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STRUCTURAL AGING PROGRAM TECHNICAL PROGRESS REPORT FOR PERIOD JANUARY 1, 1993, TO JUNE 30, 1994

D. J. Naus
C. B. Oland

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November 1994

Prepared for the
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Office of Nuclear Regulatory Research
Under Interagency Agreement No. 1886-8084-5B

NRC JCN No. B0845

Prepared by the
OAK RIDGE NATIONAL LABORATORY
Oak Ridge, Tennessee 37831
managed by
MARTIN MARIETTA ENERGY SYSTEMS, INC.
for the
U.S. Department of Energy
under Contract No. DE-AC05-84OR21400

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STRUCTURAL AGING PROGRAM TECHNICAL PROGRESS REPORT FOR PERIOD JANUARY 1, 1993, TO JUNE 30, 1994

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C. B. Oland

ABSTRACT

The Structural Aging (SAG) Program has the objective of preparing a document that will provide the Nuclear Regulatory Commission (NRC) with potential structural safety issues and acceptance criteria for use in nuclear power plant continued service evaluations. Program focus is on concrete and concrete-related materials comprising safety-related (Category I) structures in light-water reactor facilities. The program is organized into four tasks: Program Management, Materials Property Data Base, Structural Component Assessment/Repair Technology, and Quantitative Methodology for Continued Service Determinations. During this reporting period, 28 materials were added to the Structural Materials Information Center bringing the total to 144; long-term concrete properties were developed by testing core samples removed from U.S. nuclear power facilities and cylinders cast in conjunction with construction of several United Kingdom (UK) nuclear power stations; a performance assessment of concrete structures in several UK nuclear power facilities was completed; activities were initiated to identify potential issues related to aging of post-tensioning systems in nuclear power plants; results of a survey to identify in-service inspection procedures and performance of concrete structures in U.S. nuclear power plants were analyzed and published in the report presenting an overview of repair procedures for reinforced concrete structures; European repair practices for corrosion-damaged reinforced concrete structures were reported; results of an assessment of corrosion of metals in concrete were provided in a report; the initial study addressing damage assessment and repair prioritization of reinforced concrete structural elements in nuclear power plants was completed and activities initiated to quantitatively relate repair requirements to damage state and environmental exposure; and the time-dependent reliability methodology was applied to reinforced concrete flexure and shear structural elements to investigate in-service inspection and maintenance strategies to maintain failure probability below a specified target value. In addition, an annual technical progress report was published; program personnel participated in the NRC Water Reactor Safety Information Meeting, National Institute of Standards and Technology 1993 Building Technology Symposia Series and Symposium on Integrated Knowledge Systems for High-Performance Construction Materials, International Atomic Energy Agency Coordinated Research Program on Management of Aging of Concrete Containment Buildings, 1993 American Society of Mechanical Engineers Pressure Vessel & Piping Conference, 6th International Conference on Structural Safety and Reliability, 12th International Conference on Structural Mechanics in Reactor Technology, 18th Conference on Our World of Concrete & Structures, Workshop on Concrete Performance and Modeling for Low-Level Radioactive Waste Disposal, and British Nuclear Energy Society International Conference on Thermal Reactor Safety Assessment; 10 technical reports were published; 10 papers were published in conference proceedings (2 accepted for publication); 3 foreign trip reports were prepared; 9 journal articles were published; 14 presentations were made at national or international technical meetings; 3 program briefings were made; and program personnel participated as members in 6 technical committees. Appendix A presents a summary of accomplishments under each of the major task areas of the SAG Program.

1. INTRODUCTION

The Structural Aging (SAG) Program has the overall objective of preparing a document that will provide the Nuclear Regulatory Commission (NRC) license reviewers 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 predicting future, structural safety margins. The results of this study will provide an improved basis for the NRC staff to evaluate NPPs for continued service. More specifically, potential regulatory applications of this research include (1) improved predictions of long-term material and structural performance and available safety margins at future times, (2) establishment of limits on exposure to environmental stressors, (3) reduction in total reliance by licensing on inspection and surveillance through development of a methodology that will enable the integrity of structures to be assessed (either pre- or post-accident), and (4) improvements in damage inspection methodology through potential incorporation of program results into national standards that could be referenced by Standard Review Plans.

The SAG Program consists of three technical tasks and a management task. The organization of these tasks is illustrated by the Level 2 work breakdown structure for the program shown in Fig. 1.1 An overall representation of selected key activities and major milestones for each of the task areas, as well as their interrelationship, is presented in Fig. 1.2. The tasks interface with each other and are augmented by documentation and technology transfer activities. The remainder of this document is arranged according to the work breakdown structure shown in Fig. 1.1 with a chapter in this report addressing each of the four tasks. A work breakdown structure has also been developed for each task and is displayed in the corresponding chapter of this document. Each chapter also contains a brief objective for that task, a background discussion, a plan-of-action, summary of accomplishments for the reporting period, planned activities, and milestone statements and schedule.

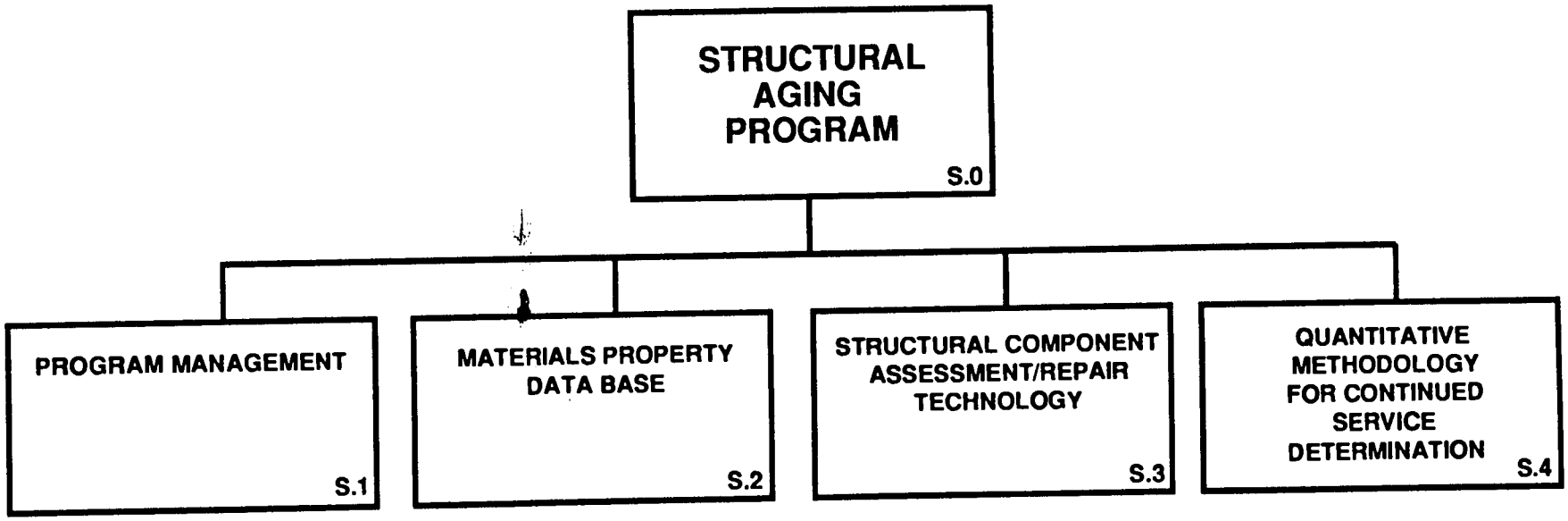

Fig. 1.1 Major work breakdown structure for the Stuctural Aging Program.

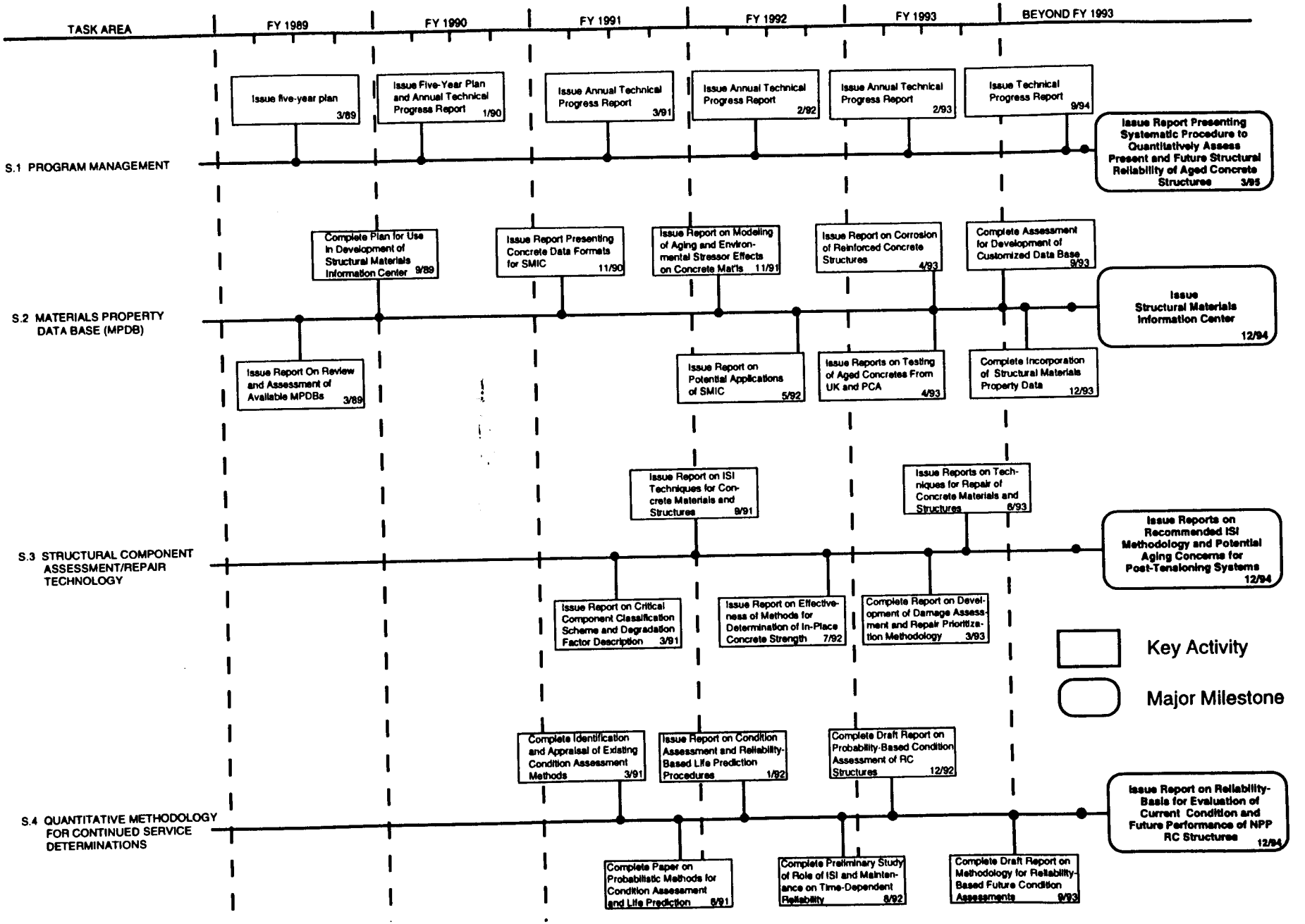

Fig. 1.2 Selected key activities and major milestones for each task of the SAG Program.

2. TASK S.1 – PROGRAM MANAGEMENT

2.1 OBJECTIVE

The overall objective of the program management task is to effectively manage the technical tasks undertaken to address priority structural safety issues related to continuing the service of nuclear power plants. Management duties include planning, integrating, monitoring, reporting, and technology transfer. A key part of the management function is the integration of the technical objectives and the efforts of various program participants. Figure 2.1 presents a work breakdown structure for the management task.

2.2 BACKGROUND

The SAG Program is administratively carried out through the Engineering Technology Division of Oak Ridge National Laboratory (ORNL). A key part of the management function is the integration of the technical objectives and the efforts of various program participants. The participants include other divisions of ORNL, other national and international laboratories, and universities and industrial organizations. The aim is to utilize capabilities and facilities in a complementing way to meet program objectives. Figure 2.2 illustrates the placement of the program within the ORNL organization and lists key staff members (as of June 1994).

Concerning technical integration, a strong relationship continues to be made with peer groups and committees. These include the American Society of Mechanical Engineer's (ASME) Boiler and Pressure Vessel Code Section XI Subcommittee; American Concrete Institute (ACI); and American Society for Testing and Materials (ASTM) Standard's Groups. In addition, efforts have continued to exchange information with other North American, European, and Asian researchers and pertinent research groups such as Fédération Internationale de la Précontrainte (FIP), International Union of Testing and Research Laboratories for Materials and Structures (RILEM), International Atomic Energy Agency (IAEA) Coordinated Research Program on Management of Aging of Concrete Containment Buildings, European Committee on Concrete (CEB), International Council for Building Research Studies and Documentation (CIB), etc.

Program developments are transferred to the technical community through progress and topical reports, program review meetings, information meetings, open-literature papers, and committee participation. Informal technical interactions with several foreign organizations have been established and every effort will be expended to enhance and expand these relationships.

2.3 PLAN OF ACTION

The plan for Task S.1 is to effectively manage the program. The Level 3 work breakdown structure for this task is shown in Fig. 2.3. The three subtasks present a more detailed breakdown of the management functions performed. The specific activities and schedules within each subtask are presented later in this chapter (Sect. 2.7) in the form of milestone charts. In addition to planning, monitoring, and reporting on the program performance in a timely manner, efforts are continuously made to maintain effective technology transfer and to maintain liaison with peer groups, committees, and programs in foreign countries.

This task administers the research and development subcontracts and technical consulting agreements that are required to supplement the ORNL work. The subcontractor progress reports are integrated into the overall program technical progress reports.

Figure 2.4 presents a listing of subcontracted activities under each of the program's primary task areas as of June 1994.

The monthly progress report is part of a monthly highlights report for the NRC Office of Nuclear Regulatory Research issued by the ORNL NRC Programs Office. An associated part of this reporting procedure is the preparation of a technical progress report.

2.4 SUMMARY OF ACCOMPLISHMENTS

During the reporting period covered by this document, activities have been conducted under each of the subtask areas listed in Fig. 2.1 for Task S.1 (Program Management).

2.4.1 Program Planning and Resource Allocation (Subtask S.1.1)

Under this subtask, the program's FY 1994 Project and Budget Proposal was prepared. Five subcontracts were completed (Taywood Engineering Limited; Wiss, Janney, Elstner, Associates; Corrpro Companies; Sargent & Lundy Engineers; and Howard University), one existing subcontract was administered (Johns Hopkins University); and two subcontracts were initiated (Mr. C. Hookham and Dr. H. Hill). Activities under each of these subcontracts are discussed later in this document under the appropriate program task (see Fig. 2.4).

2.4.2 Program Monitoring and Control (Subtask S.1.2)

Program monitoring and control activities have included the preparation of monthly management reports for inclusion in the highlights report prepared for the NRC Office of Nuclear Regulatory Research and issued by the ORNL NRC Programs Office. An annual technical progress report was prepared (Ref. 2.1) and a program overview paper was presented at the Twenty-First Water Reactor Safety Information Meeting (Ref. 2.2).

2.4.3 Documentation and Technology Transfer (Subtask S.1.3)

During this reporting period, 10 technical reports were published, 10 papers were published in conference proceedings (2 accepted for publication), 9 journal articles were published, 14 presentations were made at national or international technical meetings, and 3 program briefings were made. Appendices B and C present listings of reports/papers and presentations, respectively.

Program personnel participated in technical committees of the ACI (Service Life Prediction, Radioactive and Hazardous Waste Management, and Concrete Materials Property Database), ASME (Section XI Working Group on Plant Life Extension, Working Group on Concrete Pressure Components, and Subgroup on Containment), RILEM (Damage Classification of Concrete Structures and Methodology for Life Prediction of Concrete Structures in Nuclear Power Plants), and Fédération Internationale de la Précontrainte (Prestressed Concrete Pressure Vessels and Containment Structures).

The program was coordinated with other government agencies-related activities, for example, Low-Level Radioactive Waste and Nuclear Plant Aging Research programs sponsored by the NRC. Discussions continued with the utility sector of the nuclear power industry (e.g., Electric Power Research Institute), as well as with several utilities (e.g., Consumers Power and Commonwealth Edison), national organizations, universities, and consulting/engineering organizations. Table 2.1 presents a listing of domestic organizations that have been contacted. Program-related overview papers were presented at the National Institute of Standards and Technology 1993 Building Technology Symposia Series (Ref. 2.3), Electric Power Research Institute Life Cycle Management Subcommittee

Meeting (Ref. 2.4), ASME Pressure Vessels and Piping Conference (Ref. 2.5), Workshop on Concrete Performance and Modeling for Low-Level Radioactive Waste Disposal (Ref. 2.6) and National Institute of Standards and Technology Symposium on Integrated Knowledge Systems for High-Performance Construction Materials and Systems (Ref. 2.7).

Technology exchange has continued at the international level. Program personnel participated in the IAEA Coordinated Research Program on Aging of Concrete Containment Buildings (Ref. 2.8). The objectives of this three-year activity are to produce a summary report on current aging-management practices and experience, compile a state-of-the-art report on concrete repair techniques and materials, develop crack mapping techniques and acceptance/repair guidelines, and formulate a set of condition indicators for monitoring aging. In order to meet these objectives a questionnaire was developed and sent by IAEA in March 1994 to nuclear power plant operators. The questionnaire consisted of three primary sections: (1) general plant information, (2) inspection/investigation and preventative maintenance programs, and (3) aging-related degradation experience. Evaluation of responses to the questionnaire is scheduled for September 1994. Program-related presentations were made at the International Conference on Failures of Concrete Structures (Ref. 2.9), International Atomic Energy Agency (Ref. 2.10), 6th International Conference on Structural Safety and Reliability (Ref. 2.11), 12th International Conference on Structural Mechanics in Reactor Technology (Ref. 2.12 – 2.15), 18th Conference on Our World of Concrete & Structures (Ref. 2.16), Electricite de France (Ref. 2.17), British Nuclear Energy Society International Conference on Thermal Reactor Safety Assessment (Ref. 2.18), and Nuclear Electric (Ref. 2.19). Two foreign trips were made to visit cognizant research organizations in Germany, France, and England (Refs. 2.20 and 2.21). Information exchange continued with several European organizations: Fédération Internationale de la Précontrainte, International Union of Testing and Research Laboratories for Materials and Structures, International Atomic Energy Agency, and Commission of European Communities. Contacts have been made with 114 foreign organizations in 28 countries in pursuit of technology exchange. Table 2.2 presents a summary of foreign organizations with whom SAG Program personnel have been in communication.

2.5 PLANNED ACTIVITIES

Planned activities will in large part be directed at bringing program technical activities to a timely conclusion. The planning, organization, performance, and reporting of the technical tasks will be managed in keeping with approved program plans. The monthly reporting system will be continued and the Project and Budget Proposal (189) will be developed and issued. Selection and administration of subcontracts will continue. Participation in technical committees (ACI, ASME, RILEM, etc.), and industry and NRC nuclear plant aging management programs will continue. Efforts will continue to establish technology transfer activities both domestically and to foreign countries in keeping with NRC guidelines.

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2.21 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.

2.7 MILESTONE STATEMENT AND SCHEDULE

The statement and schedule for the milestones in Task S.1 are presented in the following charts.

MILESTONE STATEMENT AND SCHEDULE

TASK: S.1 PROGRAM MANAGEMENT

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.1 PROGRAM MANAGEMENT																							
S.1.1 Program Planning and Resource Allocation																							
A. Issue Project and Budget Proposal (189)	▲					▲											△						
B. Select and Administer Subcontract Activities on an Annual Basis																		△					
S.1.2 Program Monitoring and Control																							
A. Provide Input to Monthly Highlights Report (by fifth work day of subsequent month)																			△				
B. Issue Technical Progress Report			▲														△						

MILESTONE STATEMENT AND SCHEDULE

TASK: S.1 PROGRAM MANAGEMENT

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.1.3 Documentation and Technology Transfer																			△				
A. Participate (as requested) in NRC Reviews and Annual Information Meetings																			△				
B. Participate in Technical Committee and Industry Activities Related to Continued Service of Nuclear Power Plants																			△				
C. Coordinate Program Tasks with other Ongoing Programs such as Those Sponsored by DOE, EPRI and Private Sector																			△				
D. Exchange Technology with Foreign Countries as Approved by NRC																				△			
E. Issue Final Report on Condition Assessment of Reinforced Concrete Structures in Nuclear Power Plants																							

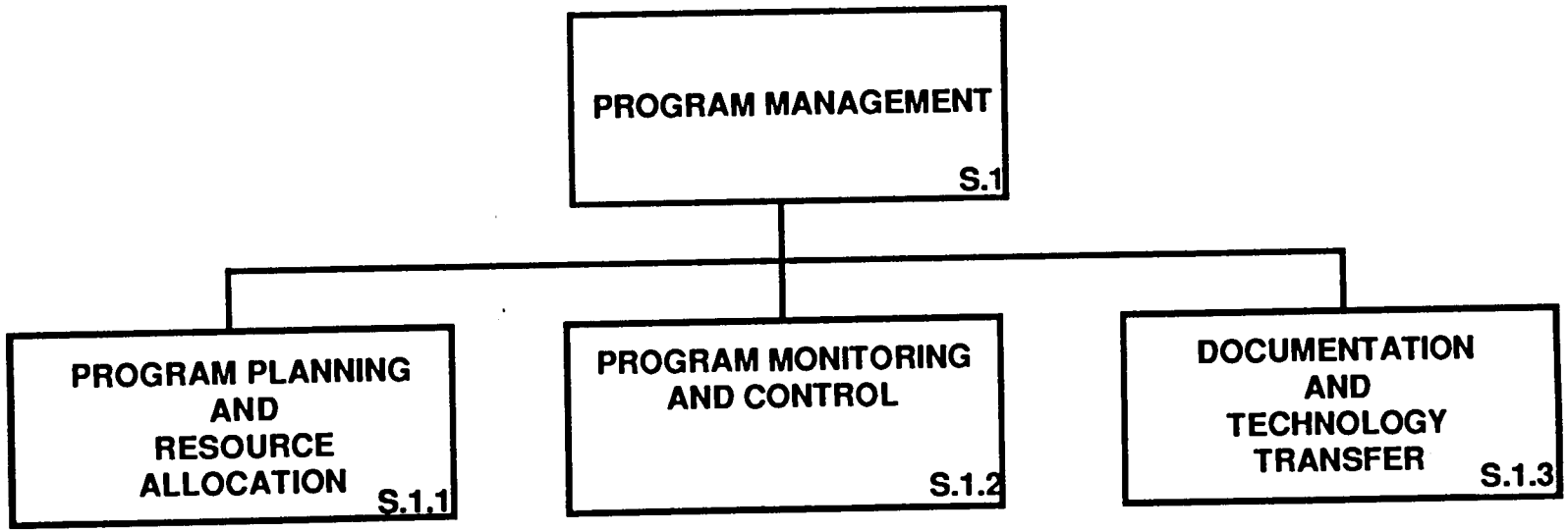

Fig. 2.1 Level 2 work breakdown structure for SAG Task S.1: Program Management.

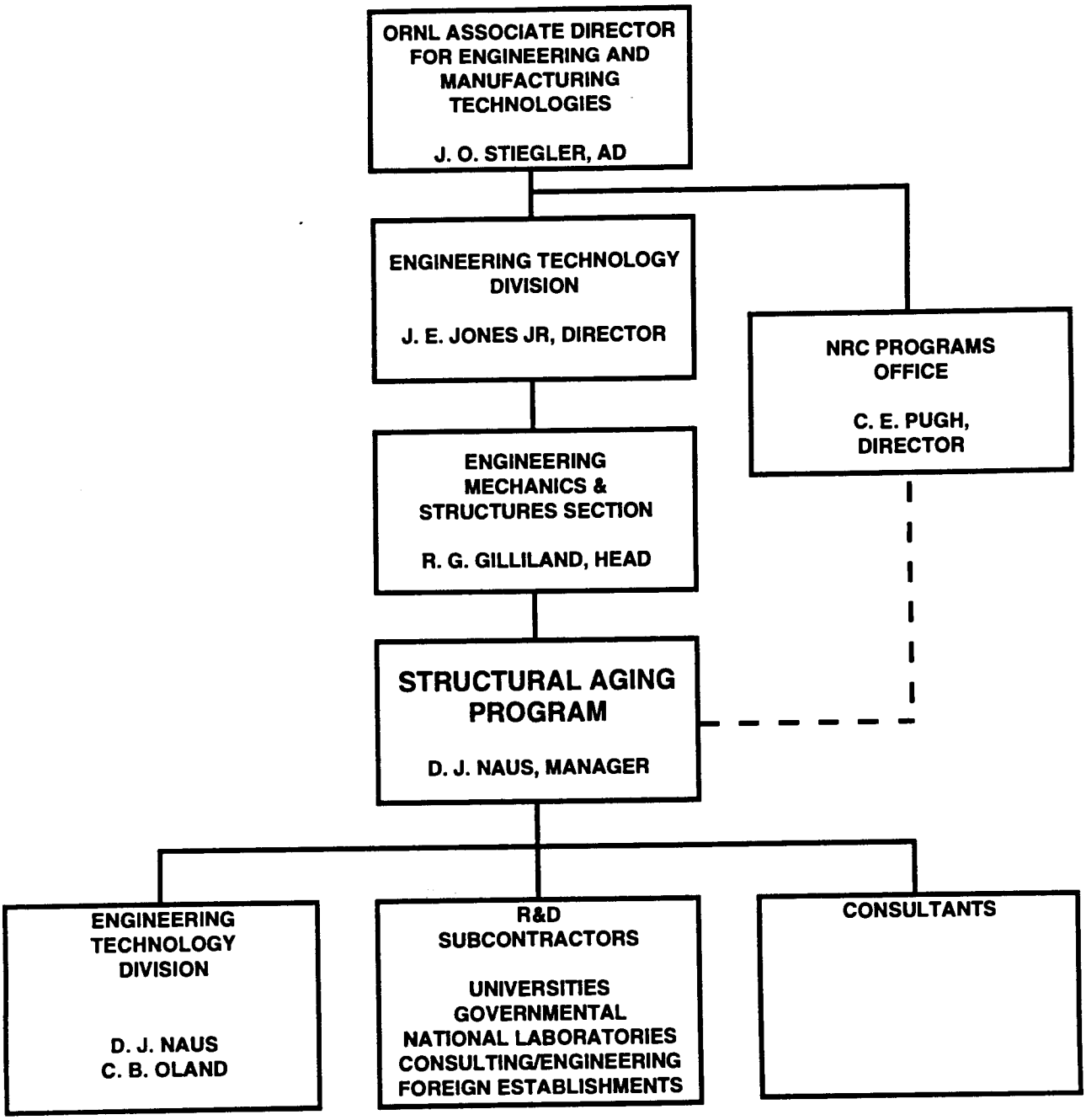

Fig. 2.2. Organization of the Structural Aging Program.

