office of Nuclear Reactor Regulation, Nuclear Regulatory Commission, Washington, DC 20555-0001
66. S. Jerath, Professor of Civil Engineering, Box 8115, University Station, Grand Forks, ND 58202
67. R. Judge, AEA Technology, Bldg. 329, Harwell, Didcot, Oxford, 0X11 0RA, United Kingdom
68. D. Kluge, Swiss Federal Nuclear Safety Inspectorate (HSK), CH-5303, Würenlingen, Switzerland
69. H. C. Kosel, Construction Technology Laboratories, Inc., 5420 Old Orchard Road, Skokie, IL 60077-1030
70. P. Krauss, Wiss, Janney, Elstner Associates, Inc., 330 Pfingsten Road, Northbrook, IL 60062-2095
71. K. R. Lauer, Professor of Civil Engineering, University of Notre Dame, Notre Dame, IN 46556-0767
72. T. C. Liu, Chief, Materials Engineering Section, Headquarters, U.S. Army Corps of Engineers, 20 Massachusetts Ave., NW, HQUSACE (CECW-EG), Washington, DC 20314
73. D. G. Manning, Central Management Office, Ontario Ministry of Transportation, 1201 Wilson Avenue, 2nd Floor, West Bldg., Downsview, Ontario M3M 1J8, Canada
74. H. W. Massie, Jr., Defense Nuclear Facilities Safety Board, 625 Indiana Ave., NW, Suite 700, Washington, DC 20004
75. M. E. Mayfield, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
76. W. McCleese, U.S. Army Corps of Engineers, Waterways Experiment Station, (CEWES-SC-A), 3909 Halls Ferry Road, Vicksburg, MS 39180-6199
77. R. Mills, Professor of Civil Engineering, University of Toronto, Toronto, Ontario M5S 1A4, Canada
78. P. K. Mukherjee, Ontario Hydro, Ontario Hydro Research Division, Civil Research Department, 800 Kipling Avenue, KR 163, Toronto, Ontario M8Z 5S4, Canada
79. K. F. Müller, Akadem Director, Materialprüfungsant fur des Bauwesen, Der Technischen Universität München, 800 München 2, Archistrasse 21, Postfach 20 24 20, Postgirokonto, München 9747-807, Germany
80. A. Murphy, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
81. W. Norris, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
82. J. Philip, Division of Regulatory Applications, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001

83. M. K. Ravindra, EQE International, 44 Montgomery Street, Suite 3200, San Francisco, CA 94104-4805
84. J. W. Reed, Jack R. Benjamin and Assoc., 444 Castro St., Suite 501, Mountain View, CA 94041
85. R. F. Sammataro, Proto-Power Corporation, 591 Poquonnock Road, Groton, CT 06340
86. R. W. Seidensticker, Argonne National Laboratory, 9700 S. Cass Avenue, Argonne, IL 60439
87. S. P. Shah, Center for ACBM, Northwestern University, 1800 Ridge Avenue, Evanston, IL 60208-4400
88. L. C. Shao, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
89. R. E. Shewmaker, Office of Nuclear Materials Safety & Safeguards, Low-Level Waste Management Branch, Nuclear Regulatory Commission, Washington, DC 20555-0001
90. C. Sjöström, Head, Materials and Structures Division, The National Swedish Institute of Building Research, P.O. Box 785, S-801 29 G, Sweden
91. G. Sommerville, Director, C & CA Services, BCA, Wexham Springs, Slough SL3 PL, England
92. I. Soroka, National Building Research Institute, Ministry of Construction and Housing, Technion-Israel Institute of Technology, Technion City, Haifa 32000, Israel 32000
93. W. J. Swiat, CORRPRO Companies, Inc., P.O. Box 1179, Medina, OH 44258
94. M. Vagins, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
95. J. P. Vora, Division of Engineering Technology, Office of Nuclear Regulatory Research, Nuclear Regulatory Commission, Washington, DC 20555-0001
96. R. Winkel, Westinghouse Hanford Company, P.O. Box 1970-HF-57, Richland, WA 99352
97. P. Zwicky, Basler & Hofmann, Forchstrasse 395, CH-8029 Zurich, Switzerland
98. Office of Assistant Manager for Energy Research and Development, DOE-OR, Oak Ridge, TN 37831