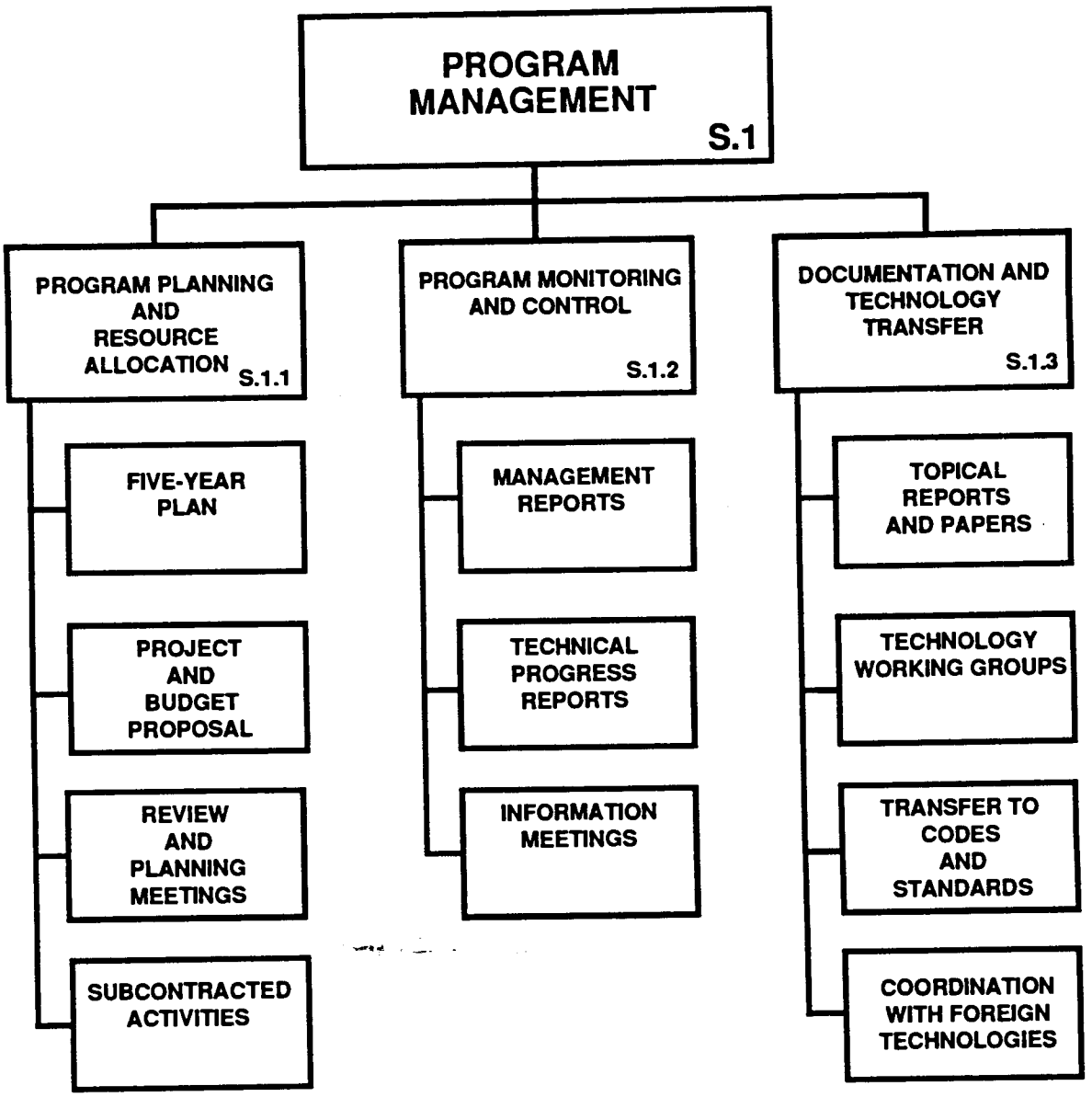

Fig. 2.3 Level 3 work breakdown structure for SAG Task S.1: Program Management.

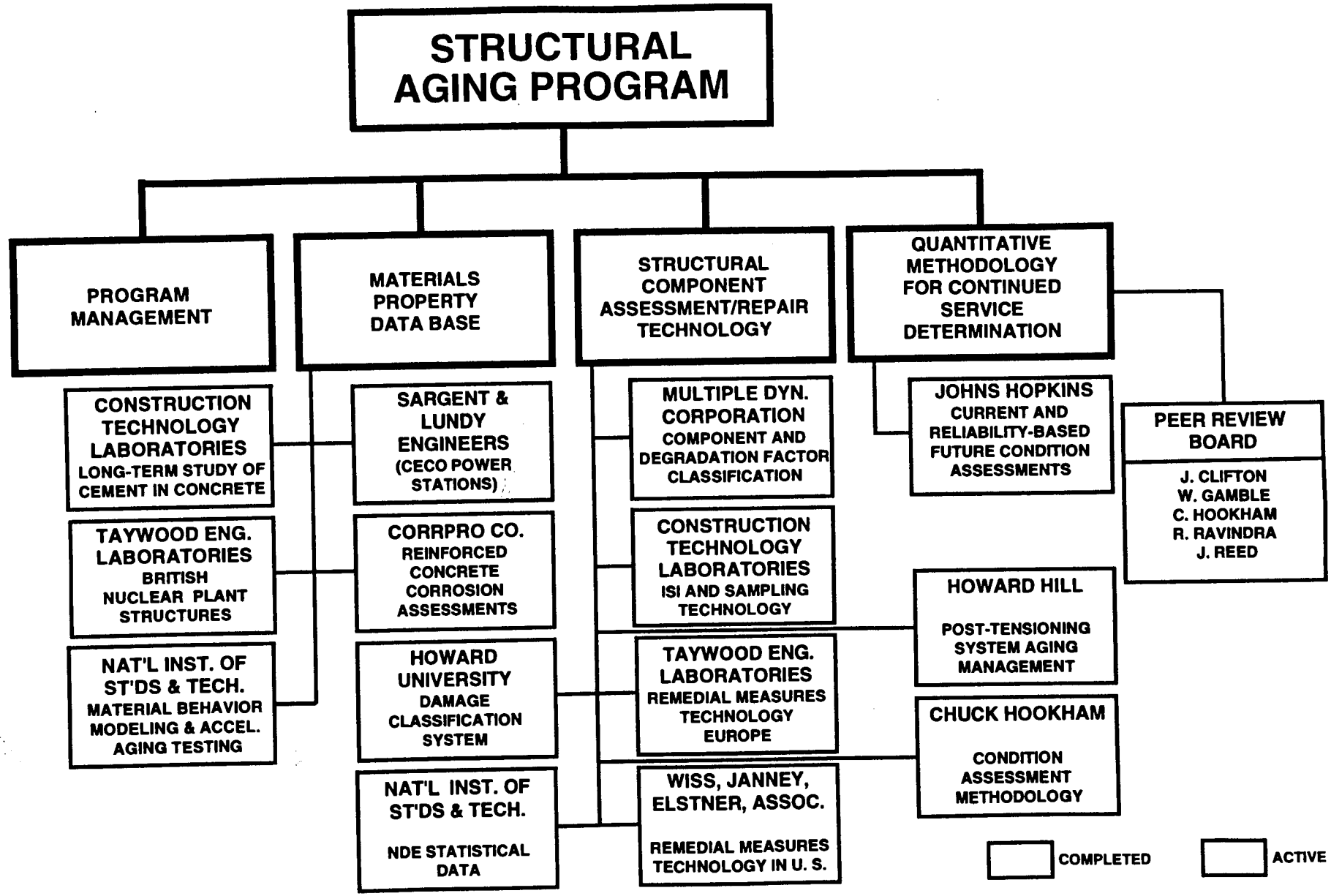

Fig. 2.4. Listing of subcontracted activities for each primary task area.

Table 2.1 SAG program contacts with domestic organizations.

Type	Organization	Location
Governmental	Argonne National Laboratory	Argonne, Ill.
	Argonne National Laboratory-West	Idaho Falls, Idaho
	Battelle Pacific Northwest Laboratories	Richland, Wash.
	Bureau of Reclamation	Denver, Colo.
	Corps of Engineers - Headquarters	Washington, D.C.
	Corps of Engineers - Waterways Experiment Station	Vicksburg, Miss.
	Defense Nuclear Facilities Safety Board	Washington, D.C.
	Federal Highway Administration	Washington, D.C.
	Hanford Engineering Development Laboratory	Richland, Wash.
	Idaho National Engineering Laboratory	Idaho Falls, Idaho
	National Institute of Standards and Technology	Gaithersburg, Md.
	Navfac	Alexandria, Va.
	Sandia National Laboratory	Albuquerque, NM.
	Savannah River Laboratories	Aiken, S.C.
	Strategic Highway Research Program	Washington, D.C.
	US DOE Reactor & Nuclear Fuels Corporation	Toyko, Japan
	US DOE Headquarters	Germantown, Md.
National	American Concrete Institute	Detroit, Mich.
	American Institute of Mining, Metallurgical and Petroleum Engineers	Warrendale, Penn.
	American Institute of Steel Construction	Chicago, Ill.
	American Iron and Steel Institute	Washington, D.C.
	American Nuclear Society	LaGrange Park, Ill.
	American Society of Civil Engineers	New York, N.Y.
	American Soc. of Mechnical Engineers	New York, N.Y.
	American Society for Testing and Mat'ls	Philadelphia, Penn.
	ASM International	Materials Park, Ohio
	Concrete Reinforcing Steel Institute	Schaumburg, Ill.
	CORRPRO Companies, Inc.	Medina, Ohio
	DYWIDAG Systems Int'l	Lincoln Park, N.J.
	ERICO Products, Inc.	Cleveland, Ohio
	Martin Marietta	Baltimore, Md.
	Master Builders	Cleveland, Ohio
	NUS Corporation	Gaithersburg, Md.
	National Assoc. of Corrosion Engineers	Houston, Tex.
	Prestressed Concrete Institute	Chicago, Ill.
	Post-Tensioning Institute	Phoenix, Ariz.
	U.S. Steel	Pittsburgh, Pa.
	Wire Reinforcing Institute	McLean, Va.

Table 2.1 (cont.) SAG program contacts with domestic organizations.

Type	Organization	Location
Consulting/Engineering	BCP Technical Services	Novato, Ca.
	Bechtel Power Corporation	Gaithersburg, Md.
	Bechtel Western	Norwalk, Calif.
	Jack R. Benjamin and Assoc.	Mountain View, Calif.
	T. M. Brown	Bartlett, Ill.
	Burns & Roe	Oradell, N.J.
	Construction Technology Laboratories	Skokie, Ill.
	Consulting Engineers Group	Mt. Prospect, Ill.
	Di Benedetto Associates	North Andover, Mass.
	Dixon & Associates	Stone Mountain, Ga.
	EQE Engineering	San Francisco, Calif.
	N. FitzSimons	Rockville, Md.
	Fluor Daniels, Inc.	Irvine, Calif.
	General Dynamics Services	Groton, Conn.
	General Electric (Vallecitos)	Pleasanton, Calif.
	Innovative Technologies	Monroeville, Pa.
	Metropolitan Water District	Los Angeles, Calif.
	Multiple Dynamics Corporation	Southfield, Mich.
	Newtech Resources	Costa Mesa, Calif.
	NSF Center for Science and Technology of Advanced Cement-Based Materials	Evanston, Ill.
	Olson Engineering Inc.	Lakewood, Colo.
	PDI Technology	Brea, Calif.
	PSC	East Chicago, Ind.
	Raths, Raths & Johnson	Willowbrook, Ill.
	Sargent & Lundy Engineers	Chicago, Ill.
	Singleton Laboratories	Louisville, Tenn.
	Stone & Webster Engineering Corp.	Boston, Mass.
	Teledyne Engineering Services	Jupiter, Fla.
	United Enginers and Constructors	Philadelphia, Pa.
	Westinghouse Scientific & Technology Center	Pittsburgh, Pa.
	Wiss, Janney, Elstner and Assoc.	Northbrook, Ill.

Table 2.1 (cont.) SAG program contacts with domestic organizations.

Type	Organization	Location
Universities	Bradley University	Peoria, Ill.
	California Institute of Technology	Pasadena, Calif.
	Cornell University	Ithaca, NY
	Howard University	Washington, D.C.
	Johns Hopkins University	Baltimore, Md.
	Michigan State University	East Lansing, Mich.
	Northwestern University	Evanston, Ill.
	Pennsylvania State University	University Park, Pa.
	Purdue University	West Lafayette, Ind.
	Rice University	Houston,Tex.
	Texas A & M University	College Station, Tex.
	University of California	Berkeley, Calif.
	University of Illinois	Urbana-Champaign, Ill.
	University of North Dakota	Grand Forks, N.D.
	University of Notre Dame	Notre Dame, Ind.
	University of Wisconsin	Madison, Wis.
	University of Wisconsin	Milwaukee, Wis.
Utility	Boston Edison	Boston, Mass.
	Commonwealth Edison Companies	Chicago, Ill.
	Consumers Power	Jackson, Mich.
	Dairyland Power	LaCrosse, Wis.
	Detroit Edison	Detroit, Mich.
	Duke Engineering Services	Charlotte, N.C.
	Electric Power Research Institute	Palo Alto, Ca.
	Metropolitan Water District	Los Angeles, Ca.
	Northeast Utilities Service Company	Hartford, Conn.
	Northern States Power	Minneapolis, Minn.
	Nuclear Management and Resources Council	Washington, D.C.
	Southern California Edison	San Diego, Ca.
	Southern Company Services	Birmingham, Ala.
	Virginia Power Company	Glen Ellen, Va.

Table 2.2 SAG program contacts with foreign organizations.

Country	Organization	Location
Australia	Cement and Concrete Assoc. of Australia	Sydney
	Institute of Minerals, Energy & Construction	Highett, Victoria
Austria	International Atomic Energy Agency	Vienna
Belgium	Association Européene du Ciment	Brussels
	Center National de Recherches Scientifiques et Techniques pour l' Industrie Cimentiere	Brussels
	Center Scientifique et Technique de la Construction	Brussels
	Studiecentrum voor Kernenergie	Mol
Canada	Atomic Energy of Canada Ltd.	Pinawa, Manitoba
	Canada Center for Mineral and Energy Technology	Ottawa, Ontario
	Chalk River Laboratory	Chalk River, Ontario
	Ministry of Transportation	Downsview, Ontario
	National Research Council of Canada	Ottawa, Ontario
	Ontario Hydro	Toronto, Ontario
	University of Saskatchewan	Saskatoon, Saskatchewan
	University of Toronto	Toronto, Ontario
Czechoslovakia	Brno Technical University	Brno
	Research Institute of Civil Engineering-VUIS	Bratislava
	Technical University	Kosice
Denmark	G. M. Idorn Consultant A/S	Birkerod
	Technical University of Denmark	Lyngby
	The Road Directorate-Ministry of Transport	Copenhagen
England	AEA Technology	Oxfordshire
	British Cement Assoc.	Wexham Springs, Slough
	Building Research Establishment	Garston, Watford
	Central Electricity Generating Board	Barnwood, Gloucester
	Central Electricity Generating Board	Leatherhead, Surrey
	Construction Industry Research and Information Assoc.	London
	Fédération Internationale de la Précontrainte	London
	Imperial College	London
	Institution of Civil Engineers	London
	Institution of Structural Engineers	London
	Nuclear Electric	Bedminster Down, Bristol

Table 2.2 (cont.) SAG program contacts with foreign organizations.

Country	Organization	Location
England (cont.)	Nuclear Installations Inspectorate	Bootle, Merseyside
	Taywood Engineering Ltd.	Southall, Middlesex
	The Concrete Society	London
	University of Bristol	Bristol
Federal Republic of Germany	Beuth Verlag GmbH	Berlin
	Bundesanstalt für Materialforschung und Prüfung (BAM)	Berlin
	Deutscher Auschuss für Stahlbeton	Berlin
	Deutscher Beton-Verein e.v.	Weisbaden
	Fachinformationszentrum-Workstoffe e.v.	Berlin
	Hochschule für Architektur und Bauwesen	Weimar
	IBR Institute für Bautechnologie	Ratingen
	Institute für Bauforschung (IBAC)	Äachen
	Institute für Baustoffe	Braunschweig
	Institute für Massivbau and Baustofftechnologie	Karlsruhe
	Institute für Nukleare Sicherheitsforschung der Kernforschungsanlange (KFA)	Jülich
	Kernforschungszentrum Karlsruhe (KfK)	Karlsruhe
	Lehrstuhl für Baustoffkunde und Werkstoffprufung der TU München	Munich
	Stangenberg, Schnellenbach and Partner GmbH	Bochum
	Technische Hochschule Darmstadt	Darmstadt
Finland	Technical Research Center of Finland	Espoo
France	Centre Experimental de Recherches et d' Etudes du Batiment et des Travaux Publics (CEBTP)	Saint Remy les Chevreuse
	Institut de Protection et Sureté Nucleaire (IPSN/DAS)	Fontenay-aux-Roses
	Electricité de France (EdF)	Villeurbanne
	Réunion Internationale des Laboratoires d' Essais et de Recherches sur les Matériaux et les Constructions	Cachan
Greece	National Technical University at Athens	Athens
India	Babaha Atomic Research Center	Trombay, Bombay
Israel	Israel Institute of Technology	Haifa

Table 2.2 (cont.) SAG program contacts with foreign organizations.

Country	Organization	Location
Italy	Comitato Nazionale per la Ricerca e per lo sviloppo del' Energia Nucleare e delle Energie Alternative (ENEA)	Rome
	Ente Nazionale per l' Energia Elettrica (ENEL)	Milan
	Industria Italiana del Cemeto	Rome
	Istituto Sperimentale Modelli e Strutture (ISMES)	Bergamo
Japan	Architecture Institute of Japan	Tokyo
	Building Research Institute-Ministry of Construction	Ibarku-ken
	Central Research Institute of Electric Power Industry (CRIEPI)	Toyko
	Japan Atomic Energy Research Institute (JAERI)	Ibarki-ken
	Japan Concrete Institute	Tokyo
	Japan Power Engineering and Inspection	Tokyo
	Kajima Institute of Construction Technology	Tokyo
	Komae Research Laboratory	Tokyo
	Nihon University	Chiba-ken
	Obayashi	Tokyo
	Research Institute of Shimizu Corp.	Kota-ku
	Shimizu Corp.	Tokyo
	Takenaka Komuten Technical Reasearch Laboratory	Tokyo
	Technical Research Institute - Obayshi	Tokyo
	Tokyo Institute of Technology	Kanagawa
	University of Kyoto	Kyoto
	University of Tokyo	Tokyo
Korea	Korea Electric Power Corp. (KEPCO)	Daejeon
	Korea Power Engineering Co. (KOPEC)	Seoul
Luxenbourg	Commission of European Communities	
Netherlands	Center for Civil Engineering Research and Codes	Gouda
	Conseil International du Batiment Pourla Research L'Etude et la Documentation (CIB)	Rotterdam
Portugal	Technica de Lisboa	Lisbon
Republic of China	Taiwan Power Co.	Taipei
Romania	Institutul de Percetári in Constructii	Bucharest
South Africa	Portland Cement Institute	Johannesburg

Table 2.2 (cont.) SAG program contacts with foreign organizations.

Country	Organization	Location
Sweden	Institute for Byggnadssatik	Stockholm
	Institute for Fire Safety Design (IDEON)	Lund
	Lund University	Lund
	National Swedish Institute of Building Research	Gävle
	Swedish Cement and Concrete Research Institute	Stockholm
	Swedish Council for Building Research	Solna
Switzerland	Basler & Hofmann	Zurich
	Comite Euro-International du Beton (CEB)	Lausanne
	Hauptabteilung fur die Sicherheit der Kernonlagen	Würenlingen
	International Association for Bridge and Structural Engineering	Zurich
Yugoslavia	Institute for Materials in Research and Structures (ZRMK)	Maribor

3. TASK S.2 – MATERIALS PROPERTY DATA BASE

3.1 OBJECTIVE

The objective of the materials property data base task is to develop a reference source that contains data and information on the time variation of material properties under the influence of pertinent environmental stressors and aging factors. The data base will have use in the prediction of potential long-term deterioration of critical structural components in nuclear power plants and in establishing limits on hostile environmental exposure for these structures. The results also will have application to the establishment of maintenance or remedial measures programs that will assist in either prolonging component service life or improving the probability of the component surviving an extreme event such as a loss-of-coolant accident. Figure 3.1 presents the Level 2 work breakdown structure for the materials property data base task.

3.2 BACKGROUND

One of the findings in NUREG/CR-4652 (Ref. 3.1) was that materials property data for concrete over an extended time period are limited. This is especially true for concretes that have been subjected to aging factors or environmental stressors characteristic of those that could occur in a nuclear power plant (see Ref. 3.1). Figure 3.2 obtained from Ref. 3.2, presents modes of degradation that were considered for the concrete components in a particular nuclear plant (Surry Unit One). Another limitation on materials property data availability is that in order for the data to be considered as being of "high quality," detailed information such as presented in Table 3.1 is desired. Unfortunately, for most structures that have been in service for the period of interest, 30 to 100 years, either detailed information about constituent materials, mix designs, plastic concrete properties, curing procedures, and exposure conditions are not available, or the time variation of material properties is unknown. Three potential means are available, however, that can be utilized to supplement the modest existing data base: (1) obtaining and testing samples from existing nuclear power plants, non-nuclear power generating facilities, and general civil engineering structures; (2) reviewing, analyzing, and trending existing data available through nuclear power plant prestressing tendon in-service inspection and integrated leak-rate test records; and (3) utilizing accelerated aging techniques.

3.3 PLAN OF ACTION

The Level 3 work breakdown structure for this task is presented in Fig. 3.3.

A vital component of the SAG Program is the development of the Structural Materials Information Center (SMIC). The SMIC consists of two formats so that information is useful to structural engineers, stress analysts, and materials engineers; for instance, the data base is being developed as an expandable, hard-copy handbook version (*Structural Materials Handbook*) and as an electronic version (*Structural Materials Electronic Data Base*) for use on an IBM or IBM-compatible personal computer. The handbook will serve as the information source upon which the electronic data base is built. Initially, information related to concrete, metallic reinforcements, prestressing steels, and structural steels are being incorporated into the data base. As the data base is developed, other structural materials will be added.

In parallel with the efforts to develop SMIC, numerous activities are being conducted relative to development of materials property data for input into SMIC. Potential sources of

information include cognizant foreign research establishments in Europe, North America, and Asia; domestic research establishments such as government facilities, universities, and consulting/engineering firms; and obtaining and testing of aged material samples from prototypical nuclear power plants or civil works structures. The open literature is being reviewed to identify representative (well documented) data for incorporation into SMIC, for example, technical reports, professional journal articles, and communications with fellow researchers.

Prediction or explanation of the complex interrelationships that occur between concrete's constituents and between concrete and its environment requires the development of mathematical models based on scientific and engineering principles. Such models play a vital role in the development of reliability-based life prediction techniques for concrete structures in nuclear power plants (Task S.4). Under this activity, modeling studies were conducted to complement those at the National Institute of Standards and Technology under another NRC program, "Performance Criteria for Concrete-Based Barriers for Low-Level Waste Disposal Facilities." Models developed under the SAG Program address aging factors and environmental stressors pertinent to safety-related concrete structures in nuclear power plants, as well as incorporate synergistic effects to predict behavior when more than one environmental stressor or aging factor is present.

3.4 SUMMARY OF ACCOMPLISHMENTS

During the reporting period covered by this document, activities have been conducted under each of the subtask areas shown in Fig. 3.1.

3.4.1 Structural Materials Information Center (Subtask S.2.1)

Utilizing results of a review and assessment of materials property data bases (Ref. 3.3), and a plan that had been prepared for development of the SMIC (Ref. 3.4), initial formatting of SMIC has been completed and the results presented in a report (Ref. 3.5). Updates of SMIC since initial formatting was completed are described in Refs. 3.6–3.9. A general description of the *Structural Materials Handbook* and the *Structural Materials Electronic Data Base* which forms the SMIC is provided below.

3.4.1.1 Structural Materials Handbook

The *Structural Materials Handbook* is being 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, for example, 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. 3.10) are presented in Figs. 3.4–3.6, respectively.

Initially, Vols. 1, 2, and 3 of the handbook will 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. 3.7. Table 3.2 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 is being collected and assembled for each material system included in the data base, for example, 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 property categories. Table 3.3 lists the property code ranges and corresponding material property categories. Reference 3.5 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 3.4. The 11 requirements utilized in the evaluation of the quality of data and values are listed in Table 3.5 with specific criteria for each of the quality levels provided in Ref. 3.5.

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.

3.4.1.2 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. 3.11) and EnPlot (Ref. 3.12). 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 word 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.

As noted previously (Refs. 3.5 and 3.6), Mat.DB was designed primarily for use with metallic materials. This has required adaptation 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 3.6 presents a list of data base management system requirements considered necessary for storing and accessing materials property data and information at the Structural Materials Information Center. 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. 3.13) is being developed by ASM International that contains features making the software easier to use. Suggestions for enhancing the software have been made by SAG Program personnel during meetings with ASM International representatives. The extent to which this revision of Mat.DB will address these suggestions cannot be determined at this time, but it is not anticipated that Mat.DB will be revised to reflect all comments.

Based on experience gained during development of the Structural Materials Information Center, advances in personal computer hardware capabilities, and corresponding developments in software tools for building customized data bases, a reassessment of candidate systems was conducted. 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, which 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 3.6 and also permits entry of existing data and information files. Object-oriented relational data base software, such as Microsoft Access (Ref. 3.14), would provide the foundation for a new data base management system, and the data base could be completely designed and built locally. More details on the overall assessment are provided in Ref. 3.15.

3.4.2 Data Collection (Subtask S.2.2)

One of the findings of Ref. 3.1 was that documented long-term and environment-dependent materials property data for concrete are limited. This is especially true for concretes that had been subjected to aging factors or environmental stressors characteristic of those that could occur in a nuclear power plant. Another limitation on material property data is that in order for the data to be considered "high quality," detailed information such as presented in Table 3.1 is desired. Unfortunately, for most concrete structures that have been in service for the period of interest (30 to 100 years), either detailed baseline information or the time variation of the properties of the construction materials is not available. Two approaches have been utilized in an attempt to expand the somewhat limited data base: (1) pursuing technology exchange with U.S. and foreign research establishments, and (2) prototypical sample evaluations.

3.4.2.1 Technology Exchange

Domestic and foreign research organizations have been contacted to obtain long-term and environment-dependent materials property data suitable for entry into the Structural Materials Information Center. In addition, efforts were made to obtain background information on the in-service inspection programs, nondestructive examination and repair procedures, and service life prediction methodologies for existing concrete structures. Table 3.7 presents a summary listing of organizations from which information and reports of interest have been acquired. Examples of some of the types of data that have been obtained from reference documents provided through domestic and foreign contacts are provided in Fig. 3.8.

3.4.2.2 Prototypical Sample Evaluations

Prototypical concretes and prestressing tendon systems have been tested and/or evaluated to obtain data and information on their performance as a function of time under representative nuclear power station environmental conditions.

Concrete Materials

In parallel with the efforts to obtain concrete properties data, several U.S. utilities, national laboratories, and concrete research organizations have been contacted to pursue the possibilities of obtaining concrete core samples from prototypical nuclear power plant structures for testing. A summary of these contacts and the status of the discussions is presented in Table 3.8. These contacts have resulted in procurement of samples from

Shippingport Power Station (Battelle Pacific Northwest Laboratories); EBR-II site (Argonne National Laboratory-West); Palisades and Midland Power Stations (Consumers Power Co.); Braidwood, Byron, Dresden, LaSalle, Quad Cities, and Zion Power Stations (Commonwealth Edison Co.); and Vallecitos Nuclear Center (General Electric Test Reactor). Testing of samples obtained from each of these plants has been completed. Figure 3.9 presents relative compressive strength test results obtained from samples for which baseline data were available. Relative to the 28-day reference values, the concrete compressive strength results from these samples at ages of about 25 years have shown increases of up to approximately 100% .

Under the subcontract with Taywood Engineering Ltd. (London, England), test results were provided from archived test specimens that had been cast in conjunction with the fabrication of several of the United Kingdom nuclear power stations. Over 100 test specimens, generally 450-mm long by 150-mm diameter and having ages from 4 to 24 years, were available from the Wylfa, Heysham I, Heysham II, Hartlepool, Torness, and Sizewell"B" stations. A small number of specimens, 300-mm long by 100-mm diameter, were also available from the Wylfa station. With the exception of the Wylfa station specimens, all the specimens had been initially cured under heat cycled conditions to simulate the early in-situ temperature rise due to cement hydration. Each specimen had been fitted with embedded measurement studs for strain measurements (200-mm gage length) using a Demec gage. Most of the specimens also contained vibrating wire strain gages (200-mm gage length). The specimens had been continuously stored in a sealed, stable moisture state at temperatures from 10° to 95°C with some having been under sustained loading (13.8 MPa). Available baseline data for the six concretes included compressive strength (up to 1 year), thermal expansion, thermal conductivity, and details on constituent materials and mixture proportions. Also, a limited number of cylinders for the Heysham I and Heysham II concretes had been tested for elastic and creep recovery, and for compressive strength at ages to 4 years. Elastic modulus and creep results are available for the Wylfa concrete up to an age of 12.5 years. Twenty-nine specimens of the over 100 available were selected for testing to provide information on the long-term performance of nuclear grade concretes. Variables investigated included age of specimen, concrete mix design, loaded or unloaded while curing, and storage temperature. The test procedure followed consisted of six steps: (1) measurement of the static modulus of elasticity up to loads of one-third the estimated compressive strength; (2) cutting the specimen into three sections and placing the center section containing the Demec gage points and vibrating wire strain gage in storage; (3) sealing the two remaining end sections with moisture-resistant plastic and storing until time of test; (4) unsealing each of the specimen end sections and subjecting them to a series of nondestructive tests (density, ultrasonic pulse velocity, Schmidt hammer, surface hardness, and dynamic modulus of elasticity); (5) resealing one of the two end sections and returning it to its original storage temperature for future monitoring and testing; and (6) saturating the remaining end section in water, testing it for saturated density, capping the section ends, and testing to determine its compressive strength. Results indicate that, except for one or two exceptions, there was a consistent trend for the concrete moduli of elasticity and compressive strengths to increase with age for each of the mixes. From the limited results that were available for specimens that were continuously stored unloaded at 10° to 20°C, increases in modulus of elasticity relative to 28-day reference values ranged from 3% for the Sizewell "B" concrete (test age of 4.8 years) to 112% for the Wylfa concrete (test age of 21 years). Increases in compressive strength values obtained from cube test specimens stored unloaded at temperatures from 10 to 20°C ranged from about 12% for the Torness concrete (test age of 11.7 years) to about 60% for the Heysham I concrete (test age of 23.6 years). Figure 3.10 presents a summary of relative compressive strength versus age results for these specimens. Details on specimens tested, test procedures and test results are presented in Ref. 3.16.

Prestressing Systems

Taywood Engineering Ltd., as part of their overall subcontract in support of the SAG Program, collated surveillance data for several prestressed concrete pressure vessels (PCPVs) in the United Kingdom. Prestressing tendon anchorage lift-off load measurement data, results of corrosion examinations, and visual examination results were compiled from surveillance records for the PCPVs at the Wylfa, Hartlepool, and Heysham I stations. Two PCPVs are located at each station.

The PCPVs at Wylfa Nuclear Power Station are concrete spheres having a 30-m internal diameter with a 4-m wall thickness. Each vessel is prestressed by over 1200 tendons, with each tendon having an ultimate tensile capacity of 820 tons. The average tendon is about 37 m long and consists of 36 seven-wire strands each having a diameter of 15.2 mm and guaranteed ultimate tensile strength (GUTS) of 227 kN. Corrosion protection of the ungrouted tendons is provided by a combination of greases and waxes.

At both Hartlepool and Heysham I Nuclear Power Stations, the PCPVs are essentially upright cylinders 29.3-m high by 25.9-m diameter, supported by concentric rings. The reactor is housed in an inner cavity 13.1-m diameter by 18.3-m high. The PCPVs are prestressed by a system of hoop and vertical tendons. The twenty hoop tendons are formed by winding wire laid under tension in steel-lined channels formed on the vessel outer surface. Each channel contains up to 35 layers of 5.08-mm diameter wire laid into the channel at a tension of 70% GUTS to provide a hoop tendon having a GUTS of 13,630 tons. The 272 vertical tendons, each having a capacity of 1040 tons GUTS, consist of 28 seven-wire drawn strands of 18-mm diameter. Corrosion protection of the ungrouted tendons is provided by a combination of greases and waxes.

Surveillance data results presented for the Wylfa, Hartlepool, and Heysham I Nuclear Power Stations cover time periods since prestressing of 23, 14.5, and 13.4 years, respectively. As expected, the tendon anchorage lift-off load measurements showed a general 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 rate of growth 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. More detailed surveillance results for each of the three power stations are contained in Ref. 3.17.

3.4.2.3 Materials Property Data Bases Completed for SMIC

As summarized in Table 3.9, the SMIC currently contains 144 material property data bases. Material codes, common names, and material descriptions for each of the data bases are provided in Tables 3.10–3.14 for the portland cement concrete, metallic reinforcement, prestressing tendon, structural steel, and rubber material systems, respectively. A summary description of each of the data base sources is presented below. More detailed information on the data base and the materials that it contains is provided in Ref. 3.18.

Portland Cement Concretes

Organizations and research establishments providing information or samples from which concrete material property data bases have been prepared and included in Chapter 1 (Vols. 1–3) of the handbook and electronic data base file CONCRETE.DB are listed in

Table 3.15. A summary of reference parameters (e.g., cement type and content, coarse aggregate type and maximum size, plastic properties, etc.) and baseline properties (e.g., compressive strength at age of either 28 or 31 days) reported for these materials is provided in Table 3.16. A summary of the environment- and time-dependent concrete properties that are available in SMIC is provided in Table 3.17. Applicable material codes for each of the information sources summarized below are identified with the source as well as in Table 3.10.

University of Wisconsin long-term study. Three experimental studies, Series A, B, and C, have been conducted at the University of Wisconsin on the long-term behavior of concrete. These studies began in 1910 and focused on compressive strength, unit weight, and volume changes of concrete materials exposed to the climate in Madison, Wisconsin. Results from Series B, which began in 1923, have been entered into the Structural Materials Information Center.

In Series B, test specimens were prepared from six concrete mixes. Each mix consisted of four batches of concrete. Each batch contained a portland cement that was produced by one of four different vendors and one type of coarse aggregate. Three types of 38-mm maximum size coarse aggregate were used in the mixes: a crushed dolomite, a crushed red granite, and a gravel. Cement contents in the mixes ranged from 200 to 369 kg/m^3 and water-to-cement ratios from 0.41 to 0.69. No chemical or mineral admixtures were used in the mixes. The test specimens were cast, cured for 28 days, stored either inside (controlled environment) or outside, and tested at selected intervals over a 50-year period. Each of the six data bases include average compressive strength values as a function of time. Average reference 28-day compressive strengths ranged from 10 to 19.2 MPa. Since tests were performed at selected times over a 50-year period, the compressive strength versus time performance curves are considered high quality and have been rated Quality Level A. Material codes related to this study include 01CA001, 01CA002, and 01CB002 – 01CB005.

Portland Cement Association long-term and elevated temperature studies. Under a subcontract with Construction Technology Laboratories (CTL), Inc., in Skokie, Illinois, results have been assembled and provided for three of four series of tests from a study that has been ongoing at the Portland Cement Association since the early 1940s to investigate the strength-producing properties of several cements. The overall program encompassed about 500 concrete mixes fabricated using a wide variety of cement and aggregate materials. Beam and cylindrical test specimens were either moist cured (continuous storage in a moist room at 100% relative humidity and 23°C), air cured (moist cured for 7 days followed by laboratory storage at ~50% relative humidity and 21° to 23°C), air cured followed by soaking (moist cure for 7 days, air cured till 48 hours before testing, soaking in water at 23°C for 48 hours), cured outdoors at Skokie, Illinois (moist cure for 7 days, store outdoors on a clay loam), or cured outdoors at Dallas, Texas (moist cure for 7 days, store outdoors on a sandy soil) until testing. Periodically, at ages from 1 day to 34 years, specimens were tested to determine values of compressive strength, modulus of elasticity, and modulus of rupture as a function of materials, environment, and time. Reference 3.19 presents summary descriptions of the overall program, material characteristics, mixture proportions, specimen geometries, curing conditions, test methods, and available data. Detailed results on the material characteristics, mix designs, curing conditions, and test results for the three test series are provided as appendices to the report. Each of the three appendices to the report corresponds to a particular test series and is divided into two volumes (Vol. 2 – Supporting Documentation and Vol. 3 – Material Data Sheets) in accordance with formatting requirements for the ORNL Structural Materials Information Center (Ref. 3.5). Of the over 500 concrete mixes prepared, 51 have been selected to provide information on a wide variety of cement types and curing conditions and material property (flexure strength, compressive strength and dynamic modulus of elasticity) versus

time performance curves were developed and entered into the Structural Materials Information Center. Material codes related to this study include 01CA010 – 01CA015, 01CB036 – 01CB065, and 01CB127 – 01CB134.

A follow-on study was conducted at Construction Technology Laboratories (CTL), Inc. to provide additional data for one of the four test series that had been cast to investigate the strength-producing properties of several cements. Twenty-two archived beam test specimens that had been continuously moist cured (100% relative humidity and 23°C) since being cast in 1950 were tested. Each 152.4- by 152.4- by 762-mm specimen was first tested in flexure using a 457.2-mm end of the beam to obtain a modulus of rupture value. Pulse velocity and unit weight were then measured on the short end of the beam. Cubes were cut from the two ends of the beam for compression tests. Also, a 25.4-mm slice of the beam was cut for petrographic examination and two nominal 101.6-mm diameter cores were extracted from the remainder of the beam for compression and unit weight tests. These tests increased the period for which the performance curve versus time results presented in Ref. 3.19 for this test series were available from 34 to 42 years. Average compressive strength results obtained from these specimens at the test age of 42 years ranged from 30.9 to 78.9 MPa. Relative to the 28-day reference compressive strength values, increases in compressive strength for the mixes ranged from 24% (253 kg/m^3 Type III cement; w/c of 0.6, by weight) to 265% (168 kg/m^3 Type IV cement; w/c of 0.78, by weight). Additional details on the mix designs, testing procedures and test results are presented in Ref. 3.20. Since tests were performed at selected times throughout the duration of the study, baseline data on constituent materials and mix designs are available, and exposure conditions are well documented, the material property versus time performance curves are considered high quality and have been rated Quality Level A. Material codes related to this study include 01CB068, 01CB069, 01CB093, 01CB0694, 01CB097 – 01CB100, 01CB110, 01CB111, and 01CB124 – 01CB126.

An investigation was conducted to examine the effects of long-term, elevated-temperature exposure on the mechanical and physical properties of two normal-weight concretes having characteristics similar to those used to construct the underground storage tanks at Hanford, Washington. Both concretes were prepared using ASTM C 150, Type II portland cement and local basalt aggregate materials. The first mix contained 298 kg/m^3 of cement, a coarse aggregate-to-cement ratio of 4.24 (by weight), a water-to-cement ratio of 0.49 (by weight), and an air-entraining admixture. The second mix contained 391 kg/m^3 of cement, a coarse aggregate-to-cement ratio of 2.87 (by weight), a water-to-cement ratio of 0.41 (by weight), and an air-entraining admixture. Cylindrical test specimens 152-mm diameter by 304-mm long were cast from each mix. After casting, some of the specimens were stored continuously in a moist room maintained at 21°C and 100% relative humidity until they were tested. Others were stored in the moist room until heat-up to either 121°, 177°, or 232°C was initiated. The reference 30-day compressive strengths for the two mixes were 34.2 MPa and 40.7 MPa, respectively. At designated times over a five-year period, specimens from each of the two mixes were tested to determine compressive strength, modulus of elasticity, Poisson's ratio, splitting-tensile strength, and thermal properties (thermal coefficient of expansion, thermal conductivity, thermal diffusivity, and specific heat). Since tests were performed at selected times throughout the duration of the study, baseline data on constituent materials and mix designs were available, and exposure conditions were well documented, the material property versus time performance curves are considered high quality and have been rated Quality Level A. Material codes related to this study include 01CB156 and 01CB157.

Elevated temperature tests were conducted as part of an effort to develop rational methods for calculating the behavior of concrete structures subjected to fire or other high-temperature conditions. Air-entrained, normal-weight and lightweight concretes were prepared using either carbonate, siliceous, or expanded-shale coarse aggregate materials having a 19-mm maximum size. The 2310 kg/m^3 carbonate aggregate mix used

236 kg/m^3 of ASTM C 150, Type I portland cement, a coarse aggregate-to-cement ratio of 4.59 (by weight), and a water-to-cement ratio of 0.55 (by weight). The 2301 kg/m^3 siliceous aggregate mix used 249 kg/m^3 of ASTM C 150, Type I portland cement, a coarse aggregate-to-cement ratio of 4.31 (by weight), and a water-to-cement ratio of 0.51 (by weight). The 1720 kg/m^3 expanded shale aggregate mix used 263 kg/m^3 of ASTM C 150, Type I portland cement, a coarse aggregate-to-cement ratio of 2.80 (by weight), and a water-to-cement ratio of 0.78 (by weight). Cylindrical test specimens 76-mm diameter by 152-mm long were cast from each mix. Twenty-four hours after casting the specimens were placed into a fog room where they remained for 6 days prior to placing into a room maintained at 21° to 24°C and 30 to 40% relative humidity. After about three to six weeks of additional curing, when the relative humidity at the center of the humidity-monitoring specimen reached 75%, elevated-temperature testing initiated. Sufficient specimens had been cast from each mix to investigate 14 different temperatures in the range of 21° to 871°C. Three test procedures were utilized: (1) specimens were heated to the test temperature while unloaded, and then loaded in compression to failure; (2) specimens were heated to the test temperature while unloaded, allowed to cool slowly to room temperature, stored for seven days in a room maintained at 21° to 24°C and 70 to 80% relative humidity, and then loaded in compression to failure; and (3) specimens were loaded to either 25, 40 or 55% of baseline compressive strength, heated to the test temperature while loaded, and then loaded in compression to failure while heated. After reaching the designated test temperature, each specimen was allowed to stabilize at temperature for three to four hours prior to continuing with the test procedure. Baseline compressive strength for all three mixes was 26.9 MPa. Since tabulated data on baseline and elevated-temperature compressive strengths were not presented, the data have been rated Quality Level B. Material codes related to this study include 01CB161, 01CB162 and 01BC001.

University of Saskatchewan (Canada) elevated-temperature study. The effect of elevated temperature on the modulus of elasticity and compressive strength of concrete was investigated in a 1981 study at the University of Saskatchewan. The concrete specimens (235-mm long by 75-mm diameter) were made using ASTM C 150, Type V sulfate-resistant portland cement, ASTM C 618, Type C fly ash from lignite coal, and 19-mm maximum size crushed stone coarse aggregate. Also included in the mix were an air-entraining agent, a conventional water-reducing admixture, and a high-range water-reducing admixture (superplasticizer). The average 28-day compressive strength and modulus of elasticity values for the concrete mix were 24.8 MPa and 24.5 GPa, respectively. After moist curing for 28 days, the specimens (sealed and unsealed) were placed into environmental chambers and heated or cooled to their designated soak temperatures. Sets of specimens were subjected to one of seven temperature levels ranging from -11° to 232°C. At the end of each of ten exposure periods, which ranged from 1 day to six months, a minimum of three sealed and three unsealed specimens were removed from the environmental chambers, permitted to thermally stabilize at room temperature, and then tested in compression. Since only average compressive strength results are presented, the data have been rated Quality Level B. The material code related to this study is 01CA003.

A second study at the University of Saskatchewan investigated the influence of temperature on normal-weight, high early strength concrete. The concrete was prepared using ASTM C 150, Type III cement, and 19-mm maximum size aggregate materials consisting of dolomite and hornblende crushed stone. The water-to-cement ratio was 0.60 and no chemical or mineral admixtures were used. Cylindrical test specimens 76-mm diameter by 235-mm long were cast, moist cured for 24 hours, demolded, sealed, and then divided into two specimen sets. The first set of specimens was immediately subjected to one of eight test temperatures from 2° to 232°C. The second set was cured at room

temperature for 14 days prior to elevated temperature exposure. At designated times over a six-month period (4, 14, 91, or 181 days), specimens were removed from their elevated temperature environment, permitted to slowly cool to room temperature, and then tested to determine their compressive strength and modulus of elasticity. Reference 28-day compressive strength and modulus of elasticity values for the concrete were 37.9 MPa and 35.9 GPa, respectively. Since only average compressive strength results are presented, the data have been rated Quality Level B. The material code related to this study is 01CA018.

A third study at the University of Saskatchewan also investigated the influence of temperature on normal-weight concrete. The concrete specimens (235-mm long by 75-mm diameter) were made using ASTM C 150, Type I portland cement, ASTM C 618, Type C fly ash from lignite coal, and 19-mm maximum size crushed stone coarse aggregate. Also included in the mix were an air-entraining agent and a conventional water-reducing admixture. After moist curing for 24 hours, each specimen was sealed and stored at room temperature for 28 days additional curing prior to testing. Specimens were tested by exposing them to one of six temperatures ranging from 21° to 232°C. At designated times over a six-month period (3, 7, 14, 28, 56, 91, or 181 days), three specimens having the same exposure condition were removed from the oven, allowed to return to room temperature, and then tested to determine compresssive strength, modulus of elasticity, and stress-strain response. Reference 28-day compressive strength and modulus of elasticity values for the concrete were 21.6 MPa and 32.1 GPa, respectively. Since only average compressive strength results are presented, the data have been rated Quality Level B. The material code related to this study is 01CA019.

CANMET (Ottawa) long-term elevated temperature study. Six normal-weight concretes were prepared as part of a Canada Centre for Mineral and Energy Technology research program to develop long-term strength data on concretes exposed to sustained elevated temperature. The first phase of the study was initiated in 1979 for the purpose of examining normal portland cement, normal portland cement plus slag, and normal portland cement plus fly ash concretes.* Either limestone or dolostone were used as the coarse aggregate materials in the mixes that had water-to-cement ratios ranging from 0.33 to 0.60. Cylindrical test specimens (203-mm long by 102-mm diameter) were cast from each mix, moist cured for 28 days, and then permitted to air dry in the laboratory for 26 weeks prior to elevated temperature exposure. Each specimen was slowly heated to a prescribed thermal soak temperature that covered a temperature range from 75° to 600°C, maintained at the soak temperature for up to four months, and then permitted to cool to room temperature. Tests conducted at room temperature following thermal exposure included pulse velocity, resonant frequency, and compressive strength. Average reference 28-day compressive strengths obtained from moist-cured specimens ranged from 32.9 to 57.7 MPa for the concrete mixes. Since only average test results are presented, results have been rated Quality Level B. The material codes related to this study are 01CA004 – 01CA009.

EBR II Site at Argonne National Laboratory-West. The fuel cycle facility for the Experimental Breeder Reactor (EBR-II) was constructed by the Argonne National Laboratory in 1959 and 1960 near Idaho Falls, Idaho using normal-weight and heavyweight concretes. The normal-weight concrete was fabricated using ASTM C 150, Type I portland cement having a cement content of 301 kg/m^3, 38-mm maximum size gravel aggregate, and a water-to-cement ratio of 0.42. Air-entraining admixture was used in the mix. The heavyweight concrete mix was fabricated using a ASTM C 150, Type I portland cement content of 301 kg/m^3, 38-mm maximum size crushed stone coarse aggregate, and a water-to-cement ratio of 0.40. A water-reducing admixture and 16-mm diameter steel shot were added to the mix to increase its placeability and shielding effectiveness, respectively. During construction, plastic concrete properties were obtained

* Only the test results for the normal portland cement concretes are presented in the data base.

and cylindrical concrete test samples were cast from each of the mixes. These samples were moist cured and tested at ages of 7 and 28 (or 31) days to verify that the material met its design compressive strength requirements. Average reference compressive strengths for the normal-weight and heavyweight concrete mixes were 37.7 and 38.4 MPa, respectively. In 1989 normal-weight concrete core samples were removed from the argon and air cell column footings, and heavyweight concrete core samples were removed from the argon cell shielding walls. Compressive strength, modulus of elasticity, and Poisson's ratio values were determined from each of the samples. Each data base includes a compressive strength versus time performance curve, but because only limited (7-day, 28-day and approximately 30-year) test results were available, each performance curve was rated Quality Level C. The material codes related to this study are 01CB001 and 01DA001.

In 1990 normal-weight concrete core samples were removed from the subbasement walls and the biological shield inside the reactor plant building at the EBR-II site. The subbasement concrete was fabricated using ASTM C 150, Type I portland cement having a cement content of 288 kg/m^3, 38-mm maximum size gravel aggregate, and a water-to-cement ratio of 0.57. Air-entraining admixture was used in the mix. The biological shield concrete was fabricated using ASTM C 150, Type I portland cement having a cement content of 307 kg/m^3, 38-mm maximum size gravel aggregate, and a water-to-cement ratio of 0.51. During construction, plastic concrete properties were obtained and cylindrical concrete test samples were cast from each of the mixes. These samples were moist cured and tested at ages of 7 and 28 days to verify that the material met its design compressive strength requirements. Average reference compressive strengths for the subbasement and biological shield concrete mixes were 35.1 and 33.6 MPa, respectively. Each data base includes a compressive strength versus time performance curve, but because only limited (7-day, 28-day and approximately 31-year) test results were available, each performance curve was rated Quality Level C. The material codes related to this study are 01CB006 and 01CB007.

Walz (Germany) long-term study. Concrete cube and slab sections were cast in 1941 by the Research and Materials Test Center for Construction at the Technical University in Stuttgart as part of a long-term study to investigate the effects of weathering on concrete and stone materials. The eight concrete materials used to cast the 200-mm cube and 150- by 400- by 700-mm slab test specimens were fabricated using cement that essentially met ASTM C 150 requirements for Type I portland cement, gravel aggregates, and water. No chemical or mineral admixtures were included in the mixes. Two nominal cement contents were used in the mixes, 200 kg/m^3 and 300 kg/m^3. Water-to-cement ratios for the 200 kg/m^3 concrete mixes ranged from 0.78 to 1.29, and from 0.50 to 0.82 for the 300 kg/m^3 cement-content concretes. Six cube and 20 slab test specimens were cast from each mix. Three of the cubes were moist cured for 28 days and then tested in compression. Reference 28-day compressive strength values for the 200 kg/m^3 concrete mixes ranged from 5.4 to 18.6 MPa, and from 15.5 to 36.5 MPa for the 300 kg/m^3 cement-content concretes. The other three cubes were continuously moist cured for 14 days and then alternately wetted and dried in air until tested in compression at an age of 2.5 years. Two of the slab sections from each mix were exposed to 100 rapid freezing-thawing cycles in the laboratory prior to testing in flexure. Six of the slab sections from each mix were stored outdoors near Stuttgart, and six each were half submerged in slow moving rivers near Blaubeuren and Partenen. After 2-3/4 and 12 years of exposure, two of the slab sections stored near Stuttgart were tested in flexure. The remaining two slab sections remained near Stuttgart until an age of about 30 years at which time 100-mm-diameter cores were removed and tested to assess depth of carbonation and compressive strength. After 12 years of exposure, three each of the slab sections at Blaubeuren and Partenen were tested in flexure. Two additional slab sections for each mix were removed from these sites at an age of about 29 years and 100-mm-diameter cores taken and tested to assess depth of

carbonation and compressive strength. Due to the limited number and differences in size of the test specimens, the time-dependent compressive strength properties for these materials have been rated Quality Level D. The material codes related to this study are 01CB008 – 01CB015.

Kondo (Japan) stray electrical current corrosion study. Nine normal-weight concrete mixes were prepared at Kyoto University as a part of a research program to investigate the electrolytic corrosion of reinforcing steel in concrete resulting from stray electrical currents. The test specimens were cast from concrete mixes that contained various combinations of fly ash (0 to 30%, by total weight cementitious materials), calcium chloride (0 to 1.0%, by weight cement), and calcium lignosulfate (0 to 0.25%, by weight cement). No other chemical or mineral admixtures were used in the mixes. Total cementitious materials contents for all the mixes was 320 kg/m^3 and the water-to-cement ratios ranged from 0.49 to 0.58. A 25-mm maximum size river gravel was used as the coarse aggregate material in all the mixes. Twelve bond test specimens (150-mm cubes) were typically cast from each concrete mix. Embedded in each concrete cube were a 0.423-mm thick galvanized iron plate and a No. 6 deformed steel reinforcing bar. After casting and demolding, specimens were submerged in 21°C water for five days. Upon removal from the water, each specimen was immediately sealed with a membrane compound to prevent moisture transfer. After nine additional days of curing in a laboratory environment, nine specimens from each batch were connected to direct current power supplies with the reinforcing bars serving as anodes and the galvanized plates as cathodes. Three each of the specimens were connected to 5-volt, 10-volt, and 20-volt power supplies. Current was applied to each specimen for 3 hours daily for 14 days. Three of the specimens were not subjected to direct current and served as controls. Two of the concrete mixes cast additional specimens to investigate the effect of 20-volt alternating current supply and of switching the anode and cathode locations. At conclusion of the 14-day exposure period, the specimens were tested in accordance with ASTM C 234-54 requirements to evaluate the effect of electrical current on the concrete-steel reinforcement bond. Since the compressive strength at the time of testing was not reported, the bond stress versus slip performance curves have been rated Quality Level B. The material codes related to this study are 01CB016 – 01CB024.