99–100. Office of Scientific and Technical Information, P.O. Box 62, Oak Ridge, TN 37831

NRC FORM 335 (2-89) NRCM 1102, 3201, 3202

U.S. NUCLEAR REGULATORY COMMISSION

BIBLIOGRAPHIC DATA SHEET

(See instructions on the reverse)

1. REPORT NUMBER *(Assigned by NRC. Add Vol., Supp., Rev., and Addendum Numbers, if any.)*

NUREG/CR-6424
ORNL/TM-13148

2. TITLE AND SUBTITLE

Report on Aging of Nuclear Power Plant Reinforced Concrete Structures

3. DATE REPORT PUBLISHED

MONTH	YEAR
March	1996

4. FIN OR GRANT NUMBER

B0845

5. AUTHOR(S)

D.J. Naus, C.B. Oland, ORNL
B.R. Ellingwood, JHU

6. TYPE OF REPORT

7. PERIOD COVERED *(Inclusive Dates)*

8. PERFORMING ORGANIZATION — NAME AND ADDRESS *(If NRC, provide Division, Office or Region, U.S. Nuclear Regulatory Commission, and mailing address; if contractor, provide name and mailing address.)*

Oak Ridge National Laboratory
Oak Ridge, Tennessee 37831-6285

Subcontractor:
The Johns Hopkins University
3400 North Charles Street
Baltimore, Maryland 21218-2699

9. SPONSORING ORGANIZATION — NAME AND ADDRESS *(If NRC, type "Same as above"; if contractor, provide NRC Division, Office or Region, U.S. Nuclear Regulatory Commission, and mailing address.)*

Division of Engineering Technology
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

10. SUPPLEMENTARY NOTES

H. Graves, NRC Project Manager

11. ABSTRACT *(200 words or less)*

The Structural Aging Program provides the U.S. Nuclear Regulatory Commission with potential structural safety issues and acceptance criteria for continued service assessments of safety-related nuclear power plant concrete structures. The program was organized under four task areas — Program Management, Materials Property Data Base, Structural Component Assessment/Repair Technology, and Quantitative Methodology for Continued Service Determinations. Under these tasks, over 90 papers and reports were prepared addressing pertinent aspects associated with aging management of nuclear power plant reinforced concrete structures. Contained in this report is a summary of program results in the form of information related to longevity of nuclear power plant reinforced concrete structures, a data base presenting data and information on the time variation of concrete materials under the influence of environmental stressors and aging factors, in-service inspection and condition assessments techniques, repair materials and methods, evaluation of nuclear power plant reinforced concrete structures and a reliability-based methodology for current and future condition assessments. Recommendations for future activities are also provided.

12. KEY WORDS/DESCRIPTORS *(List words or phrases that will assist researchers in locating the report.)*

Aging Management
Concrete Condition Assessment
Concrete Containments
Degradation Models
Electronic Data Base
In-Service Inspection
Materials Handbook
Non-Destructive Evaluation
Prestressed Concrete
Reinforced Concrete
Repair
Service Life
Structural Reliability

13. AVAILABILITY STATEMENT

unlimited

14. SECURITY CLASSIFICATION

(This Page) unclassified

(This Report) unclassified

15. NUMBER OF PAGES

16. PRICE

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, DC 20555-0001

OFFICIAL BUSINESS
PENALTY FOR PRIVATE USE, $300

SPECIAL FOURTH-CLASS MAIL
POSTAGE AND FEES PAID
USNRC
PERMIT NO. G-67

W. E. Norris
Mail Stop T10-L1