Ontario Hydro long-term studies. Concrete test cylinders (305-mm long by 152-mm diameter) were cast by Ontario Hydro in conjunction with fabrication of the Sir Adam Beck Niagara Generating Station (SABNGS) No. 2 in 1953. This study was part of Ontario Hydro's dam safety program in which a wide variety of concretes were prepared, ranging from very lean, without admixtures, to air entrained with high cement content. The objective of the program is to investigate the long-term performance of these materials. Sufficient control specimens were cast at each site to periodically obtain concrete compressive strength data at ages from 7 days to 100 years. The SABNGS concrete was made using ASTM C 150, Type I portland cement, fly ash, natural sand, and 38-mm maximum size coarse aggregates. A total of nine different mixes were used to fabricate the generating station, three mixes each having cementitious materials contents of either 281, 311, or 363 kg/m^3. The first set of three mixes contained no chemical or mineral admixtures, the second set contained an air-entraining admixture, and the third set contained no air-entraining admixture, but fly ash provided 30% of the total weight of cementitious materials. After fabrication, each specimen was cured in a moist room located at the site until the field laboratory was dismantled. The remaining specimens were then either temporarily buried at the site or placed into tunnels at the facility. Later, all remaining samples were sent to the Ontario Hydro Research Center (Toronto) where they were buried in the ground to a minimum depth of one meter to expose the specimens to moist curing without the threat of freezing and thawing. During the summer prior to testing, specimens were removed from the ground and stored in a laboratory moist room until scheduled for testing. Reference 28-day compressive strengths for the mixes ranged from 25.0 to 39.4 MPa. To date, compressive strength tests have been conducted at 7, 28, and 90

days, and 1, 2, 3, 5, 10, 20, and 25 years. Sufficient specimens remain for testing at ages of 50 and 100 years. Since tests were conducted at selected times over a 25-year period and all baseline characteristics for the materials were reported, the compressive strength versus time performance curves are considered high quality and rated Quality Level A. The material codes related to this study are 01CB025 – 01CB033.

Ontario Hydro also cast concrete cylinders (305-mm long by 152-mm diameter) in conjunction with fabrication of the Stewartville Generating Station in 1948. This study was also part of Ontario Hydro's dam safety program and, thus, utilized the same specimen preparation, curing, and testing procedures as the SABNGS study. The Stewartville concrete was made using ASTM C 150, Type I portland cement, natural sand, and 38-mm maximum size coarse aggregate. No mineral or chemical admixtures were added to the mixes. Two main mixes were used in construction of the generating station, a face concrete and a core concrete. The face concrete mix had a water-to-cement ratio of 0.70 and a cement content of 274 kg/m^3. The core concrete had a water-to-cement ratio of 0.98 and a cement content of 334 kg/m^3. Reference 28-day compressive strengths for the two mixes were 25.1 and 14.1 MPa, respectively. To date, compressive strength tests have been conducted at 28 and 90 days, and 1, 2, 3, 5, 10, 15, and 25 years. Sufficient specimens remain for testing at ages of 50 and 100 years. Since tests were conducted at selected times over a 25-year period and all baseline characteristics for the materials were reported, the compressive strength versus time performance curves are considered high quality and rated Quality Level A. The material codes related to this study are 01CB034 and 01CB035.

Background data and test results also were provided by Ontario Hydro for a high-density shielding wall concrete. The 2.13-m thick wall was constructed as part of a Canadian nuclear power facility using a concrete mix containing hematite, ilmenite, and feldspar aggregate materials (20-mm maximum size) to produce a mix having a unit weight over 3400 kg/m^3. An ASTM C 150, Type IV, low heat of hydration cement was used in a quantity of 348 kg/m^3. The mix used a water-reducing admixture and had a 0.49 water-to-cement ratio. During construction a number of 300-mm long by 152-mm diameter cylindrical test specimens were cast, moist cured, and tested at ages of 7 and 28 days. At an age of 20 years, two 150-mm diameter cores 1.8-m long were taken from the 2.13-m thick wall. The cores were used to determine compressive strength, modulus of elasticity, specific heat, thermal diffusivity, thermal conductivity, and coefficient of thermal expansion values for the shielding concrete. Due to the limited number and ages at which specimens were tested, the time-dependent compressive strength properties for these materials have been rated Quality Level C. The material code related to this study is 01DA002.

General Electric test reactor at Vallecitos Nuclear Center. Six 152-mm diameter by 750-mm long concrete core specimens were provided by the Vallecitos Nuclear Center for testing. The cores were removed from different sections of the biological shield of the 50 -MW, pressurized, light-water cooled, high-flux reactor that began full-power operation in 1959. Three cores were normal-weight concrete and three cores were heavyweight concrete. The normal-weight concrete had been prepared using 323 kg/m^3 of ASTM C 150, Type I portland cement, natural sand, 32-mm maximum size gravel coarse aggregate, and a water-to-cement ratio of 0.54. The heavyweight concrete had been prepared using 307 kg/m^3 of ASTM C 150, Type I portland cement, magnetite sand, 38-mm maximum size crushed magnetite coarse aggregate, a water-to-cement ratio of 0.55, and a water-reducing chemical admixture. The compressive strengths at 28-days age for the normal-weight and heavyweight concretes were 26.2 and 20.7 MPa, respectively. Compressive strength test results were also available for both mixes at concrete ages of 7 days, 90 days, 20 years, and 34 years. Since constituent material information, mix designs and plastic concrete properties are available as well as strength results at several concrete ages, the compressive strength versus time performance curves have been rated Quality Level B. The material codes related to this study are 01CB151 and 01DA003.

Midland Nuclear Power Station. Two normal-weight concrete cores from a floor slab located inside the auxiliary building and seven normal-weight concrete cores from two floor slabs located inside the Unit 2 containment building were provided by Consumers Power for testing. The concrete from the auxiliary building floor slab had been prepared using 271 kg/m^3 ASTM C 150, Type II portland cement, 48 kg/m^3 ASTM C 618, Type F fly ash, 19-mm maximum size crushed stone aggregate, a water-to-cementitious materials ratio of 0.47, and air-entraining and water-reducing admixtures. The concrete from the containment building floor slabs had been prepared using 403 kg/m^3 ASTM C 150, Type II portland cement, 71 kg/m^3 ASTM C 618, Type F flyash, 38-mm maximum size crushed stone aggregate, a water-to-cementitious materials ratio of 0.34, and air-entraining and water-reducing admixtures. The auxiliary and containment building concretes had 28-day compressive strengths of 34.3 and 41.4 MPa, respectively. Compressive strength test results are also available for both mixes at concrete ages of 7 days, 90 days, and 14 years. Although constituent material information, mix designs, and plastic concrete properties are available for these materials, no compressive strength results are available for concrete ages between 90 days and 14 years. The compressive strength versus time performance curves for these materials, therefore, have been rated Quality Level C. The material codes related to this study are 01CA016 and 01CA017.

United Kingdom Nuclear Power Stations. A multi-year testing program was conducted by Taywood Engineering Ltd. to investigate the short-term properties and long-term integrity of the concrete containment structures at the Wylfa, Heysham I, Heysham II, Hartlepool, Torness, and Sizewell "B" Nuclear Power Stations. The normal-weight, non-air-entrained concretes used at these stations were made using cement that essentially met the requirements of ASTM C 150, Type I portland cement, either crushed stone or gravel aggregates, and a water-reducing chemical admixture. Fly ash was included in the concretes used at Heysham II, Torness, and Sizewell "B" stations. The cementitious material contents ranged from 355 to 455 kg/m^3, and the water-to-cementitious materials ratios ranged from 0.39 to 0.49. Long-term concrete properties (ages ranging from 4 to 24 years) were determined from specimens that were cast in conjunction with each of these stations. Prior to testing, the specimens had been continuously stored in a sealed, stable moisture state at temperatures from 10° to 95°C, with some having been under sustained loading (13.8 MPa). Section 3.4.2.2, "Prototypical Sample Evaluations," provides additional information on this test program. The results of this study have been rated Quality Level C since only average compressive strength results are presented, and the time between tests is significant. The material codes related to this study are 01CA023 – 01CA027, and 01CB167.

University of California elevated-temperature study. Eighteen cylindrical test specimens 152-mm diameter by 305-mm long were tested to investigate the influence of thermal exposure on the mechanical properties of a limestone aggregate concrete. The specimens were cast from a concrete mix prepared using 404 kg/m^3 ASTM C 150, Type II portland cement, 19-mm maximum size aggregate material having a coarse aggregate-to-cement ratio of 2.62 (by weight), and a water-to-cement ratio of 0.43 (by weight). No chemical or mineral admixtures were used. Each specimen contained a 102-mm long strain gage positioned along the axis of the specimen to measure longitudinal strains, and a thermocouple for measuring temperature. Two days after casting, the specimens were demolded and sealed in copper jackets. After a 90-day curing period, two control specimens were tested to determine compressive strength, modulus of elasticity, Poisson's ratio, and stress-strain characteristics of the concrete. The remaining specimens were placed into environmental chambers and heated to 149°C. Some of these specimens were maintained at this temperature for either 4 hours, 7 days, 14 days, or 25 days prior to testing, while others were exposed to up to 14 thermal cycles ranging from 21° to 149° to 21°C, with a complete heating-cooling cycle taking 48 hours. The copper jackets of three

of the specimens were punctured after they reached 149°C to permit steam and moisture to escape. Thirteen of the specimens were tested at 149°C and the three that had a punctured jacket were tested at room temperature after removing the jackets. Since only one data point is available for each parameter addressed (e.g., exposure period and number of thermal cycles), the results of this study have been rated Quality Level B. The material code related to this study is 01CA020.

Sir Robert McAlpine and Sons sulfate exposure study. Two normal-weight concrete materials were fabricated to investigate the long-term durability of concretes for furnace foundation applications that could be subjected to sulfate attack. The first mix used 387 kg/m^3 of cement similar to ASTM C 150, Type I portland cement, flint gravel coarse aggregate having a coarse aggregate-to-cement ratio of 2.81 (by weight), and a water-to-cement ratio of 0.47 (by weight). The second mix used 388 kg/m^3 of sulfate-resistant cement similar to ASTM C 150, Type V portland cement, flint gravel coarse aggregate having a coarse aggregate-to-cement ratio of 2.81 (by weight), and a water-to-cement ratio of 0.46 (by weight). Each mix was used to cast 100-mm cube and 75- by 75- by 300-mm prismatic-shaped test specimens. Some of the specimens from each mix were stored continuously in water maintained at 19°C. The remainder of the specimens were placed into a 2000 ppm sodium sulfate solution maintained at 65°C. This solution was changed every 28 days for the first five years of the study and then regularly at six-month intervals. At nine designated times over the 13-year period of the study, specimens were removed from the water and sodium sulfate solutions and tested to determine compressive strengths (cube specimens) and dynamic moduli of elasticity (prismatic specimens) values for the concretes. Reference 28-day cube compressive strengths for the ordinary and sulfate-resistant cement mixes were 37.2 MPa and 49.8 MPa, respectively. These strengths correspond to equivalent cylinder compressive strengths of 29.8 MPa and 39.9 MPa, respectively. Since the average results were presented in the form of plots and not in the form of individual numerical test values, the results of this study have been rated Quality Level C. The material codes related to this study are 01CB153 and 01CB154.

Central Research Institute Electric Power Industry creep study. Concrete physical and mechanical property, and creep tests were conducted in support of the design and construction of prestressed concrete pressure vessels and containments for nuclear reactors in Japan. A normal-weight concrete was used to cast cylindrical-shaped specimens either 152-mm diameter by 305-mm long (physical and mechanical property tests) or 152-mm diameter by 610-mm long (creep tests). Each creep specimen contained a 254-mm long Carlson strain meter positioned along the axis of the specimen to measure longitudinal strains. The concrete mix used 343 kg/m^3 of cement similar to ASTM C 150, Type I portland cement, river gravel coarse aggregate having a coarse aggregate-to-cement ratio of 3.83 (by weight), and a water-to-cement ratio of 0.40 (by weight). A chemical water-reducing admixture was used in the mix. The day after casting, specimens that had been cast in copper jackets were sealed to prevent loss of moisture and then placed into a room maintained at 20°C. Unsealed specimens were either stored in water at 20°C or placed into a room maintained at 20°C and 50% relative humidity. Modulus of elasticity and compressive strength determinations were made at specified times up to concrete ages of 280 days using unsealed specimens that had been maintained at temperatures of either 20°, 40°, or 70°C, after initial moist curing at 20°C for either 27 or 96 days. Modulus of elasticity and compressive strength determinations were also made at specified times up to concrete ages of 280 days using sealed specimens that had been stored at 20°C for 27 days prior to exposure to either 20°, 40°, or 70°C. Creep (strain per unit stress) results were determined using both sealed and unsealed specimens that had been subjected to a constant stress of 8.8 MPa and maintained at either 20°, 40°, or 70°C after an initial conditioning period of 28 days at 20°C. Creep results were also determined using sealed and unsealed specimens that had been maintained at 20°C for either 96, 100, or 105 days prior to being

loaded and then subjected to temperatures of either 20°, 40°, or 70°C, respectively. Concrete porosity and drying shrinkage results were determined from unsealed specimens that had been maintained for periods up to 103 days at 20°C and 50% relative humidity prior to being subjected to either 40° or 70°C. Since individual numerical test values were not presented, the results of this study have been rated Quality Level B. The material code related to this study is 01CB155.

University of Birmingham sulfuric acid exposure study. The relative resistance to sulfuric acid attack of concretes made with ASTM C 150, Type I portland cement, 19-mm maximum size siliceous or limestone coarse aggregates, and different water-to-cement ratios was evaluated. Three concrete mixes were prepared and used to fabricate 102-mm cube test specimens. The concrete mixes had cement contents of 444 ± 3 kg/m^3, coarse aggregate-to-cement ratios (by weight) of either 2.37 (limestone) or 2.67 (siliceous), and water-to-cement ratios from 0.37 to 0.41 (by weight). Nine cube test specimens were cast from each mix and moist cured at 19 ± 1°C until 28-days old. After curing, three of the cubes from each mix were tested to determine the concrete compressive strength. The remaining six cubes from each mix were suspended in a hydraulic channel filled with a sulfuric acid solution maintained at 23 ± 4°C During the first 12 weeks of testing, the sulfuric acid concentration was 0.0016% (by weight). For the remainder of the 56-week exposure period, the sulfuric acid concentration was increased to 0.02%. Periodically, specimens were removed from the acid solution, cleaned by immersing in fresh water, surface dried, and weighed. The results were used to prepare weight loss versus time performance curves. Reference 28-day cube compressive strengths for the limestone coarse aggregate mixes having water-to-cement ratios of 0.41 and 0.39 were 61.8 MPa and 63.8 MPa, respectively. These strengths correspond to equivalent cylinder compressive strengths of 59.3 MPa and 61.2 MPa, respectively. Reference 28-day cube compressive strengths for the siliceous coarse aggregate mix having a water-to-cement ratio of 0.37 was 68.8 MPa. This strength corresponds to an equivalent cylinder compressive strength of 66.0 MPa. Since only average weight loss results are presented graphically, the data have been rated Quality Level B. The material codes related to this study are 01CA021, 01CA022, and 01CB163.

CEN/Saclay biological shield concrete study. A heavyweight concrete (2740 kg/m^3) was prepared as a part of a study to investigate the effects of elevated temperatures and neutron exposure on the physical and mechanical properties of biological shield concretes for nuclear power plant applications in France. The concrete was prepared using aluminous cement, crushed corundum and serpentine aggregates, and other aggregate materials containing rare earths. Two mix designs were used to prepare test samples with each mix containing 500 kg/m^3 of cement and having a water-to-cement ratio (by weight) of 0.38. The first mix contained 8-mm maximum size coarse aggregate and was used to cast 40- by 40- by 160-mm prisms. The second mix, a micro-concrete, contained 5-mm maximum size aggregate and was used to cast 25- by 25- by 100-mm prisms. After casting, the specimens were air cured at room temperature for either two or three days and then divided into three groups. Specimens of the first group were positioned into one of two stainless steel containers and then placed into the Triton reactor. These specimens were exposed for up to 180 days to neutron irradiation [fast fluence up to 1.4 x 10^{19} neutrons per sq cm (E > 1.0 Mev)], and thermal cycling to temperatures as high as 260°C. Specimens in the second group were subjected to the same thermal cycling as the first group, but without irradiation. Specimens from the third group were maintained at room temperature and served as control specimens. The average 100-day cube compressive strength for both concretes was 45 MPa, as determined using portions of prisms previously broken in flexure. This strength corresponds to an equivalent cylinder compressive strength of 36.0 MPa. The average 100-day flexural strength for both concretes was 3.5 MPa. Dynamic modulus of elasticity, compressive strength, flexural strength, and weight

loss versus time performance curves have been prepared. Since no 28-day baseline test results are available and only a limited number of specimens were tested, the data have been rated Quality Level C. The material code related to this study is 01DA004.

Technische Universität Braunschweig elevated temperature concrete-rebar bond study. The bond between concrete and different types of steel reinforcing bars at temperatures up to 800°C was investigated in Germany. Ten batches of concrete having the same mix proportions were prepared using 360 kg/m^3 of ordinary cement, 16-mm maximum size siliceous gravel coarse aggregate having an aggregate-to-cement ratio of 3.52 (by weight), and a water-to-cement ratio of 0.49 (by weight). Bond pull-out test specimens, 172-mm diameter by 191-mm long and 200-mm cubes were cast from each mix. Embedded along the longitudinal axis of each cylindrical test specimen was either a 16-mm diameter cold deformed steel bar (80-mm bond length), a 7.5-mm diameter deformed high-tensile strength prestressing steel bar (40-, 80-, or 110-mm bond length), or an 8-mm diameter plain rusted round mild steel bar (40-mm bond length). Twenty-four hours after casting, the specimens were demolded and placed into water to cure. After seven days, the specimens were removed from the water and stored in a room maintained at 20°C and 65% relative humidity until tested. The average 28-day cube compressive strength for the concrete mixes was 54.5 MPa. This strength corresponds to an equivalent cylinder compressive strength of 40.9 MPa. Two methods were used to test the bond pull-out specimens. In the first method, specimens with cold deformed steel bars were heated to either 300°, 370°, 500°, 600°, or 800°C; specimens with deformed high-tensile strength prestressing bars were heated to either 160°, 330°, 450°, 550°, or 600°C; and specimens with plain rusted round steel bars were heated to either 80°, 200°, 300°, 400°, 500°, or 600°C. While at temperature, the embedded steel bars were loaded and the rebar slip at the non-loaded end of the concrete specimen was measured as a function of applied pull-out force. Companion pull-out tests were conducted at room temperature. In the second method, the bond zone of the test specimen was first loaded to a specified value at room temperature (20°C). While maintaining this load, the specimen was heated until the bond between the concrete and steel reinforcement failed. Steel reinforcement slip versus temperature performance curves were developed from this data. Since the constituent materials and mix proportions were adequately identified, processing information was presented, and only bond stress versus slip graphs were reported, the performance curves have been rated Quality Level B. The material code related to this study is 01CB164.

Takenaka Komuten elevated-temperature study. The effect of elevated temperature on selected mechanical properties of normal-weight concretes having applications to nuclear power plants, water desalination plants, etc., was investigated in Japan. Two concrete mixes were used in the study. The first mix used 330 kg/m^3 of ASTM C 150, Type I portland cement, river gravel coarse aggregate having a coarse aggregate-to-cement ratio of 3.17 (by weight), a water-to-cement ratio of 0.50 (by weight), and had a 28-day compressive strength of 37.1 MPa. The second mix used 390 kg/m^3 of ASTM C 150, Type I portland cement, river gravel coarse aggregate having a coarse aggregate-to-cement ratio of 2.50 (by weight), a water-to-cement ratio of 0.50 (by weight), and had a 28-day compressive strength of 42.5 MPa. Bond pull-out test specimens containing plain steel reinforcing bars and 100-mm-diameter by 200-mm-long cylinders were cast from each mix. After casting, the specimens were moist cured at 20°C until 28-days old and then stored at 21°C and 85% relative humidity until 90-days old. At an age of 90 days some of the specimens were tested at ambient temperature to determine baseline properties and the remainder of the specimens (unsealed) were exposed to environments of either 20°C and 45% relative humidity, 35°C and 40% relative humidity, 50°C and 35% relative humidity, 65°C and 30% relative humidity, 80°C and 25% relative humidity, 110°C, 200°C, or 300°C. After exposure to these environments for 90 days, the specimens were permitted to slowly cool to room temperature prior to removal, weighing, and testing. Modulus of

elasticity, compressive strength, and splitting-tensile strength tests were conducted on both dry cylinders and cylinders that had been soaked in water for two days prior to testing. Pull-out tests were conducted on dry specimens to determine the maximum bond strength and the bond stress corresponding to a free-end slip of 0.025 mm. Although the constituent materials and mix proportions were adequately identified, the number of specimens tested at each exposure condition was not presented, therefore, the performance curves have been rated Quality Level B. The material codes related to this study are 01CB165 and 01CB166.

Brookhaven National Laboratory irradiation and sulfate exposure study. Three portland cement mortars were prepared as part of a research effort to identify modes of degradation and time periods for failure of concretes used in underground low-level radioactive waste disposal applications. The first mix used 508 kg/m^3 ASTM C 150, Type I portland cement, Ottawa sand fine aggregate having a fine aggregate-to-cement ratio of 2.75 (by weight), and a water-to-cement ratio of 0.48 (by weight). Materials and mix proportions for the second mix were the same as the first mix except for use of a ASTM C 150, Type V sulfate-resisting portland cement. The third mix used 437 kg/m^3 ASTM C 150, Type V sulfate-resisting portland cement, 77 kg/m^3 silica fume (pozzolan), Ottawa sand fine aggregate having a fine aggregate-to-cement plus pozzolan ratio of 2.75 (by weight), and a water-to-cement plus pozzolan ratio of 0.40 (by weight). Test specimens cast from each mix included 25.4-mm cubes and 25.4- by 25.4- by 254-mm prisms. After casting, the molds were sealed to prevent loss of moisture and stored for 27 to 34 days at 21°C prior to demolding. At the end of the curing period, the prismatic-shaped specimens from all three mixes were demolded and separated into two groups. The first group was subjected to a cyclic procedure involving immersion in a 2.1% Na_2SO_4 solution at room temperature for 16 hours, forced-air drying at 54° ± 1°C for 7 hours 40 minutes, and 20 minutes cooling. This procedure was repeated daily over the test duration except that on weekends the samples were left in the immersion cycle for 64 hours. The second group of specimens used the same cyclic procedure except that deionized water was used as the immersion fluid. Prior to the start of immersion cycling, and after 48 and 115 cycles, specimen lengths were measured. Following approximately 28-days of curing, the cube specimens were measured, weighed, and separated into four batches, with each batch containing specimens from each of the three mixes. The first batch was irradiated in air at about 10°C at a Co-60 gamma dose rate of 0.0086 Gy/s. Batch two was irradiated in air at about 10°C at a Co-60 gamma dose rate of 1.06 Gy/s. The remaining two batches were used as controls with one batch maintained in air at room temperature (20°C) and the second maintained in air at 10°C (the average temperature of the irradiation facility). At designated times over the 365 day test duration, sets of cubes were removed from each batch and tested to determine compressive strength. Since the constituent materials and mix proportions were adequately identified, processing information was presented, and individual test results were provided, the performance curves have been rated Quality Level A. The material codes related to this study are 01CB158 – 01CB160.

Metallic Reinforcements

Properties for uncoated, deformed, and uncoated plain carbon steel reinforcing bar material, conforming to ASTM A 615 Grades 40, 60 and 75 requirements (Ref. 3.21), are contained in Chapter 2 (Vols. 1–3) of the handbook and electronic data base file REBAR.DB. Information provided includes ambient and temperature-dependent engineering stress versus strain performance curves for both plain and deformed Grade 40, 60, and 75 materials and S-N (fatigue) performance curves for the Grade 40 material. Since the data presented do not reflect test results for each possible bar size, the performance curves were rated Quality Level B. The material codes related to these results

are 02AA004 – 02AA006, and 02AC004 – 02AC006.

Properties for uncoated, deformed, and uncoated plain billet-steel reinforcing bar material, conforming to ASTM A 15 requirements (Ref. 3.22), are contained in Chapter 2 (Vols. 1–3) of the handbook and electronic data base file REBAR.DB. The material is produced in three grades (Structural, Intermediate, and Hard) by either the open-hearth, basic oxygen, electric-furnace, or acid bessemer processes. Information provided includes ambient engineering stress versus strain performance curves for each of the grades, and S-N (fatigue) performance curves and temperature-dependent stress versus strain performance curves for the deformed, uncoated, Intermediate Grade material. Since the data presented do not reflect test results for each possible bar size, the performance curves were rated Quality Level B. The material codes related to these results are 02AA001 – 02AA003, and 02AC001 – 02AC003.

Prestressing Tendons

Properties for a carbon steel wire material conforming to ASTM A 421, Type BA wire (Ref. 3.23) having a diameter of 7.01 mm and used to post-tension concrete components, are contained in Chapter 3 (Vols. 1–3) of the handbook and electronic data base file TENDON.DB. Information provided includes a temperature-dependent engineering stress versus strain performance curve, a tensile yield strength versus temperature performance curve, an ultimate tensile strength versus temperature performance curve, and an ultimate tensile elongation versus temperature performance curve. Since these performance curves reflect test results from one reference source, each performance curve has been rated Quality Level B. The material code related to this result is 03BA003.

Structural Steels

Properties for a hot-rolled, carbon steel material conforming to ASTM A 36 requirements (Ref. 3.24) are contained in Chapter 4 (Vols. 1 – 3) of the handbook and electronic data base file STEEL.DB. Information provided includes temperature-dependent engineering stress versus strain performance curves, a tensile yield strength versus temperature performance curve, an ultimate tensile strength versus temperature performance curve, and an ultimate tensile elongation versus temperature performance curve. Since these performance curves reflect test results from only one reference source, each performance curve has been rated Quality Level B. The material code related to this result is 04AA002.

Properties for a hot-rolled, carbon steel material conforming to ASTM A 7 requirements (Ref. 3.25) are contained in Chapter 4 (Vols. 1 – 3) of the handbook and electronic data base file STEEL.DB. Information provided includes an ambient-temperature engineering stress versus strain performance curve for elastic-plastic conditions. Since this performance curve depends on the modulus of elasticity and yield strength, and was developed using the modulus of elasticity from one source and minimum yield strength specified in another source, each performance curve has been rated Quality Level C. The material code related to this result is 04AA001.

Rubbers

Temperature-dependent hardness versus time curves for an ethylene propylene diene rubber conforming to ASTM D 1418 (Ref. 3.26), are contained in Chapter 5 (Vols. 1–3) of the handbook and electronic data base file RUBBER.DB. The test results were obtained from a study sponsored by the Swedish Standards Institute and Swedish Institute of Rubber Technology. The purpose of the study was to develop a data base on the effect of various liquids on the behavior of some common rubber polymers. Samples, 25-mm diameter by 2-mm thick, were supplied by several laboratories to a hardness specification

of ~70 IRHD (International Rubber Hardness Degrees). The test specimens were immersed in water maintained at either 23°, 70°, or 100°C. After a prescribed test period of up to 56 days, each specimen was removed from the water, weighed in air and water, and tested for hardness. Since the materials were supplied from several sources and the compositions were not reported, this data has been rated Quality Level C. The material code related to this study is 05AA001.

3.4.3 Material Behavior Modeling (Subtask S.2.3)

Modeling studies under this activity are designed to complement, as closely as possible, those conducted at the National Institute of Standards and Technology under another NRC program, "Performance Criteria for Concrete-Based Barriers for Low-Level Waste (LLW) Disposal Facilities." Program personnel have participated in meetings of the American Concrete Institute Committee 365, "Service Life Predictions," RILEM Committee TC-104, "Damage Classification of Concrete Structures," RILEM Committee MLN, "Methodology for Life Prediction of Concrete Structures in Nuclear Power Plants," and an NRC-sponsored meeting "Workshop on Concrete Performance and Modeling for Low-Level Radioactive Waste Disposal (Ref. 3.27)." During a previous reporting period, a review and evaluation was conducted to identify and evaluate models and accelerated aging techniques and methodologies that can be used in making predictions of the remaining service life of concrete in nuclear power plants (Ref. 3.28). Recent activities have focused on the collation of survey data and review of the durability assessment of reinforced concrete structures at nuclear power stations in the United Kingdom.

Taywood Engineering Ltd. has completed a survey of selected United Kingdom nuclear power stations for the purpose of providing data that can be used to predict the onset of corrosion of steel reinforcement due to either carbonation or chloride ingress. Corrosion of steel reinforcement was recognized to be a two stage process: activation and propagation. The present state-of-the-art for modeling carbonation and chloride migration into concrete was reviewed along with factors that affect the time to corrosion activation, e.g., concentration level, environmental conditions, and diffusion coefficient. Nuclear power stations from which data were obtained include the Windscale Advanced Gas-Cooled Reactor (WAGR), Hinkley Point "A," Bradwell, and Wylfa. Data previously obtained and reported by Nuclear Electric at Trawsfynydd, Oldbury and Dungeness "A" stations were also included in the study. Survey data obtained at selected locations in these stations included chloride content, carbonation depth, compressive strength, resistivity, moisture content, sorptivity, oxygen diffusion, and petrography. Environmental conditions (temperature, relative humidity, and rainfall) and survey results were used to predict the onset of steel reinforcement corrosion due to either carbonation or chloride ingress at selected locations in these stations. Results of the condition surveys at these stations were used to derive several general conclusions. The steel reinforcement and internal concrete were visibly sound in the majority of the structures, with the limited occurrences of fine cracking not generally associated with reinforcement corrosion. Examination of the external concrete indicated that visible degradation was limited to a few localized areas of cracking and spalling, and corrosion of exposed reinforcement or bolts. The main exception to the above general findings was the cooling plant structure at Wylfa where some cracking, rust staining, and spalling was observed, with substantial loss of reinforcement section. The cause of this deterioration was attributed to high levels of chloride penetration resulting from the highly saline environment. Measured depths of carbonation of the internal concrete were up to 50 mm in certain cases (WAGR), however, when considering the measured concrete strength, it was clear that this was a function of the severity of the exposure (in terms of carbonation), rather than poor concrete. Where interior concrete surfaces had been treated with a coating, they were less susceptible to carbonation than uncoated surfaces. Chloride penetration depths for both external and

internal concrete were generally low (e.g., 0.05% by weight within 10 mm of concrete surface), except at the Wylfa cooling water plant. Steel reinforcement potential measurements indicated that there was no active corrosion occurring, however, additional measurements of resistivity and moisture content suggested that external concrete and wetted internal concrete were in an environment that would support corrosion once activated. The concretes examined were free of alkali-silica reactions and sulfate attack, and exhibited compressive strengths of 30 to 70 MPa, except at Wylfa where strengths as low as 23 MPa were measured. A model to predict chloride-induced corrosion of steel reinforcement and a refined model developed to predict carbonation of concrete, taking into account environment, cement type, curing, and concrete strength (primarily water-to-cement ratio), were applied to the survey data. Predicted minimum age for steel reinforcement corrosion activation due to carbonation or chloride ingress ranged from 31 to >140 years and 43 to >140 years, respectively. More detailed information on these results is presented in Ref. 3.29.

3.5 PLANNED ACTIVITIES

Incorporation of materials property data into the SMIC will be completed and a report issued. The report updating the original program report that reviewed materials property databases (Ref. 3.3.) will be issued.

3.6 REFERENCES

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3.14 *Microsoft Access,* **Version 2.0**, Microsoft Corporation, One Microsoft Way, Redmond, Washington.

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3.17 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.

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3.19 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, Il.), Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tennessee, January 1992.

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3.26 "Practice for Rubber and Rubber Lattices," ASTM D 1418, *Annual Book of ASTM Standards*, **Vol. 09.01**, American Society of Testing and Materials, Philadelphia, Pennsylvania, 1989.

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3.29 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.

3.7 MILESTONE STATEMENT AND SCHEDULE

The statement and schedule for the milestones in Task S.2 are given in the following charts.

MILESTONE STATEMENT AND SCHEDULE

TASK: S.2 MATERIALS PROPERTY DATABASE

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.2 MATERIALS PROPERTY DATA BASE (MPDB)																							
S.2.1 Structural Materials Information Center (SMIC) Forumulation/ Implementation																							
A. Issue Report Describing Potential Applications of SMIC (Activity Completed)		a							a					b	c								
B. Issue Reports Presenting SMIC Developed for Concrete and Concrete-Related Materials																							
a. Complete Annual Revision of SMIC and Provide Results to Sponsor																							
b. Complete Draft Report on Data Base Management Systems Assessment and Provide Recommendations																							
c. Complete Draft Report Summarizing Materials Contained in SMIC																							
C. Issue Structural Materials Handbook Volumes and Electronic Data Base Files to NRC																							

MILESTONE STATEMENT AND SCHEDULE

TASK: S.2 MATERIALS PROPERTY DATABASE

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.2.2 Data Collection																							
A. Complete incorporation of available materials property data into SMIC (literature, foreign contacts, etc.)																							
B. Complete Incorporation of Prototypical Structural Materials Property Data Into SMIC (nuclear, fossil plants, etc.			a, b							c													
a. Issue Reports on Portland Cement Association Long-Term Study and Testing of United Kingdom Nuclear Power Station Concretes																							
b. Issue Report on Durability Assessments of Reinforced Concrete Structures in Selected United Kingdom Nuclear Power Stations																							
c. Complete Testing and Report Results for Specimens Obtained from Prototypical Concrete Structures																							

MILESTONE STATEMENT AND SCHEDULE

TASK: S.2 MATERIALS PROPERTY DATABASE

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.2.3 Material Behavior Modeling/Evaluations																							
A. Issue Report on Mathematical Modeling of Aging and Environmental Stressor Effects (Activity Completed)			a														b						
B. Complete Special Technical Assessments of Structural Material Behavior																							
a. Issue Report on Chloride- and Non-Chloride-Induced Corrosion of Reinforced Concrete Structures																							
b. Complete Additional Materials Evaluations (as identified)																							

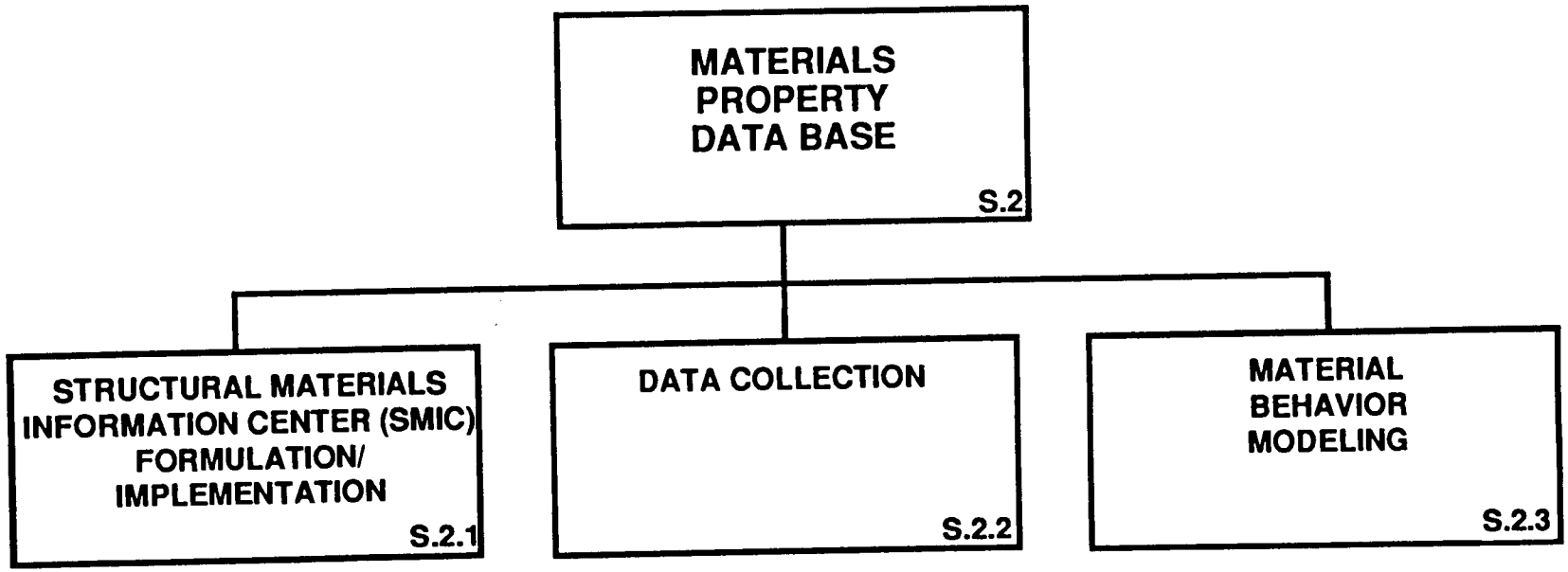


Fig. 3.1 Level 2 work breakdown structure for SAG Task S.2: Material Property Data Base.

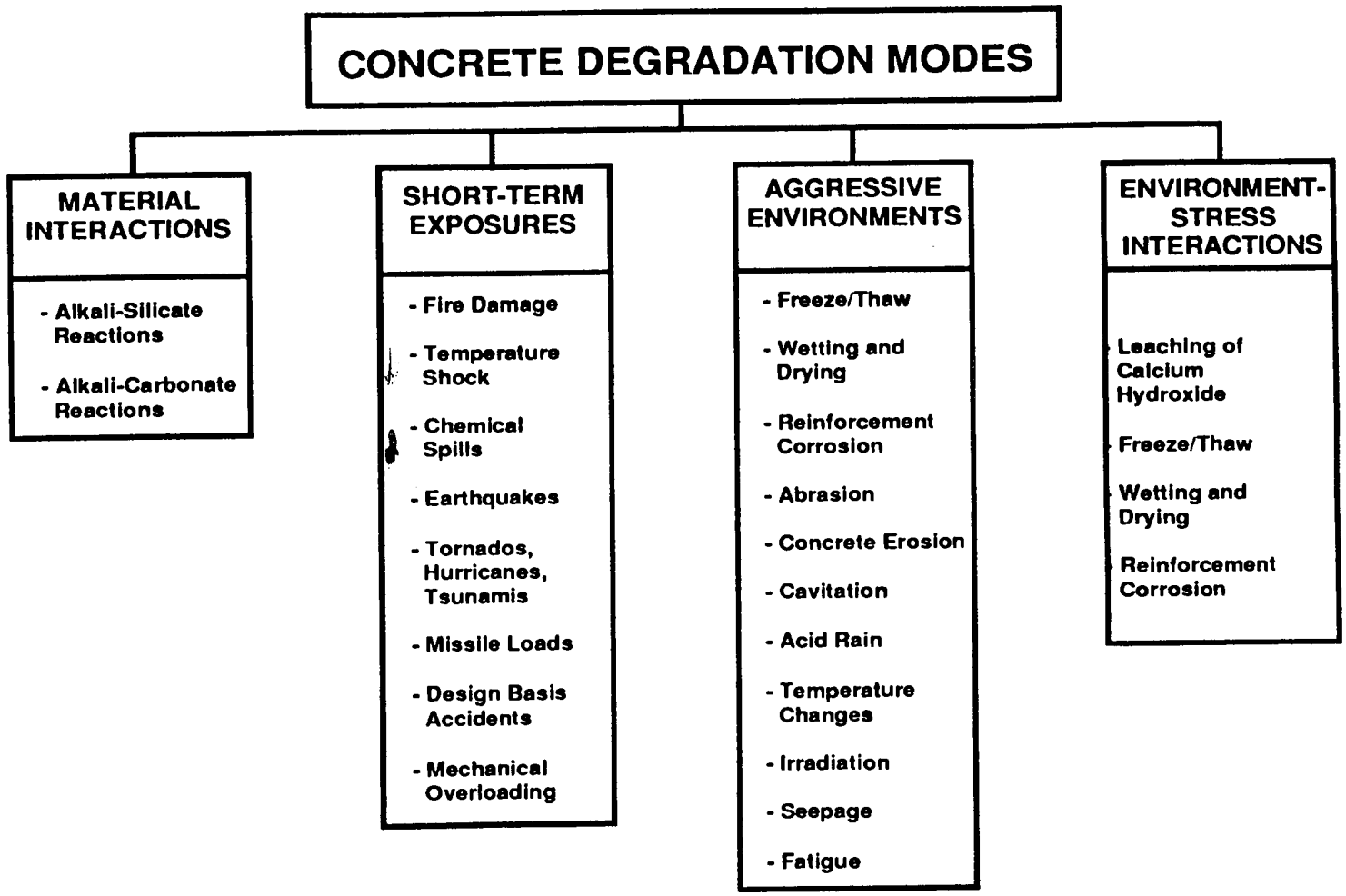


Fig. 3.2 Listing of potential concrete degradation factors.

Source: 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, February 1988).

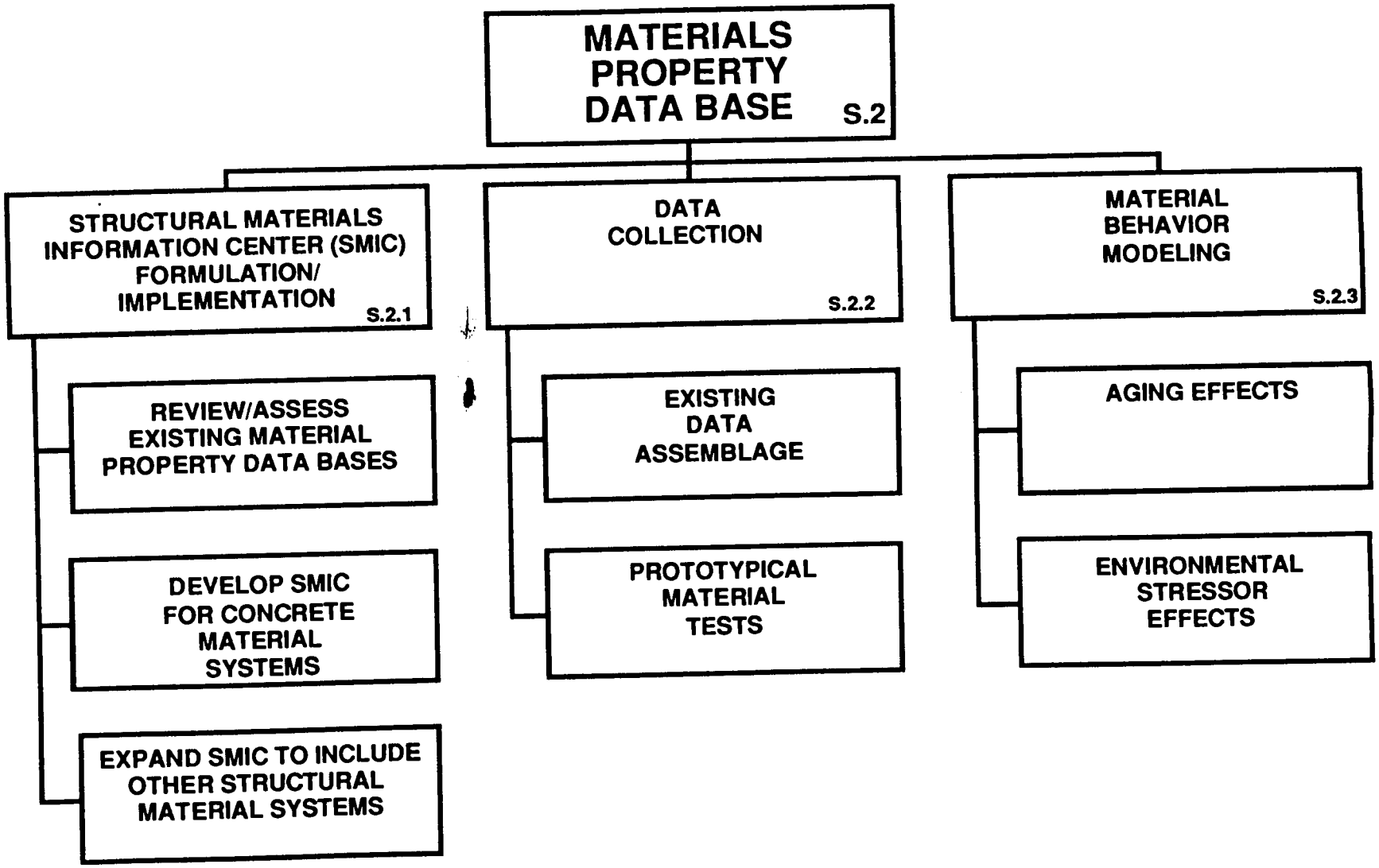


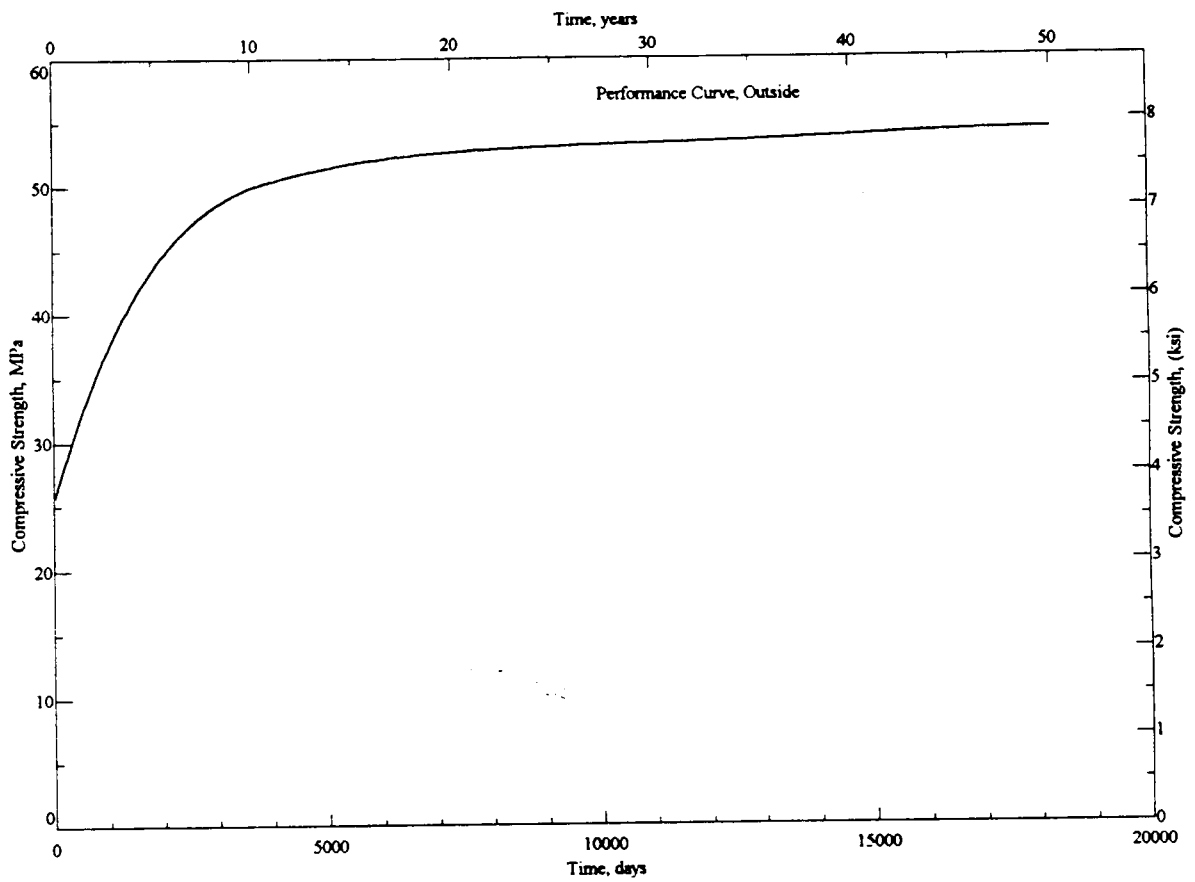
Fig. 3.3 Level 3 work breakdown structure for SAG Task S.2: Material Property Data Base.

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. 3.4 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 Day	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 27).

Fig. 3.5 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		
	Mix Proportions per Unit Volume		
Constituent Material	kg/m^3	lb/yd^3	Property Code
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 27).

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. 3.6 Example of page from Vol. 3 (Material Data Sheet) of Structural Materials Handbook.

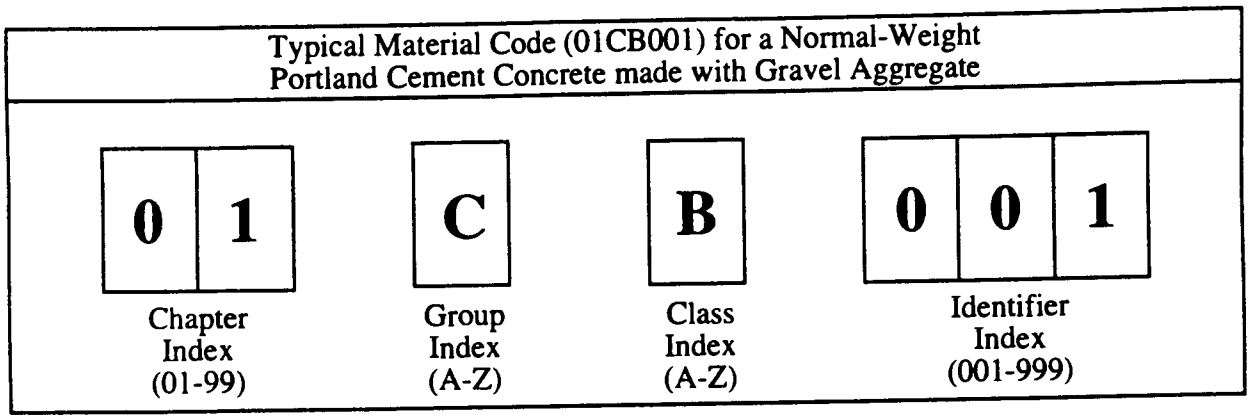
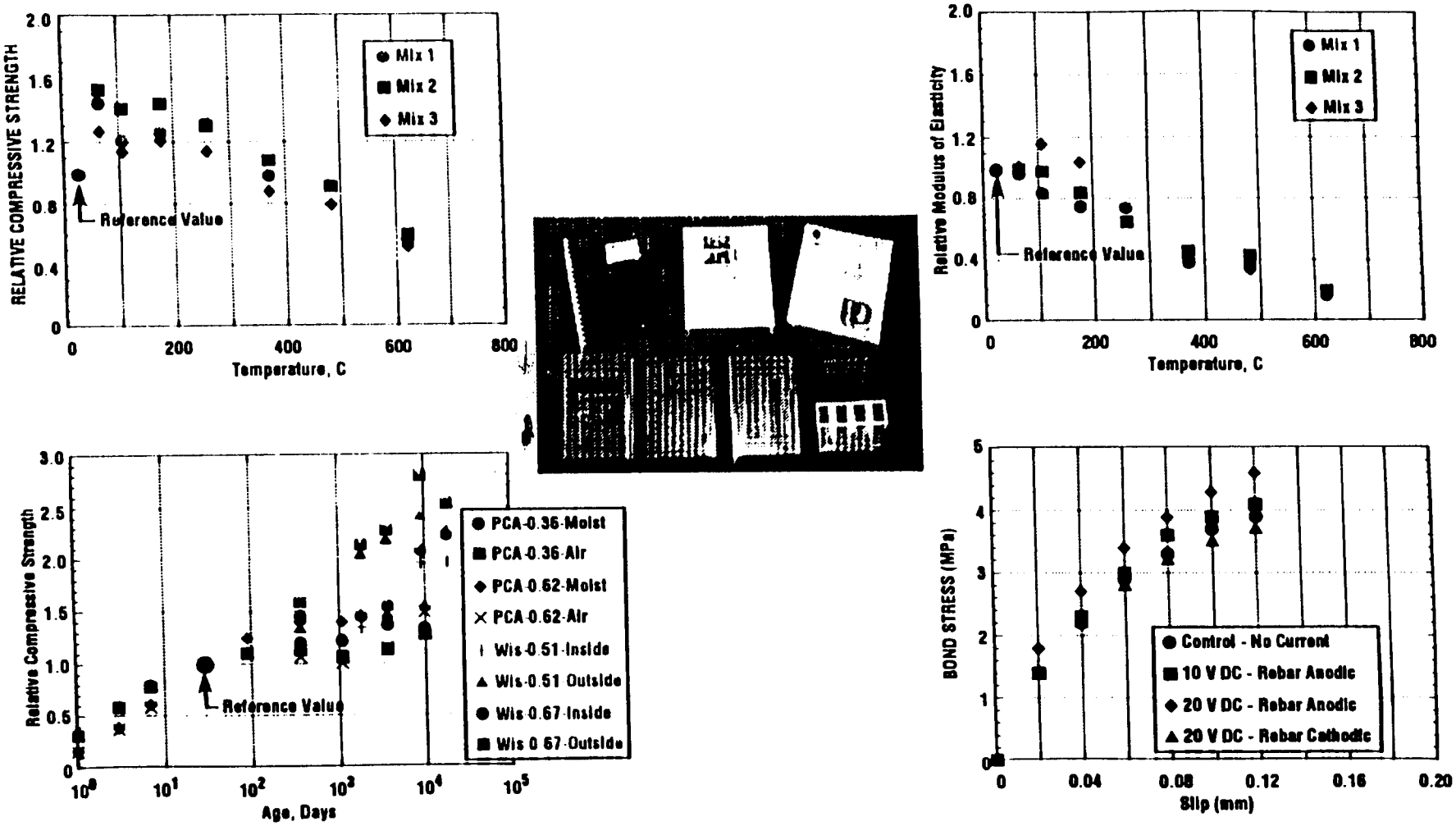


Fig. 3.7. Material code arrangement.



ORNL-DWG 92M-4159C ETD

Fig. 3.8 Examples of types of data contained in the Structural Materials Information Center that were derived from reference documents.

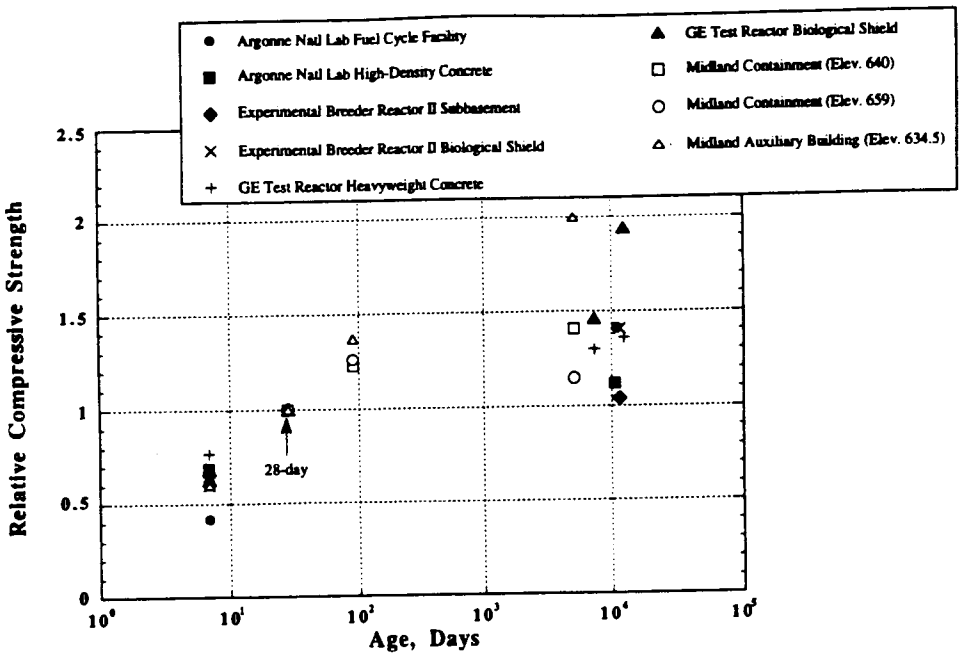


Fig. 3.9 Relative compressive strength for concretes obtained from several U.S. nuclear power facilities

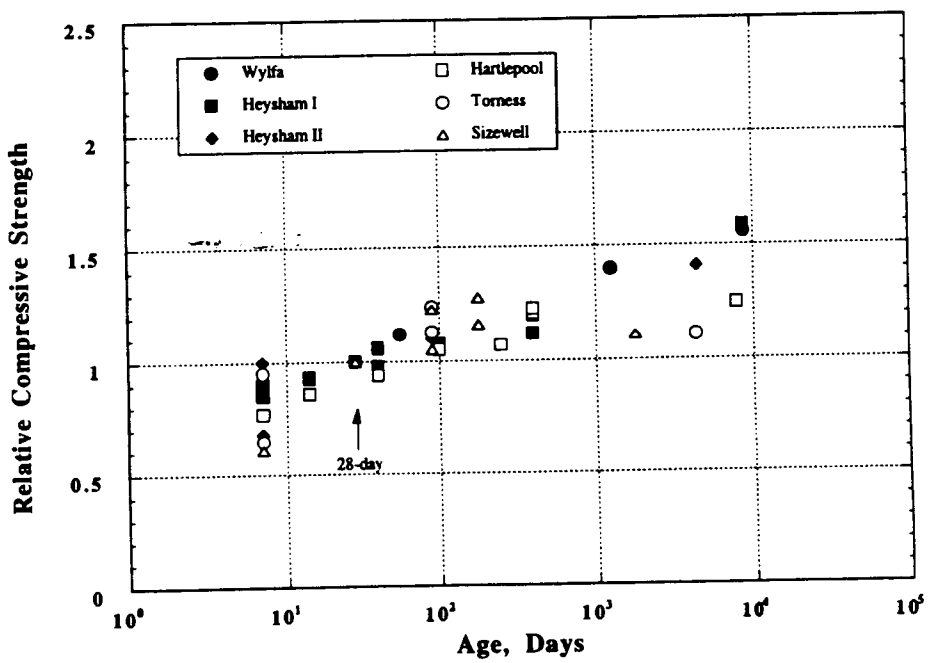


Fig. 3.10 Relative compressive strength for concretes obtained from several U.K. nuclear power facilities

Table 3.1 Baseline information to classify data base entry as "high quality."

1. Material Selection

Mixture proportions
Constituent composition/characteristics
Admixtures

2. Mixing/Placing/Curing

Mixing conditions
Schedule
Placing/casting conditions
Curing/environmental conditions

3. Properties

Unit weight
Air content/porosity/pore size distribution/permeability
Time of set
Bleeding
Compressive strength
Modulus of elasticity
Poisson's ratio
Tensile strength
Bond strength
Coefficient of thermal expansion
Coefficient of thermal diffusivity
Coefficient of conductivity

Table 3.2 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-product
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 Feature
A	Carbon Steels	A	Hot- or Cold-Rolled Steels
B	Stainless Steels	B	Bolting Materials
		C	Special Materials

Table 3.2 (cont.) Material code identification and description.

Rubbers Handbook Chapter Index 05 - Electronic Data Base File RUBBER.DB			
Group Index	Group Index Description Type of Rubber	Class Index	Class Index Description Type of Rubber
A	ASTM D 1418, Class M	A	ACM, AEM, ANM, CM, CFM, CSM, EAM, EPDM, EPM, FFKM, or FKM (ASTM D 1418, Class M rubbers having a saturated chain of the polymethylene type)
B	ASTM D 1418, Class N		
C	ASTM D 1418, Class O		
D	ASTM D 1418, Class R	B	(ASTM D 1418, Class N rubbers having nitrogen, but not oxygen or phosphorus in the polymer chain)
E	ASTM D 1418, Class Q		
F	ASTM D 1418, Class T	C	CO, ECO, or GPO (ASTM D 1418, Class O rubbers having oxygen in the polymer chain)
G	ASTM D 1418, Class U		
H	ASTM D 1418, Class Z	D	ABR, BIIR, BR, CIIR, IIR, IR, NBR, NCR, NIR, NR, PBR, PSBR, SBR, SCR, or SIR (ASTM D 1418, Class R rubbers having an unsaturated carbon chain)
I	Other types of rubbers		
		E	XSBR or XNBR (ASTM D 1418, Class R rubbers having substitute carboxylic acid (COOH) groups on the polymer chain)
		F	FVMQ, PMQ, PVMQ, MQ, or VMQ (ASTM D 1418, Class Q rubbers having silicon and oxygen in the polymer chain)
		G	OT or EOT (ASTM D 1418, Class T rubbers having sulfur in the polymer chain)
		H	AFMU, AU, or EU (ASTM D 1418, Class U rubbers having carbon, oxygen, and nitrogen in the polymer chain)
		I	FZ or PZ (ASTM D 1418, Class Z rubbers having phosphorus and nitrogen in the polymer chain)
		J	Mixtures of rubbers

Table 3.3 Property code range description.

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 3.4 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 3.5 Requirements for evaluating the quality of data and values.

Requirements for Determining the Quality of Data and Values	
Requirement Number	Requirement Description
1	Completeness of the material description
2	Stability of the material
3	Type of input from the references or sources [actual experimental observations (data) or results of previous analyses (values)]
4	Completeness of the data or values search
5	Completeness of the resources (completeness of the consideration given to all available data or values)
6	Quality of the references or sources
7	Availability of the data or values (completeness of data or values field coverage)
8	Consistency of the data or values with respect to related properties
9	Precision or scatter of the data or values
10	Uncertainty of the data or values (systematic error or bias)
11	Method used to determine the reported property (averaging, curve fitting, synthesization, derivation or extrapolation of the source data or values)

Table 3.6. Requirements for a data base management system for the Structural Materials Information Center.

Data Base Management System Requirement	Mat.DB, Version 1.22 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. 3.5 and 3.7). 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 characteristics. 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 comparisons of time- and environment-dependent performance relatively easy and accurate.	A new version of Mat.DB is currently being developed using Microsoft Windows (Ref. 3.13). This version is expected to include the ability to overlay multiple windows of data, but superimposing graphs may not be possible.

Table 3.6. (cont.) Requirements for a data base management system for the Structural Materials Information Center.

Data Base Management System Requirement	Mat.DB, Version 1.22 Weaknesses and Limitations
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.
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 answer 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 identity of reported data and values. These fields do not contain information that provides guidance to the end user.

Table 3.6. (cont.) Requirements for a data base management system for the Structural Materials Information Center.

Data Base Management System Requirement	Mat.DB, Version 1.22 Weaknesses and Limitations
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 3.7 Organizations providing material property data.

Organization	Result
a. U.S. Research Establishments	
Bradley University	Report on hydraulic conductivity of cement pastes
Bureau of Reclamation	Reports containing baseline information and 10-year property data on dams
Concrete Reinforcing Steel Institute	Information on properties of concrete reinforcing materials
Construction Technology Laboratories	Reports on long-term properties of concrete and fatigue of rebars
Corps of Engineers - Vicksburg, Miss.	Information on maintenance and repair materials, and petrographic and physical examination of radioactive concrete
CORRPRO Companies, Inc.	Information on chloride- and non-chloride induced corrosion of reinforced concrete
Howard University	Information on damage classification of concrete structures
Idaho National Engineering Laboratory	Report on service life modeling of reinforced concrete
Materials Research Laboratories (Pennsylvania State University)	Information on cementitious materials for waste fixation, and service-life prediction
Michigan State University	Reports on concrete durability and repair
Multiple Dynamics Corp.	Reports on durability of concrete materials and structures in nuclear power plants
NSF Center for Science and Technology of Advanced Cement-Based Materials	Information on high performance concretes and concrete fracture
National Institute of Standards and Technology	Reports on modeling to predict service life of concrete components
Prestressed Concrete Institute	Information on properties of concrete prestressing materials
Singleton Laboratories	Reports on properties of concrete at elevated temperature
Strategic Highway Research Program	Information on multiyear concrete research program addressing alkali-silica reactions, durability, inspection and repair
University of Notre Dame	Reports on damage and damage classification of concrete due to chemical processes
University of Wisconsin	Data on concrete properties to 50 years age
Virginia Power Co.	Information on inspection and aging of nuclear power plant concrete structures
Wiss, Janney, Elstner Assoc.	Information on petrographic examinations, mix design development, and mechanical testing of concrete materials for nuclear power plants

Table 3.7 (cont.) Organizations providing material property data.

Organization	Result
b. Foreign Research Establishments	
AEA Technology (England)	Reports on long-term durability of concrete, corrosion of steel in concrete, and non-destructive testing techniques
Basler & Hofmann (Switzerland)	Report on repair techniques; information on concrete structures in nuclear power plants
British Cement Assoc. (England)	Information on European research related to service-life prediction and damage classification of concrete structures; reports on alkali-silica reactions
Building Research Establishment (England)	Information and reports on alkali-silica reactions, structural assessments and long-term performance of concrete
Canada Center for Mineral and Energy Technology (Canada)	Reports on long-term durability and strength development of concrete
CEC (Luxenbourg)	Report on historical examination of concrete; listing of publications; annual report on R&D program for decommissioning nuclear installations
Central Electricity Generating Board (England)	Report on elevated temperature effects on concrete
Central Research Institute of Electric Power Industry (CRIEPI) (Japan)	Reports on long-term and elevated temperature behavior of concretes associated with design of high-level radioactive waste facilities
Darmstadt Technische Hochschule (Germany)	Reports on concrete properties, nondestructive testing and structural behavior
Deutscher Auschuss für Stahlbeton (Germany)	Report on 30-year properties of concrete
Electricite' de France (France)	Reports and information on containment design and long-term performance of nuclear power plant concrete structures
ENEA (Italy)	Information on elevated temperature effects on concrete properties
German Commission on Reinforced Concrete (Germany)	Reports on water content and porosity of concrete at high temperature
Institut fur Massivbau und Baustofftechnologie (Germany)	Reports on creep and repair of concrete
Industria Italiana del Cemento (Italy)	Reports on reliability evaluation of existing structures
Institute for Fire Safety and Design (IDEON) (Sweden)	Reports on elevated temperature behavior of concrete and reinforcing materials
Japan Atomic Energy Research Institute (JAERI) (Japan)	Data from testing of concrete cores obtained from Japan power demonstration reactor

Table 3.7 (cont.) Organizations providing material property data.

Organization	Result
b. Foreign Research Establishments (cont.)	
Japan Power Engineering and Inspection Corporation (JAPEIC) (Japan)	Information on Japanese nuclear power plant life extension program and how it relates to the concrete materials
Kajima Technical Research Institute (Japan)	Reports on properties of concrete at elevated temperature and concrete removed from Japan power demonstration reactor; information on rehabilitation of concrete structures
Lehrstuhl für Baustoffkunde und Werkstoffprufung der TU München (Germany)	Information on long-term, elevated temperature and multiaxial performance of concrete; information on damage classification
National Swedish Council for Building Research (Sweden)	Information on performance of building materials and structures
Nuclear Electric (England)	Information on long-term performance of prestressed concrete pressure vessels
Obayashi Corporation (Japan)	Reports on properties of concrete at elevated temperature; information on corrosion of concrete reinforcing materials, cementitious materials use in radioactive waste depositories, and boron-containing concretes for neutron absorption
Ontario Hydro (Canada)	Reports on service performance of nuclear containment concrete and aging of materials and components
Ontario Ministry of Transportation (Canada)	Reports on cathodic protection of reinforced concrete structures
Taywood Engineering, Ltd. (England)	Reports on concrete properties, repair, service life, durability, and performance of prestressing systems
Universität Karlsruhe (Germany)	Reports on effects of elevated temperatue and irradiation on concrete and its constituents
University of Toronto (Canada)	Reports on concrete permeability and migration of moisture and their effects on aging of concrete containments

Table 3.8 Organizations contacted to provide prototypical data or concrete samples for testing.

Organization	Candidate facility	Status
Argonne National Laboratory	EBR-II	Samples provided/tested from air cell foundation, shield wall, and containment building
	EBWR and CP-5	Terminated
Battelle Pacific Northwest Laboratory	Shippingport Power Station	Samples provided/tested from reactor enclosure refueling ring and reactor enclosure basement
Commonwealth Edison	Braidwood Byron Dresden LaSalle Quad Cities Zion	Samples provided/tested from each plant, awaiting baseline information
Construction Technology Laboratories	Long-term study	Archived specimens tested providing concrete data for ages to 42 years
Consumers Power Co.	Midland Palisades	Samples provided from Midland and Palisades plants, Midland samples tested
Dairyland Power Cooperative	LaCrosse	Terminated, not cost effective
Detroit Edison	Fermi	Terminated, no utility interest
General Electric	Vallicetos	Samples of normal-weight and heavyweight concretes tested
Northern States Power	Pathfinder	Terminated, original records unavailable
Taywood Engineering, Ltd.	Several British nuclear power stations	Specimens tested from Wylfa, Heysham I, Hartlepool, Heysham II, Torness, and Sizewell B
Union Carbide	CintiChem	Terminated, original records unavailable

Table 3.9 Summary of materials in the Structural Materials Information Center.

Material Type	Number of Materials
Portland Cement Concretes	
Lightweight	1
Normal-Weight	123
Heavyweight	4
Metallic Reinforcements	12
Prestressing Tendons	1
Structural Steels	2
Rubbers	1
Total	144

Table 3.10 Portland cement concretes in the Structural Materials Information Center.

Material Code	Common Name	Material Description
01BC001	Abrams, Mix V Portland Cement Association, Skokie, Illinois	Structural Lightweight Manufactured Aggregate
01CA001	Series B, Lannon Dolomite University of Wisconsin, Madison,Wisconsin	Normal-Weight Crushed Stone Aggregate
01CA002	Series B, Red Granite University of Wisconsin, Madison,Wisconsin	Normal-Weight Crushed Stone Aggregate
01CA003	Nasser and Chakraborty, 1981 University of Saskatchewan, Canada	Normal-Weight Crushed Stone Aggregate
01CA004	CANMET, Limestone Mix 1 Canada Centre for Mineral and Energy Technology	Normal-Weight Crushed Stone Aggregate
01CA005	CANMET, Limestone Mix 2 Canada Centre for Mineral and Energy Technology	Normal-Weight Crushed Stone Aggregate
01CA006	CANMET, Limestone Mix 3 Canada Centre for Mineral and Energy Technology	Normal-Weight Crushed Stone Aggregate
01CA007	CANMET, Dolostone Mix 4 Canada Centre for Mineral and Energy Technology	Normal-Weight Crushed Stone Aggregate
01CA008	CANMET, Dolostone Mix 5 Canada Centre for Mineral and Energy Technology	Normal-Weight Crushed Stone Aggregate
01CA009	CANMET, Dolostone Mix 6 Canada Centre for Mineral and Energy Technology	Normal-Weight Crushed Stone Aggregate
01CA010	PCA - Series 356, Mix XL1 Portland Cement Association, Skokie, Illinois	Normal-Weight Crushed Stone Aggregate
01CA011	PCA - Series 356, Mix XL2 Portland Cement Association, Skokie, Illinois	Normal-Weight Crushed Stone Aggregate
01CA012	PCA - Series 356, Mix XL3 Portland Cement Association, Skokie, Illinois	Normal-Weight Crushed Stone Aggregate
01CA013	PCA - Series 356, Mix XhL1 Portland Cement Association, Skokie, Illinois	Normal-Weight Crushed Stone Aggregate
01CA014	PCA - Series 356, Mix XhL2 Portland Cement Association, Skokie, Illinois	Normal-Weight Crushed Stone Aggregate
01CA015	PCA - Series 356, Mix XhL3 Portland Cement Association, Skokie, Illinois	Normal-Weight Crushed Stone Aggregate

Table 3.10 (cont.) Portland cement concretes in the Structural Materials Information Center.

Material Code	Common Name	Material Description
01CA016	Midland, Mix E-2 Midland Nuclear Power Plant, Midland, Michigan	Normal-Weight Crushed Stone Aggregate
01CA017	Midland, Mix C-1 Midland Nuclear Power Plant, Midland, Michigan	Normal-Weight Crushed Stone Aggregate
01CA018	Nasser and Lohtia, 1971 University of Saskatchewan, Canada	Normal-Weight Crushed Stone Aggregate
01CA019	Nasser and Marzouk, 1979 University of Saskatchewan, Canada	Normal-Weight Crushed Stone Aggregate
01CA020	Bertero and Polivka, 1972 University of California, Berkeley, California	Normal-Weight Crushed Stone Aggregate
01CA021	U. of Birmingham, Mix L1 University of Birmingham, United Kingdom	Normal-Weight Crushed Stone Aggregate
01CA022	U. of Birmingham, Mix C1 University of Birmingham, United Kingdom	Normal-Weight Crushed Stone Aggregate
01CA023	Wylfa NPS Taywood Engineering Ltd., London, England	Normal-Weight Crushed Stone Aggregate
01CA024	Heysham I NPS Taywood Engineering Ltd., London, England	Normal-Weight Crushed Stone Aggregate
01CA025	Heysham II NPS Taywood Engineering Ltd., London, England	Normal-Weight Crushed Stone Aggregate
01CA026	Hartlepool NPS Taywood Engineering Ltd., London, England	Normal-Weight Crushed Stone Aggregate
01CA027	Torness NPS Taywood Engineering Ltd., London, England	Normal-Weight Crushed Stone Aggregate
01CB001	ANL-West Fuel Cycle Facility Argonne National Lab. - West, Idaho Falls, Idaho	Normal-Weight Gravel Aggregate
01CB002	Series B, Janesville, 0.51 University of Wisconsin, Madison, Wisconsin	Normal-Weight Gravel Aggregate
01CB003	Series B, Janesville, 0.67 University of Wisconsin, Madison, Wisconsin	Normal-Weight Gravel Aggregate
01CB004	Series B, Janesville, 0.41 University of Wisconsin, Madison, Wisconsin	Normal-Weight Gravel Aggregate

Table 3.10 (cont.) Portland cement concretes in the Structural Materials Information Center.

Material Code	Common Name	Material Description
01CB005	Series B, Janesville, 0.69 University of Wisconsin, Madison, Wisconsin	Normal-Weight Gravel Aggregate
01CB006	EBR-II, Lift 5, Subbasement Argonne National Lab. - West, Idaho Falls, Idaho	Normal-Weight Gravel Aggregate
01CB007	EBR-II, Biological Shield Argonne National Lab. - West, Idaho Falls, Idaho	Normal-Weight Gravel Aggregate
01CB008	Walz, Series A Stuttgart, Germany	Normal-Weight Gravel Aggregate
01CB009	Walz, Series B Stuttgart, Germany	Normal-Weight Gravel Aggregate
01CB010	Walz, Series C Stuttgart, Germany	Normal-Weight Gravel Aggregate
01CB011	Walz, Series D Stuttgart, Germany	Normal-Weight Gravel Aggregate
01CB012	Walz, Series E Stuttgart, Germany	Normal-Weight Gravel Aggregate
01CB013	Walz, Series F Stuttgart, Germany	Normal-Weight Gravel Aggregate
01CB014	Walz, Series G Stuttgart, Germany	Normal-Weight Gravel Aggregate
01CB015	Walz, Series H Stuttgart, Germany	Normal-Weight Gravel Aggregate
01CB016	Kondo, Control University of Kyoto, Japan	Normal-Weight Gravel Aggregate
01CB017	Kondo, 10% Fly Ash University of Kyoto, Japan	Normal-Weight Gravel Aggregate
01CB018	Kondo, 20% Fly Ash University of Kyoto, Japan	Normal-Weight Gravel Aggregate
01CB019	Kondo, 30% Fly Ash University of Kyoto, Japan	Normal-Weight Gravel Aggregate
01CB020	Kondo, 0.25% Calcium Chloride University of Kyoto, Japan	Normal-Weight Gravel Aggregate

Table 3.10 (cont.) Portland cement concretes in the Structural Materials Information Center.

Material Code	Common Name	Material Description
01CB021	Kondo, 1.0% Calcium Chloride University of Kyoto, Japan	Normal-Weight Gravel Aggregate
01CB022	Kondo, Calcium Lignosulfonate University of Kyoto, Japan	Normal-Weight Gravel Aggregate
01CB023	Kondo, 0.25% and 0.25% University of Kyoto, Japan	Normal-Weight Gravel Aggregate
01CB024	Kondo, 1.0% and 0.25% University of Kyoto, Japan	Normal-Weight Gravel Aggregate
01CB025	SABNGS No.2, 5.4 Bag Plain Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB026	SABNGS No.2, 6 Bag Plain Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB027	SABNGS No.2, 7 Bag Plain Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB028	SABNGS No.2, 5.4 Air Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB029	SABNGS No.2, 6 Air Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB030	SABNGS No.2, 7 Air Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB031	SABNGS No.2, 5.4 Fly Ash Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB032	SABNGS No.2, 6 Fly Ash Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB033	SABNGS No.2, 7 Fly Ash Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB034	Stewartville G.S., Core Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB035	Stewartville G.S., Face Ontario Hydro, Toronto, Ontario, Canada	Normal-Weight Gravel Aggregate
01CB036	PCA - Series 356, Mix AV1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate

Table 3.10 (cont.) Portland cement concretes in the Structural Materials Information Center.

Material Code	Common Name	Material Description
01CB037	PCA - Series 356, Mix AV2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB038	PCA - Series 356, Mix AV3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB039	PCA - Series 356, Mix DV1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB040	PCA - Series 356, Mix DV2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB041	PCA - Series 356, Mix DV3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB042	PCA - Series 356, Mix EV1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB043	PCA - Series 356, Mix EV2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB044	PCA - Series 356, Mix EV3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB045	PCA - Series 356, Mix XV1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB046	PCA - Series 356, Mix XV2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB047	PCA - Series 356, Mix XV3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB048	PCA - Series 356, Mix XW1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB049	PCA - Series 356, Mix XW2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB050	PCA - Series 356, Mix XW3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB051	PCA - Series 356, Mix AhV1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB052	PCA - Series 356, Mix AhV2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate

Table 3.10 (cont.) Portland cement concretes in the Structural Materials Information Center.

Material Code	Common Name	Material Description
01CB053	PCA - Series 356, Mix AhV3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB054	PCA - Series 356, Mix DhV1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB055	PCA - Series 356, Mix DhV2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB056	PCA - Series 356, Mix DhV3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB063	PCA - Series 356, Mix XhW1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB064	PCA - Series 356, Mix XhW2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB065	PCA - Series 356, Mix XhW3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB068	PCA - Series 374, Mix 11T1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB069	PCA - Series 374, Mix 11T2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB093	PCA - Series 374, Mix 19B1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB094	PCA - Series 374, Mix 19B2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB097	PCA - Series 374, Mix 21A Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB098	PCA - Series 374, Mix 21B Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB099	PCA - Series 374, Mix 21T1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB100	PCA - Series 374, Mix 21T2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB110	PCA - Series 374, Mix 31A Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate

Table 3.10 (cont.) Portland cement concretes in the Structural Materials Information Center.

Material Code	Common Name	Material Description
01CB111	PCA - Series 374, Mix 31B Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB124	PCA - Series 374, Mix 43A3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB125	PCA - Series 374, Mix 51A Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB126	PCA - Series 374, Mix 51B Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB127	PCA - Series 436, Mix 1 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB128	PCA - Series 436, Mix 2 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB129	PCA - Series 436, Mix 3 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB130	PCA - Series 436, Mix 4 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB131	PCA - Series 436, Mix 5 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB132	PCA - Series 436, Mix 6 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB133	PCA - Series 436, Mix 7 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB134	PCA - Series 436, Mix 8 Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB151	GETR, Mix 57-1842 General Electric Test Reactor, Vallecitos, California	Normal-Weight Gravel Aggregate
01CB153	Mears, Mix BF Sir Robert McAlpine and Sons, Ltd., London, England	Normal-Weight Gravel Aggregate
01CB154	Mears, Mix AF Sir Robert McAlpine and Sons, Ltd., London, England	Normal-Weight Gravel Aggregate
01CB155	Seki and Kawasumi, CRIEPI Central Research Institute Electric Power Industry, Japan	Normal-Weight Gravel Aggregate

Table 3.10 (cont). Portland cement concretes in the Structural Materials Information Center.

Material Code	Common Name	Material Description
01CB156	Hanford Concrete, Mix 3K Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB157	Hanford Concrete, Mix 4.5K Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB158	BNL, Portland I Brookhaven National Lab., Upton, New York	Normal-Weight Gravel Aggregate
01CB159	BNL, Portland V Brookhaven National Lab., Upton, New York	Normal-Weight Gravel Aggregate
01CB160	BNL, Portland V/SF Brookhaven National Lab., Upton, New York	Normal-Weight Gravel Aggregate
01CB161	Abrams, Mix I Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB162	Abrams, Mix III Portland Cement Association, Skokie, Illinois	Normal-Weight Gravel Aggregate
01CB163	U. of Birmingham, Mix M1 University of Birmingham, United Kingdom	Normal-Weight Gravel Aggregate
01CB164	Tech. U Braunschweig, 1981 Technical Univ. Braunschweig, Germany	Normal-Weight Gravel Aggregate
01CB165	Takenaka, Mix 1 Takenaka Komuten Co., Ltd., Tokyo, Japan	Normal-Weight Gravel Aggregate
01CB166	Takenaka, Mix 2 Takenaka Komuten Co., Ltd., Tokyo, Japan	Normal-Weight Gravel Aggregate
01CB167	Sizewell 'B' NPS Taywood Engineering Ltd., London, England	Normal-Weight Gravel Aggregate
01DA001	ANL-West Argon Cell Wall Argonne National Lab. - West, Idaho Falls, Idaho	Heavyweight Crushed Stone Aggregate
01DA002	HD (High-Density) Shielding Wall Concrete Ontario Hydro, Toronto, Ontario, Canada	Heavyweight Crushed Stone Aggregate
01DA003	GETR, Mix 58-1970 General Electric Test Reactor, Vallecitos, California	Heavyweight Crushed Stone Aggregate
01DA004	SERCOTER Concrete C. E. N., Saclay, France	Heavyweight Crushed Stone Aggregate

Table 3.11 Metallic reinforcements in the Structural Materials Information Center.

Material Code	Common Name	Material Description	Property Code	Property Code Description
02AA001	ASTM A 15, Structural Plain	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated without Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
02AA002	ASTM A 15, Intermediate Plain	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated without Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
02AA003	ASTM A 15, Hard Plain	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated without Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
02AA004	ASTM A 615, Gr. 40, Plain	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated without Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
02AA005	ASTM A 615, Gr. 60, Plain	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated without Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
02AA006	ASTM A 615, Gr. 75, Plain	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated without Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
02AC001	ASTM A 15, Structural Deformed	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated with Deformations	3701	Engineering Stress-Strain (Temperature Dependent)
02AC002	ASTM A 15, Intermediate Deformed	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated with Deformations	3701 3731	Tensile Yield Strength versus Temperature S-N Diagram

Table 3.11 (cont.) Metallic reinforcements in the Structural Materials Information Center.

Material Code	Common Name	Material Description	Property Code	Property Code Description
02AC003	ASTM A 15, Hard Deformed	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated with Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
02AC004	ASTM A 615, Gr. 40, Deformed	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated with Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
			3731	S-N Diagram
02AC005	ASTM A 615, Gr. 60, Deformed	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated with Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
			3702	Engineering Stress-Strain (Temperature Dependent)
			3711	Tensile Yield Strength versus Temperature
			3712	Ultimate Tensile Strength versus Temperature
			3721	Ultimate Tensile Elongation versus Temperature
			3731	S-N Diagram
02AC006	ASTM A 615, Gr. 75, Deformed	Metallic Reinforcement Carbon Steel Reinforcing Bar Uncoated with Deformations	3701	Engineering Stress-Strain (Ambient Conditions)
			3731	S-N Diagram

Table 3.12. Prestressing tendons in the Structural Materials Information Center.

Material Code	Common Name	Material Description	Property Code	Property Code Description
03BA003	ASTM A 421, Type BA, 7.01 mm	Prestressing Tendon Carbon Steel Wire Without Deformations	3702	Engineering Stress-Strain (Temperature Dependent)
			3711	Tensile Yield Strength versus Temperature
			3712	Ultimate Tensile Strength versus Temperature
			3721	Ultimate Tensile Elongation versus Temperature

Table 3.13. Structural steels in the Structural Materials Information Center.

Material Code	Common Name	Material Description	Property Code	Property Code Description
04AA001	ASTM A 7	Structural Steel Carbon Steel Hot-Rolled Steel	3701	Engineering Stress-Strain (Ambient Temperature)
04AA002	ASTM A 36	Structural Steel Carbon Steel Hot-Rolled Steel	3702	Engineering Stress-Strain (Temperature Dependent)
			3711	Tensile Yield Strength versus Temperature
			3712	Ultimate Tensile Strength versus Temperature
			3721	Ultimate Tensile Elongation versus Temperature

Table 3.14. Rubbers in the Structural Materials Information Center.

Material Code	Common Name	Material Description	Property Code	Property Code Description
05AA001	SIS, EPDM, 75 IRHD	Rubber ASTM D 1418, Class M EPDM	3662	Hardness versus Time (Temperature Dependent)

Table 3.15 Organizations with concretes included in the Structural Materials Information Center.

Organization or Research Establishment	Number of Portland Cement Concretes Included in the Structural Materials Information Center		
	Structural Lightweight	Normal-Weight	Heavyweight
Argonne National Laboratory - West Idaho Falls, Idaho	0	3	1
Brookhaven National Laboratory Upton, New York	0	3	0
Canada Centre for Mineral and Energy Technology (CANMET)	0	6	0
C.E.N. Saclay, France	0	0	1
Central Research Institute Electric Power Industry (CRIEPI) Japan	0	1	0
General Electric Test Reactor (GETR) Vallecitos, California	0	1	1
Midland Nuclear Power Plant Midland, Michigan	0	2	0
Ontario Hydro Toronto, Ontario, Canada	0	11	1
Portland Cement Association Skokie, Illinois - Hanford Tanks	0	2	0
Portland Cement Association Skokie, Illinois - M.S. Abrams	1	2	0
Portland Cement Association Skokie, Illinois - Series 356	0	30	0
Portland Cement Association Skokie, Illinois - Series 374	0	13	0
Portland Cement Association Skokie, Illinois - Series 436	0	8	0
Sir Robert McAlpine and Sons, Ltd. London, England	0	2	0
Takenaka Komuten Co., Ltd. Tokyo, Japan	0	2	0
Taywood Engineering Ltd. London, England	0	6	0
Technical University of Braunschweig Germany	0	1	0
University of Birmingham England	0	3	0
University of California Berkeley, California	0	1	0
University of Kyoto Japan	0	9	0
University of Saskatchewan Canada	0	3	0
University of Wisconsin Madison, Wisconsin	0	6	0
Kurt Walz Stuttgart, Germany	0	8	0

Table 3.16 Mix characteristics and baseline properties for the concrete materials contained in the Structural Materials Information Center.

Material Code	Cement		Pozzolan		Coarse Aggregate		Chemical Admixture Type	W/C†	Unit Weight (kg/m**3)	Slump (mm)	Air Content (percent)	Baseline Properties		Performance Curve Property Codes
	Type	Content (kg/m**3)	Type	Content (kg/m**3)	Type	Max Size mm						Comp Strength (MPa)	Test Age (days)	
01BC001	I	263	Not Used	N/A	Expanded Shale	19	ASTM C 260	0.78	1720	Not Reported	5.9	26.9	Not Reported	3672
01CA001	I	308	Not Used	N/A	Dolomite	38	Not Used	0.54	2420	76	Not Reported	19.1	28	3621
01CA002	I	302	Not Used	N/A	Red Granite	38	Not Used	0.51	2389	76	Not Reported	17.3	28	3621
01CA003	V	201	Fly Ash Class C	50	Dolomite	19	ASTM C 260 ASTM C 494, Types A and F	0.60	2283	191	4.8	24.8	28	3612, 3622
01CA004	I	207	Not Used	N/A	Limestone	19	Not Used	0.60	2400	65	2.1	32.9	28	3622
01CA005	I	285	Not Used	N/A	Limestone	19	Not Used	0.45	2420	65	2.0	47.9	28	3622
01CA006	I	396	Not Used	N/A	Limestone	19	Not Used	0.33	2435	40	2.0	57.7	28	3622
01CA007	I	209	Not Used	N/A	Dolostone	19	Not Used	0.60	2430	50	2.0	31.4	28	3622
01CA008	I	287	Not Used	N/A	Dolostone	19	Not Used	0.45	2450	70	1.9	48.1	28	3622
01CA009	I	403	Not Used	N/A	Dolostone	19	Not Used	0.33	2485	40	1.9	57.6	28	3622
01CA010	I	418	Not Used	N/A	Limestone	51	Not Used	0.40	2234	51	1.0	53.6	28	3621, 3631
01CA011	I	297	Not Used	N/A	Limestone	51	Not Used	0.53	2192	56	1.3	40.0	28	3621, 3631
01CA012	I	223	Not Used	N/A	Limestone	51	Not Used	0.71	2160	46	1.5	26.9	28	3621, 3631
01CA013	III	425	Not Used	N/A	Limestone	51	Not Used	0.40	2233	41	0.8	59.3	28	3621, 3631
01CA014	III	292	Not Used	N/A	Limestone	51	Not Used	0.53	2191	51	1.0	46.5	28	3621, 3631
01CA015	III	220	Not Used	N/A	Limestone	51	Not Used	0.71	2162	53	1.3	33.3	28	3621, 3631
01CA016	II	403	Fly Ash Class F	71	Crushed Stone	38	ASTM C 260 ASTM C 494, Type A	0.34	2368	76	4.8	41.4	28	3621
01CA017	II	271	Fly Ash Class F	48	Crushed Stone	38	ASTM C 260 ASTM C 494, Type A	0.47	2320	89	7.4	34.3	28	3621
01CA018	III	265	Not Used	N/A	Crushed Stone	19	Not Used	0.60	2320	Not Reported	Not Reported	37.9	28	3612, 3622

†W/C = water-cement or water-cementitious material ratio.

Table 3.16 (cont.) Mix characteristics and baseline properties for the concrete materials contained in the Structural Materials Information Center.

Material Code	Cement		Pozzolan		Coarse Aggregate		Chemical Admixture Type	W/C†	Unit Weight (kg/m**3)	Slump (mm)	Air Content (percent)	Baseline Properties		Performance Property Codes
	Type	Content (kg/m**3)	Type	Content (kg/m**3)	Type	Max. Size mm						Comp. Strength (MPa)	Test Age (days)	
01CA019	I	201	Fly Ash Class C	50	Crushed Stone	19	ASTM C 260 ASTM C 494, Type A	0.60	2284	Not Reported	Not Reported	21.6	31	3612, 3622
01CA020	II	404	Not Used	N/A	Limestone	19	Not Used	0.43	2455	102	1.4	44.7	28	3602
01CA021	I	445	Not Used	N/A	Limestone	19	Not Used	0.41	2403	13	Not Reported	59.3	28	4631
01CA022	I	447	Not Used	N/A	Limestone	19	Not Used	0.39	2403	19	Not Reported	61.2	28	4631
01CA023	I	410	Not Used	N/A	Limestone	40	ASTM C 494, Type A	0.43	2400	Not Reported	Not Reported	34.1	28	3612, 3621
01CA024	I	435	Not Used	N/A	Crushed Stone	20	ASTM C 494, Type A	0.44	2435	Not Reported	Not Reported	42.5	28	3612, 3621
01CA025	I	270	Fly Ash Class F	90	Crushed Stone	20	ASTM C 494, Type A	0.46	2545	Not Reported	Not Reported	35.0	28	3612, 3621
01CA026	I	420	Not Used	N/A	Crushed Stone	40	ASTM C 494, Type A	0.49	2535	Not Reported	Not Reported	46.2	28	3612, 3621
01CA027	I	340	Fly Ash Class F	115	Crushed Stone	20	ASTM C 494, Type A	0.39	2440	Not Reported	Not Reported	45.0	28	3612, 3621
01CB001	I	301	Not Used	N/A	Gravel	38	ASTM C 260	0.42	2291	102	3.0	37.7	31	3621
01CB002	I	287	Not Used	N/A	Gravel	38	Not Used	0.51	2406	76	Not Reported	19.2	28	3621
01CB003	I	281	Not Used	N/A	Gravel	38	Not Used	0.67	2401	254	Not Reported	12.3	28	3621
01CB004	I	369	Not Used	N/A	Gravel	38	Not Used	0.41	2421	76	Not Reported	25.7	28	3621
01CB005	I	200	Not Used	N/A	Gravel	38	Not Used	0.69	2398	76	Not Reported	10.0	28	3621
01CB006	I	288	Not Used	N/A	Gravel	38	Not Used	0.57	2240	95	2.1	35.1	28	3621
01CB007	I	307	Not Used	N/A	Gravel	38	Not Used	0.51	2288	64	Not Reported	33.6	28	3621
01CB008	I	204	Not Used	N/A	Gravel	30	Not Used	0.78	2360	Not Reported	Not Reported	18.6	28	3621
01CB009	I	201	Not Used	N/A	Gravel	30	Not Used	1.02	2330	Not Reported	Not Reported	13.1	28	3621
01CB010	I	197	Not Used	N/A	Gravel	30	Not Used	1.02	2230	Not Reported	Not Reported	9.0	28	3621
01CB011	I	195	Not Used	N/A	Gravel	30	Not Used	1.29	2220	Not Reported	Not Reported	5.4	28	3621

†W/C = water-cement or water-cementitious material ratio.

Table 3.16 (cont.) Mix characteristics and baseline properties for the concrete materials contained in the Structural Materials Information Center.

Material Code	Cement		Pozzolan		Coarse Aggregate		Chemical Admixture Type	W/C†	Unit Weight (kg/m**3)	Slump (mm)	Air Content (percent)	Baseline Properties		Performance Curve Property Codes
	Type	Content (kg/m**3)	Type	Content (kg/m**3)	Type	Max. Size mm						Comp. Strength (MPa)	Test Age (days)	
01CB012	I	303	Not Used	N/A	Gravel	30	Not Used	0.50	2380	Not Reported	Not Reported	36.5	28	3621
01CB013	I	302	Not Used	N/A	Gravel	30	Not Used	0.66	2341	Not Reported	Not Reported	28.9	28	3621
01CB014	I	302	Not Used	N/A	Gravel	30	Not Used	0.64	2260	Not Reported	Not Reported	20.0	28	3621
01CB015	I	300	Not Used	N/A	Gravel	30	Not Used	0.82	2260	Not Reported	Not Reported	15.5	28	3621
01CB016	I	320	Not Used	N/A	Gravel	25	Not Used	0.58	2335	190	Not Reported	Not Reported	N/A	3651
01CB017	I	288	Fly Ash	32	Gravel	25	Not Used	0.55	2378	190	Not Reported	Not Reported	N/A	3651
01CB018	I	256	Fly Ash	64	Gravel	25	Not Used	0.54	2375	190	Not Reported	Not Reported	N/A	3651
01CB019	I	224	Fly Ash	96	Gravel	25	Not Used	0.53	2365	190	Not Reported	Not Reported	N/A	3651
01CB020	I	320	Not Used	N/A	Gravel	25	ASTM D 98	0.58	2335	190	Not Reported	Not Reported	N/A	3651
01CB021	I	320	Not Used	N/A	Gravel	25	ASTM D 98	0.58	2335	190	Not Reported	Not Reported	N/A	3651
01CB022	I	320	Not Used	N/A	Gravel	25	ASTM C494, Type A	0.49	2342	190	Not Reported	Not Reported	N/A	3651
01CB023	I	320	Not Used	N/A	Gravel	25	ASTM D 98 ASM C 494, Type A	0.49	2342	190	Not Reported	Not Reported	N/A	3651
01CB024	I	320	Not Used	N/A	Gravel	25	ASTM D 98 ASM C 494, Type A	0.49	2342	190	Not Reported	Not Reported	N/A	3651
01CB025	I	281	Not Used	N/A	Gravel	38	Not Used	0.65	2459	89	1.0	27.1	28	3621
01CB026	I	311	Not Used	N/A	Gravel	38	Not Used	0.61	2451	76	1.1	30.8	28	3621
01CB027	I	363	Not Used	N/A	Gravel	38	Not Used	0.52	2459	83	1.0	33.2	28	3621
01CB028	I	281	Not Used	N/A	Gravel	38	ASTM C 260 ASM C 494, Type A	0.55	2419	102	3.9	25.0	28	3621
01CB029	I	311	Not Used	N/A	Gravel	38	ASTM C 260 ASM C 494, Type A	0.50	2435	102	3.5	34.4	28	3621

†W/C = water-cement or water-cementitious material ratio.

Table 3.16 (cont.) Mix characteristics and baseline properties for the concrete materials contained in the Structural Materials Information Center.

Material Code	Cement		Pozzolan		Coarse Aggregate		Chemical Admixture Type	W/C†	Unit Weight (kg/m**3)	Slump (mm)	Air Content (percent)	Baseline Properties		Performance Curve Property Codes
	Type	Content (kg/m**3)	Type	Content (kg/m**3)	Type	Max Size mm						Comp Strength (MPa)	Test Age (days)	
01CB030	I	363	Not Used	N/A	Gravel	38	ASTM C 260 ASM C 494, Type A	0.45	2403	102	5.3	39.4	28	3621
01CB031	I	196	Fly Ash Class F	84	Gravel	38	Not Used	0.56	2451	51	1.3	29.5	28	3621
01CB032	I	218	Fly Ash Class F	93	Gravel	38	Not Used	0.56	2451	45	1.3	28.5	28	3621
01CB033	I	255	Fly Ash Class F	109	Gravel	38	Not Used	0.50	2451	57	1.3	35.1	28	3621
01CB034	I	198	Not Used	N/A	Gravel	38	Not Used	0.98	2405	Not Reported	Not Reported	14.4	28	3621
01CB035	I	274	Not Used	N/A	Gravel	38	Not Used	0.70	2470	Not Reported	Not Reported	25.1	28	3621
01CB036	I	367	Not Used	N/A	Gravel	51	Not Used	0.40	2410	61	0.7	59.7	28	3613, 3621, 3631
01CB037	I	269	Not Used	N/A	Gravel	51	Not Used	0.53	2397	58	1.1	43.2	28	3613, 3621, 3631
01CB038	I	210	Not Used	N/A	Gravel	51	Not Used	0.71	2377	61	1.2	27.6	28	3613, 3621, 3631
01CB039	I	371	Not Used	N/A	Gravel	51	Not Used	0.40	2407	28	1.0	52.9	28	3613, 3621, 3631
01CB040	I	274	Not Used	N/A	Gravel	51	Not Used	0.53	2396	53	1.0	41.5	28	3613, 3621, 3631
01CB041	I	210	Not Used	N/A	Gravel	51	Not Used	0.71	2377	64	1.3	29.0	28	3613, 3621, 3631
01CB042	I	364	Not Used	N/A	Gravel	51	Not Used	0.40	2415	38	0.7	56.5	28	3613, 3621, 3631
01CB043	I	274	Not Used	N/A	Gravel	51	Not Used	0.53	2397	53	1.0	44.6	28	3613, 3621, 3631
01CB044	I	210	Not Used	N/A	Gravel	51	Not Used	0.71	2377	51	1.3	28.5	28	3613, 3621, 3631
01CB045	I	365	Not Used	N/A	Gravel	51	Not Used	0.40	2411	61	0.9	56.4	28	3613, 3621, 3631
01CB046	I	271	Not Used	N/A	Gravel	51	Not Used	0.53	2397	71	1.2	44.4	28	3613, 3621, 3631
01CB047	I	210	Not Used	N/A	Gravel	51	Not Used	0.71	2377	81	1.4	27.3	28	3613, 3621, 3631
01CB048	I	379	Not Used	N/A	Gravel	51	Not Used	0.40	2384	71	0.9	51.5	28	3621, 3631
01CB049	I	267	Not Used	N/A	Gravel	51	Not Used	0.53	2367	66	1.2	41.5	28	3621, 3631

†W/C = water-cement or water-cementitious material ratio.

Table 3.16 (cont.) Mix characteristics and baseline properties for the concrete materials contained in the Structural Materials Information Center.

Material Code	Cement		Pozzolan		Coarse Aggregate		Chemical Admixture Type	W/C†	Unit Weight (kg/m**3)	Slump (mm)	Air Content (percent)	Baseline Properties		Performance Curve Property Codes
	Type	Content (kg/m**3)	Type	Content (kg/m**3)	Type	Max Size mm						Comp Strength (MPa)	Test Age (days)	
01CB050	I	201	Not Used	N/A	Gravel	51	Not Used	0.71	2347	64	1.7	27.4	28	3621, 3631
01CB051	III	381	Not Used	N/A	Gravel	51	Not Used	0.40	2406	71	0.4	62.2	28	3613, 3621, 3631
01CB052	III	270	Not Used	N/A	Gravel	51	Not Used	0.53	2399	74	0.7	51.0	28	3613, 3621, 3631
01CB053	III	206	Not Used	N/A	Gravel	51	Not Used	0.71	2382	61	1.1	34.2	28	3613, 3621, 3631
01CB054	III	387	Not Used	N/A	Gravel	51	Not Used	0.40	2398	69	0.5	58.7	28	3613, 3621, 3631
01CB055	III	276	Not Used	N/A	Gravel	51	Not Used	0.53	2392	86	0.7	46.6	28	3613, 3621, 3631
01CB056	III	206	Not Used	N/A	Gravel	51	Not Used	0.71	2380	86	1.2	32.8	28	3613, 3621, 3631
01CB063	III	393	Not Used	N/A	Gravel	51	Not Used	0.40	2382	84	0.5	56.3	28	3621, 3631
01CB064	III	266	Not Used	N/A	Gravel	51	Not Used	0.53	2366	66	1.1	46.8	28	3621, 3631
01CB065	III	195	Not Used	N/A	Gravel	51	Not Used	0.71	2351	51	1.5	32.5	28	3621, 3631
01CB068	IA	332	Not Used	N/A	Gravel	38	Not Used	0.40	2310	61	6.7	32.8	28	3613, 3621, 3631
01CB069	IA	250	Not Used	N/A	Gravel	38	Not Used	0.47	2295	58	7.6	26.5	28	3613, 3621, 3631
01CB093	I	336	Not Used	N/A	Gravel	38	Not Used	0.45	2419	74	1.3	39.4	28	3613, 3621, 3631
01CB094	I	251	Not Used	N/A	Gravel	38	Not Used	0.60	2401	66	1.6	26.3	28	3613, 3621, 3631
01CB097	II	335	Not Used	N/A	Gravel	38	Not Used	0.43	2435	66	1.1	41.4	28	3613, 3621, 3631
01CB098	II	253	Not Used	N/A	Gravel	38	Not Used	0.56	2426	64	1.3	29.8	28	3613, 3621, 3631
01CB099	IIA	334	Not Used	N/A	Gravel	38	Not Used	0.39	2389	51	3.9	33.6	28	3613, 3621, 3631
01CB100	IIA	251	Not Used	N/A	Gravel	38	Not Used	0.49	2360	53	4.9	27.7	28	3613, 3621, 3631
01CB110	III	335	Not Used	N/A	Gravel	38	Not Used	0.49	2400	74	1.2	45.2	28	3613, 3621, 3631
01CB111	III	253	Not Used	N/A	Gravel	38	Not Used	0.60	2407	76	1.2	37.5	28	3613, 3621, 3631
01CB124	IV	169	Not Used	N/A	Gravel	38	Vinsol Resin	0.78	2349	53	4.3	8.8	28	3613, 3621, 3631

†W/C = water-cement or water-cementitious material ratio.

Table 3.16 (cont.) Mix characteristics and baseline properties for the concrete materials contained in the Structural Materials Information Center.

Material Code	Cement		Pozzolan		Coarse Aggregate		Chemical Admixture Type	W/C†	Unit Weight (kg/m**3)	Slump (mm)	Air Content (percent)	Baseline Properties		Performance Curve Property Codes
	Type	Content (kg/m**3)	Type	Content (kg/m**3)	Type	Max Size mm						Comp Strength (MPa)	Test Age (days)	
01CB125	V	335	Not Used	N/A	Gravel	38	Not Used	0.41	2451	53	1.3	42.7	28	3613, 3621, 3631
01CB126	V	251	Not Used	N/A	Gravel	38	Not Used	0.54	2430	61	1.5	26.8	28	3613, 3621, 3631
01CB127	I	439	Not Used	N/A	Gravel	38	Not Used	0.35	2432	66	1.0	49.1	28	3621, 3631
01CB128	I	269	Not Used	N/A	Gravel	38	Not Used	0.49	2439	56	1.4	39.6	28	3621, 3631
01CB129	I	221	Not Used	N/A	Gravel	38	Not Used	0.62	2421	64	1.4	25.0	28	3621, 3631
01CB130	I	317	Not Used	N/A	Gravel	38	Not Used	0.42	2446	69	1.1	46.0	28	3621, 3631
01CB131	IS	498	Not Used	N/A	Gravel	38	Not Used	0.35	2386	61	1.2	51.4	28	3621, 3631
01CB132	IS	284	Not Used	N/A	Gravel	38	Not Used	0.49	2425	71	1.2	44.8	28	3621, 3631
01CB133	IS	228	Not Used	N/A	Gravel	38	Not Used	0.62	2411	71	1.3	29.5	28	3621, 3631
01CB134	IS	367	Not Used	N/A	Gravel	38	Not Used	0.41	2418	71	1.2	52.6	28	3621, 3631
01CB151	I	323	Not Used	N/A	Gravel	32	Not Used	0.55	2440	102	Not Reported	26.2	28	3621
01CB153	I	387	Not Used	N/A	Gravel	Not Reported	Not Used	0.47	2323	89	Not Reported	29.8	28	3614, 3622
01CB154	V	388	Not Used	N/A	Gravel	Not Reported	Not Used	0.46	2323	89	Not Reported	39.9	28	3614, 3622
01CB155	I	343	Not Used	N/A	Gravel	40	ASTM C 494, Type A	0.40	2399	50	2.5	43.6	28	3612, 3614, 3622 3642, 4634, 4636
01CB156	II	298	Not Used	N/A	Gravel	Not Reported	ASTM C 260	0.49	2438	71	3.4	34.2	30	3612, 3614, 3617 3619, 3622, 3635
01CB157	II	391	Not Used	N/A	Gravel	Not Reported	ASTM C 260	0.41	2412	86	3.8	40.7	30	3612, 3614, 3617 3619, 3622, 3635
01CB158	I	508	Not Used	N/A	Ottawa Sand	N/A	Not Used	0.49	2150	Not Reported	Not Reported	39.1	28	3621, 4637
01CB159	V	508	Not Used	N/A	Ottawa Sand	N/A	Not Used	0.49	2150	Not Reported	Not Reported	40.3	28	3621, 4637
01CB160	V	437	Silica Fume	77	Ottawa Sand	N/A	ASTM C 494, Type F	0.40	2150	Not Reported	Not Reported	59.6	28	3621, 4637
01CB161	I	236	Not Used	N/A	Gravel	19	ASTM C 260	0.55	2310	Not Reported	6.0	26.9	Not Reported	3672
01CB162	I	249	Not Used	N/A	Gravel	19	ASTM C 260	0.51	2301	Not Reported	5.0	26.9	Not Reported	3672

†W/C = water-cement or water-cementitious material ratio.

Table 3.16 (cont.) Mix characteristics and baseline properties for the concrete materials contained in the Structural Materials Information Center.

Material Code	Cement		Pozzolan		Coarse Aggregate		Chemical Admixture Type	W/C†	Unit Weight (kg/m**3)	Slump (mm)	Air Content (percent)	Baseline Properties		Performance Curve Property Codes
	Type	Content (kg/m**3)	Type	Content (kg/m**3)	Type	Max. Size mm						Comp. Strength (MPa)	Test Age (days)	
01CB163	I	441	Not Used	N/A	Gravel	19	Not Used	0.37	2403	19	Not Reported	66.0	28	4631
01CB164	I	360	Not Used	N/A	Gravel	16	Not Used	0.49	2390	Not Reported	Not Reported	40.9	28	3652
01CB165	I	330	Not Used	N/A	Gravel	Not Reported	Not Used	0.50	2415	58	1.7	37.1	28	3672, 3674, 3676 3678, 4639
01CB166	I	390	Not Used	N/A	Gravel	Not Reported	Not Used	0.50	2391	206	1.2	42.5	28	3672, 3674, 3676 3678, 4639
01CB167	I	215	Fly Ash Class F	140	Gravel	20	ASTM C 494, Type A	0.39	2385	Not Reported	Not Reported	45.8	28	3612, 3621
01DA001	I	301	Not Used	N/A	Crushed Stone	38	Not Used	0.42	3489	76	1.3	38.4	28	3621
01DA002	IV	348	Not Used	N/A	Crushed Stone	20	ASTM C 494, Type A	0.49	3740	75	Not Reported	34.5	28	3621
01DA003	I	307	Not Used	N/A	Crushed Stone	38	ASTM C 494, Type A	0.55	3732	102	Not Reported	17.9	28	3621
01DA004	Aluminous	500	Not Used	N/A	Crushed Stone	Not Reported	Not Used	0.38	2740	Not Reported	Not Reported	36.0	28	3613, 3621 3631, 4631

†W/C = water-cement or water-cementitious material ratio.

Table 3.17 Environment- and time-dependent concrete properties in the Structural Materials Information Center.

a. Environment-Dependent Properties

Property Code	Property Description	Type of Exposure	Number of Concretes
3602	Engineering Stress-Strain Diagram	Elevated Temperatures	1
3642	Creep of Concrete in Compression	Elevated Temperatures	1
3651	Bond Stress vs Slip	Ambient Conditions	9
3652	Bond Stress vs Slip	Elevated Temperatures	1
3672	Compressive Strength Ratio vs Temperature	Elevated Temperatures	5
3674	Modulus of Elasticity Ratio vs Temperature	Elevated Temperatures	2
3676	Tensile Strength Ratio vs Temperature	Elevated Temperatures	2
3678	Bond Strength Ratio vs Temperature	Elevated Temperatures	2
4639	Weight Change vs Temperature	Elevated Temperatures	2

Four-digit Property Codes are used to identify specific performance curves. Ambient conditions include natural temperature fluctuations and exposure conditions associated with outdoor environments or exposure conditions associated with indoor or laboratory environments. Elevated temperature conditions include exposure to temperatures ranging from below to above room temperature.

Table 3.17 (cont.) Environment- and time-dependent concrete properties in the Structural Materials Information Center.

b. Time-Dependent Properties

Property Code	Property Description	Type of Exposure	Number of Concretes
3612	Modulus of Elasticity vs Time	Elevated Temperatures	12
3613	Dynamic Modulus of Elasticity vs Time	Ambient Conditions	32
3614	Dynamic Modulus of Elasticity vs Time	Elevated Temperatures	6
3617	Poisson's Ratio vs Time	Elevated Temperatures	2
3619	Dynamic Poisson's Ratio vs Time	Elevated Temperatures	2
3621	Ultimate Compressive Strength vs Time	Ambient Conditions	95
3622	Ultimate Compressive Strength vs Time	Elevated Temperatures	15
3631	Ultimate Flexural Strength vs Time	Ambient Conditions	52
3635	Splitting Tensile Strength vs Time	Elevated Temperatures	2
4631	Weight Change vs Time	Ambient Conditions	4
4634	Porosity vs Time	Elevated Temperatures	1
4636	Shrinkage vs Time	Elevated Temperatures	1
4637	Length Change vs Time	Ambient Conditions	3

Four-digit Property Codes are used to identify specific performance curves. Ambient conditions include natural temperature fluctuations and exposure conditions associated with outdoor environments or exposure conditions associated with indoor or laboratory environments. Elevated temperature conditions include exposure to temperatures ranging from below to above room temperature.

4. TASK S.3 – STRUCTURAL COMPONENT ASSESSMENT/REPAIR TECHNOLOGY

4.1 OBJECTIVE

The overall objectives of this task are to (1) develop a systematic methodology that can be used to make a quantitative assessment of the presence, magnitude, and significance of any environmental stressors or aging factors that could impact the durability of safety-related concrete components in nuclear power plants (NPPs), and (2) provide recommended in-service inspection 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 in meeting the objectives of this task include the 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. Results of this task will have a major impact on NPP continued service considerations (see Chapter 5 – Task S.4) since residual life determinations cannot be made unless all degradation mechanisms are evaluated and environmental factors quantified and each structure is examined and evaluated in detail. Figure 4.1 presents the work breakdown structure for the structural component assessment/repair technology task.

4.2 BACKGROUND

4.2.1 Evaluation of Concrete Material Systems

Since the ability of a concrete component to meet its functional and performance requirements over an extended period of time is dependent on the durability of its constituents, techniques for the detection of concrete component degradation should address evaluation of the concrete, mild steel reinforcing, prestressing system, and anchorage embedments.

Concrete cracking, voids, and delaminations can be detected by visual inspections, nondestructive testing (ultrasonic and stress wave, acoustic impact, radiography, penetrating radar, thermal mapping), and core examination. In-situ concrete strength determinations are through either direct (core tests) or indirect techniques (surface hardness, rebound, penetration, pull-out resistance, break-off resistance, and ultrasonic pulse velocity). The primary distress to which mild steel reinforcement could be subjected would be corrosive attack. Techniques available for corrosion monitoring and inspection of steel in concrete include visual, mechanical and ultrasonic tests, core sampling in conjunction with chemical and physical tests, potential and thermal mapping, and rate of corrosion probes. The condition and functional capability of unbonded post-tensioning systems in nuclear power plants is assessed through selection of a random, representative sample of tendons, examination of the anchorage assembly hardware of the selected tendons, determination of the stress level in each sample tendon, examination of previously stressed wires or strands from one tendon of each type in the structure, and an analysis of a grease sample from each tendon in the surveillance. The present basis for conducting tendon inspections is presented in RG 1.35, "Inservice Inspections of Ungrouted Tendons in Prestressed Concrete Containments (Rev. 3)," and companion RG 1.35.1, "Determining Prestressing Forces for Inspection of Prestressed Concrete Containments." Failure of an

embedment will generally occur as a result of either improper installation or deterioration of the concrete within which it is embedded. A combination of visual examinations and mechanical tests is used to evaluate the general condition of an embedment.

Table 4.1 (Ref. 4.1) summarizes primary and secondary nondestructive evaluation techniques for inspection of concrete components. More detailed information on many of these techniques can be obtained from Ref. 4.2. Quantitative interpretation of the results obtained from many of the nondestructive evaluation methods can be difficult, however, due to the requirement for correlation curves. Also, many of the methods only make surface determinations of concrete properties that can be quite different from internal properties, particularly where a component may be several meters thick. In addition, none of the techniques provide rate effect data that can be used for continued service considerations or residual life predictions.

4.2.2 Remedial Measures

Objectives of remedial work are to restore the component's structural integrity, arrest the mechanism producing distress, and ensure, as far as possible, that the cause of distress will not reoccur. Basic components of a program to meet these objectives 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 (Ref. 4.3).

Typical types of distress that occur in light-water reactor (LWR) concrete facilities include cracking, spalling or delamination, nonvisible voids, and fracturing or shattering. Although a wide variety of materials are available for the repair or maintenance of concrete exhibiting distress, they generally include one or more of the following materials: epoxy resins, shotcrete, preplaced aggregate concrete, epoxy ceramic foams, replacement mortar or concrete, wedge anchors and additional reinforcement, and miscellaneous sealant materials (Ref. 4.4). Selection of the technique for repair of a concrete structure depends to a large degree on the size, depth, and area of repair required. Existing elements can also become inadequate due to either a change in performance requirements or occurrence of an overload condition. Under these conditions, retrofitting may be required to reestablish serviceability. Retrofitting can be accomplished by either strengthening of existing elements, replacement, addition of new force-resisting elements, a combination of element strengthening and addition, or use of supplemental connecting devices (Ref. 4.4). Reference 4.3 notes that a satisfactory repair meeting requirements for strength, durability, appearance, and economy can be made if the cause of the distress is eliminated, the area is prepared by removal of degraded materials, and the proper repair technique selected and correctly implemented.

4.3 PLAN OF ACTION

Assessment of the ability of concrete components to meet their functional and performance characteristics is an important consideration in continuing the service life of nuclear facilities. Given the complex nature of the various environmental stressors and aging factors that potentially can exert deteriorating influences on the concrete components, a systems approach is probably best in addressing an evaluation of a structure for continued service. Basic components of such an approach would encompass the development of (1) a classification scheme for structures, elements, and deterioration causes and effects; (2) a methodology for conducting a quantitative assessment of the presence of active deteriorating influences; and (3) remedial measure considerations to reestablish the capability of degraded structures or components to meet potential future requirements, such as a loss-of-coolant accident (LOCA). The Level 3 work breakdown structure for this task is presented in Fig. 4.2.

4.3.1 LWR Critical Concrete Component Classification

Basic activities under this subtask include the identification of safety-related concrete structural components in LWR plants as well as aging factors and environmental stressors that can impact the ability of these components to meet current and future functional and performance requirements. An aging assessment methodology has been developed for use in identifying structural components of most importance to aging and the degradation factors that can potentially impact the performance of these components (i.e., focus in-service inspection activities). Critical components are described and a listing of their functional and performance requirements provided. Each component is classified in terms of its relative importance to safety. Subelements of primary structures are identified and their importance to the overall integrity of the primary structure assessed. The expected type and range of degradation factors (aging, environmental stressors, operation, testing, etc.) that can impact each subelement are identified and a rating system used to indicate the significance of each degradation mechanism. Critical performance characteristics and properties that can serve as indicators of the severity of each type of degradation are defined. Structural performance characteristics and degradation factor significance are combined through use of a ranking (or rating) scheme.

4.3.2 NDE/Sampling Inspection Technology

4.3.2.1 In-Service Inspection Techniques

Detection of age- or environmental stressor-related degradation, as well as its magnitude and rate of occurrence, is a key factor in maintaining the readiness of safety-related concrete components to continue their functions in the unlikely event that a condition, such as a LOCA, would occur. In order to simplify continued service evaluations of nuclear safety-related concrete components, it would be advantageous to have a standardized in-service inspection program that would provide data for use in identifying and quantifying deteriorating influences as well as providing rate of change information on these influences (e.g., degradation versus time relationships such as shown in Fig. 4.3).

Basic activities under this subtask are related to evaluation of nondestructive and sampling procedures that are available for performing in-service inspections of the critical concrete components (see Sect. 4.3.1). Also, recommended criteria will be provided for use in inspection of these components to assess their current condition as well as to develop trending information for use in continued service assessments. Existing destructive and nondestructive testing and inspection methods have been reviewed. These methods were evaluated to identify capabilities, accuracies, and limitations in the detection/assessment of aging factors and environmental stressors to which the critical concrete components in NPPs may be subjected. Included in the review were potential new techniques that, although they may not be widely used at present, may possess significant potential (e.g., modal analysis/dynamic testing). Methods that can be utilized to evaluate the severity of each of the various degradation factors were identified and rated according to their effectiveness. Included in the rating were any uncertainties associated with each method as well as its effectiveness in performing volumetric examinations. Included as a part of this review was an activity to provide correlation curves and other statistical data for the most common techniques used to assess in-situ concrete compressive strength.

4.3.2.2 Structural Integrity Assessments

Once the critical components, degradation factors that can act on these components, and techniques that can be utilized to evaluate the components have been identified, a methodology for conducting a structural assessment of the components needs to be

developed as well as criteria for its application. A schematic of such a concrete component evaluation methodology is presented in Fig. 4.4. When developed and implemented, the concrete component evaluation methodology will provide the required data and procedures for performing a safety condition (reliability) assessment of the critical safety-related concrete components in NPPs.

Basic activities under this subtask are related to providing recommended criteria for inspection of critical concrete components to assess their current structural reliability as well as to develop data for use in assessments of future performance (see Chapter 5). Available methodologies (i.e., the application of inspection/evaluation techniques) that have been or are presently being utilized to perform condition assessments of concrete components/structural members have been reviewed. Included in the review were techniques that have been used in NPPs, what these techniques have revealed in the form of degradation, an assessment of their effectiveness, and if any of the techniques have been utilized for residual life determinations. Data requirements and recommended inspection intervals for use in developing the required data for use in continued service evaluations also will be established (e.g., implementation of predictive monitoring techniques to better evaluate component performance). A methodology will be developed that can be utilized for performing an assessment of the structural reliability of safety-related concrete components in NPPs as well as developing the information required to trend concrete component performance for use in continued service evaluations. Periodic inspections performed in compliance with this methodology will help identify any potential problems before they become safety-significant and allow corrective actions to be taken at an early stage.

4.3.3 Remedial/Preventive Measures Considerations

The life of reinforced concrete components in NPPs is expected to be greater than any likely period for which the plant would operate (original design lifetime plus continued service lifetime plus decommissioning period), provided neither environmental factors, applied load, nor a combination of load and environmental factors compromise the structural integrity (Ref. 4.5). In fact, when concrete structures have been fabricated with close attention to the details related to the production of good concrete [material selection, production control, desirable properties, economy (Ref. 4.6)] the concrete should exhibit extended durability; however, when there has been a breakdown in any of these details or if the component has been subjected to an extreme environmental stressor or adverse aging factor, distress can occur.

Results of a literature review (Ref. 4.1), in which examples were identified where the pre- and post-repair performance of structural members were compared, indicates that remedial measures are capable of completely restoring a component's structural integrity. The effectiveness of the repair techniques in these studies was the direct consequence of careful consideration given to the selection and application of the particular repair technique under controlled (laboratory) conditions. Successful application of these techniques to concrete structures in NPPs requires development of criteria for their application as well as an assessment of their overall effectiveness under representative (field) conditions.

Basic activities under this subtask are related to an assessment of repair procedures for concrete material/structural systems and establishment of criteria for their utilization. Techniques that are available for the repair, replacement, or retrofitting of degraded structural subelements will be reviewed and an assessment performed of their effectiveness. Methods available for evaluating the performance of repair materials as well as any potential impact of a repair on the inspection procedures will be addressed. Techniques that can be used to mitigate the effects of environmental stressors or aging factors will be identified. Recommended preventative measure procedures that can be used to effectively offset, counteract, or minimize any minor deterioration effects to prevent them from becoming significant will be established.

4.4 SUMMARY OF ACCOMPLISHMENTS

4.4.1 LWR Critical Concrete Component Classification (Subtask S.3.1)*

The methodology developed provides a logical basis for identifying the critical concrete structural elements in a nuclear plant as well as the degradation factors that can potentially impact their performance. Because of the variability in likelihood of occurrence of degradation of concrete structures in U.S. light-water reactor plants due to design differences, material utilization, geographical location, etc., the degradation factor grading system established is stated in terms of a possible range of values. The resulting degradation factor grading values (between 1 and 10, with 10 being highest) for an individually evaluated subelement are combined into a single degradation factor significance value by summing the degradation factor grading values and dividing by the number of degradation factors, for instance,

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

where

DFS = degradation factor significance value, rounded to nearest integer,
DFG = degradation factor grading value, and
n = number of degradation factors, up to a total of three.

Determination of the relative ranking of Category I structures is based on the weighted contributions of the (1) structural importance of subelements, (2) their safety significance, (3) environmental exposure, and (4) degradation factor significance 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 [Eq. (4.1)], and
w_1, w_2, w_3 = weighting factors having recommended values of 4, 9, and 7, respectively.

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

$$CR = \sum_{i=1}^{N} SR_i / N, \tag{4.3}$$

where

CR = cumulative rank,
SR = subelement rank, and

* Activities completed during a previous reporting period (Ref. 4.7). A summary of results is provided for information purposes.

N = number of subelements for the particular primary structure.

The methodology has been applied to a pressurized-water reactor (PWR) with large dry metal containment, a boiling-water reactor (BWR) with reinforced concrete Mark II containment, and a PWR with large dry prestressed concrete containment. The highest ranking primary concrete structure for each of these plants was found to be the shield building, containment building, and containment building, respectively.

4.4.2 NDE/Sampling Inspection Technology (Subtask S.3.2)

This subtask evaluates nondestructive and sampling/analysis procedures that are available for performing in-service inspections of the critical concrete components in nuclear power plants. Also, recommended criteria will be provided for use in inspection of these components to assess their current condition as well as to develop trending information for use in continued service assessments. Primary activities under this subtask were conducted through subcontracts with Construction Technology Laboratories, Inc. (Skokie, Illinois), the National Institute of Standards and Technology (Gaithersburg, Maryland), Dr. H. Hill (Novato, California), and Mr. C. Hookham (Ann Arbor, Michigan).

A review and assessment of in-service inspection techniques and methodologies for application to concrete structures in nuclear power plants has been completed.* Direct and indirect techniques that can be used to detect degradation of concrete materials and structures have been reviewed (Ref. 4.8). Capabilities, accuracies, and limitations of available nondestructive evaluation techniques were assessed (i.e., audio, electrical, impulse radar, infrared thermography, magnetic, stress wave reflection/refraction, modal analysis, radioactive/nuclear, rebound hammer, and ultrasonic). Information also was assembled on destructive (i.e., air permeability, break-off, chemical, coring, probe penetration, and pull-out) and emerging (i.e., leakage flux, nuclear magnetic resonance, capacitance-based, polarization resistance, ultraviolet radiation, and half-cell potential using impulse radar) techniques. Tables 4.2 and 4.3 summarize the information that was assembled for nondestructive and destructive testing techniques, respectively. Recommendations were provided on testing methods to identify and assess damage resulting from typical factors that can degrade reinforced concrete structures (e.g., see Table 4.1).

One of the conclusions of the previous study was that many of the nondestructive testing methods for concrete structures require correlation curves. Typically a small number of destructive and nondestructive tests are conducted in tandem at noncritical locations in a structure to develop a regression relation between the two tests. If destructive tests are not permitted in the structure and laboratory tests can not be performed using specimens fabricated from the same concrete mix design, the assessment of in-place strength must be based on published results. Correlation curves and other statistical data were developed for selected nondestructive testing techniques by applying monovariant linear regression analyses (Ref. 4.9) to data obtained from publications on selected nondestructive testing techniques (i.e., break-off, pull-out, rebound hammer, ultrasonic pulse velocity, and probe penetration) (Ref. 4.10). These techniques were selected since they comprise an overwhelming majority of the nondestructive tests performed. For each of the nondestructive techniques investigated, the data identified were subdivided by coarse aggregate type and coarse aggregate content (by weight). This subdivision was based on results provided in the literature indicating that the techniques are influenced by aggregate characteristics (e.g., the pullout and break-off tests are dependent on the aggregate type and maximum aggregate size, and the probe penetration and rebound hammer results are

* These activities were completed during a previous reporting period. A summary of results is provided for information purposes.

influenced by the aggregate hardness). Unfortunately, insufficient data were available to further subdivide the data by maximum coarse aggregate size. Since all of the data used in the study were not the result of careful experimentation, a quality rating system was developed based on the 11 criteria used to evaluate data quality for the Structural Materials Information Center (see Table 3.5). The data source rating system developed includes nine criteria, e.g., completeness of material description, type of input, completeness of data, completeness of resources, quality of resources, consistency of results, precision and scatter, uncertainty and bias, and statistical methods used. Each data source used was given a rating from A to D (A being highest), based on these criteria. The regression analyses accounted for errors in both the nondestructive and compressive strength data and their constant coefficient of variation. The methods developed can be used for point estimates and estimates of the true mean. Variance of point estimates yields information about the distribution of the strength population that is required for calculating the characteristic strength needed to assess structural integrity. Figure 4.5 presents an example of the relationship between concrete cylinder compressive strength and rebound number that was obtained from gravel concretes having water-to-cement ratios from 0.37 to 0.56 (by weight) and coarse aggregate contents from 0.45 to 0.49 (by mass).

The ability of a prestressed concrete nuclear power plant containment to withstand the loadings that would develop as a result of a loss-of-coolant accident depends on the continued integrity of the prestressing tendons. In the U.S., the condition and functional capability of unbonded post-tensioning systems must be periodically assessed. This is accomplished, in part, through an in-service inspection program that must be developed and implemented for each containment. Requirements for containment tendon surveillance programs are provided in documents such as Regulatory Guide 1.35, Regulatory Guide 1.35.1, ASME Section XI Subsection IWL, and the U.S. Standard Technical Specification for Tendon Surveillance. Although the overall performance of the post-tensioning systems has been very good, there have been several instances of degradation. Examples include voids under tendon bearing plates resulting from improper concrete placement, cracking of anchor heads due to stress-corrosion cracking or embrittlement, containment dome delaminations due to low quality coarse aggregate material and absence of radial reinforcement or unbalanced prestressing forces, and low prestressing forces. A report is being prepared by Dr. Hill in which potential structural issues related to aging of post-tensioning systems in nuclear power plant containments are discussed. An overview of current requirements associated with in-service inspection of the post-tensioning systems is providing the basis for development of a life management program for these systems. Potential aging- and environmental-stressor related items that can impact the performance of these systems are being identified (e.g., corrosion; loss of prestressing force due to relaxation, concrete creep, concrete shrinkage; etc.). The effectiveness of current life management programs in identifying these incidences is being assessed and commentary prepared on how post-tensioning system-aging issues have been addressed or could be addressed in the future as these systems may reach a critical stage for a decision on plant license renewal. For example, can post-tensioning tendons with prestressing force levels approaching the lower bound of acceptable performance merely be retensioned? What are the long-term effects on the mechanical performance of the post-tensioning system of being under load? That is, does the tendon ultimate tensile strength and elongation capacity decrease with age under load? Results developed under this activity are being correlated with those being conducted to develop damage assessment criteria for reinforced concrete structures (see next paragraph) and reliability-based methodologies to indicate current condition and predict future performance (Chapter 5).

In-service inspection requirements are imposed on nuclear plants through documents such as the following: 10CFR50, NRC Regulatory Guides, Plant Technical Specifications, Inspection and Enforcement Bulletins, NRC letters, and the American Society of Mechanical Engineers *Boiler and Pressure Vessel Code* (Ref. 4.11).

However, because each nuclear plant has its unique construction permit issue and operating license issue dates, each plant could potentially have a different set of minimum in-service inspection requirements. Therefore, to simplify continued service evaluations, it would be advantageous to have a standardized in-service inspection program that could be used to identify and also to quantify any deteriorating influences. The availability of such a standardized inspection program would also help ensure that a uniform set of evaluation criteria are applied to each plant. Limited information on criteria, inspection, and testing requirements for development of such a procedure are available in the form of documents published by the American Concrete Institute (e.g., *Guide for Making a Condition Survey of Concrete in Service* (Ref. 4.12), *Strength Evaluation of Existing Concrete in Service* (Ref. 4.13), *Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions* (Ref. 4.14), and *Guide for Concrete Inspection* (Ref. 4.15)). Additional information is contained in Refs. 4.16 – 4.18. However, the application of requirements presented in these documents to nuclear safety-related concrete structures requires evaluation with respect to items such as accessibility, service history, functional requirements, construction materials, etc. A report is being prepared by Mr. Hookham that has the overall objective of providing a suggested in-service inspection approach for reinforced and prestressed concrete structures in nuclear power plants. Criteria are being established for a recommended procedure that can be utilized to assess the current structural reliability of the safety-related concrete structures and to develop data for use in assessments of future performance. Specific activities include (1) a review and assessment of current NRC and industry-related in-service inspection requirements for reinforced concrete structures; (2) an evaluation of the applicability of available information on criteria, inspection, and testing requirements for general civil engineering concrete structures provided through organizations such as the American Concrete Institute, American Society for Testing and Materials, International Union of Testing and Research Laboratories for Materials and Structures (RILEM), Nuclear Management and Resources Council Industry Reports, etc.; and (3) development of a structural component condition assessment methodology that establishes criteria for relating damage state and environmental exposure in terms of a "three-tiered hierarchy" such as shown in Fig. 4.6. Visual acceptance criteria, per this hierarchy, are being developed to parallel the effort of American Concrete Institute Committee 349 (Ref. 4.19). Basic criteria for acceptance without further evaluation and acceptance after review based on visual inspections have been developed for (1) exposed concrete surfaces; (2) lined concrete surfaces; (3) areas around embedments in concrete; (4) joints, coatings, and non-structural components; and (5) prestressing steel systems. Any condition outside the criteria for these two conditions is considered unacceptable and requires additional nondestructive testing, destructive testing, analytical assessment, or a combination of the three. Degradation-based acceptance criteria are being established for concrete cracking, loss of concrete section, conventional and prestressing steel corrosion, and loss of prestressing force. When completed, the structural component condition assessment methodology will provide guidance for dispositioning of conditions or findings from in-service inspections.

4.4.3 Remedial/Preventative Measures Considerations (Subtask S.3.3)

Under this subtask, basic activities are related to an assessment of repair procedures for concrete material/structural systems and establishment of criteria for their utilization. Techniques that can be used to mitigate the effects of environmental stressors or aging factors will be identified. Recommended preventative measure procedures that can be used to effectively offset, counteract, or minimize any minor deterioration effects to prevent them from becoming significant will be established. Current work is directed at reviews of European and North American practices for repair of reinforced concrete structures, an

assessment of corrosion of reinforced concrete structures, and formulation of a damage assessment and repair prioritization methodology.

Taywood Engineering Ltd. (London, England) has completed an assessment of repair procedures from the European perspective, primarily addressing those for corrosion damage of reinforced concrete (Ref. 4.20). Results obtained through Taywood Engineering Ltd.'s participation in collaborative European projects addressing repair of reinforced concrete structures and methods that can be used to extend the lifetime of a structure, either during the construction stage (incorporation of chemical admixtures or alternative cementing materials) or post-construction (use of surface coating and treatments), have been used in the assessment. Included in the report presenting results of this activity are discussions of (1) criteria used in selection of a particular repair procedure, (2) descriptions of the various repair materials and procedures currently used, (3) an evaluation of the effectiveness of the various repair techniques as determined by both in situ evaluation (testing) or performance history, and (4) an indication of the future direction of concrete repairs in Europe. Depending on the degree of deterioration and the residual strength of an element, the function of a repair may be structural, protection, cosmetic, or a combination of these three requirements. Basic elements of a repair procedure include assessment, choice of repair strategy, detailed design of repair procedure and choice of materials, planning and execution of repair procedure, and maintenance and monitoring of the structure. Repair strategies and procedures are provided in the reference in the form of flow diagrams. In order to repair corrosion-damaged reinforced concrete, it is sufficient to stop one of the three processes governing corrosion occurrence: anodic, cathodic, or electrolytic. Basic repair solutions include (1) realkilization by either direct replacement of contaminated concrete with new concrete, use of a cementitious materials overlay, or application of electrochemical means to accelerate diffusion of alkalis into carbonated concrete; (2) limiting the corrosion rate by changing the environment (e.g., drying) to reduce electrolytic conductivity; (3) steel reinforcement coating (e.g., epoxy); (4) chloride extraction by passing an electrical current (DC) from an anode attached to the concrete surface through the concrete to the reinforcement (chloride ions migrate to the anode); and (5) cathodic protection. Since damage occurring from chloride presence and carbonation are the most important sources of concrete distress in Europe, research activities are concentrating on these two mechanisms. For chloride attack, efforts are underway to provide an improved understanding of the corrosion process, the mechanism of incipient anode development, and the use of cathodic protection to overcome the problem. For carbonation, the emphasis is being placed on anti-carbonation surface treatments, protective properties of patch materials, and the durability/compatibility of these materials. Currently in Europe, the patch repair is the most widely applied method to restore both mechanical damage and spalling caused by corroding reinforcement. Cementitious, polymer-modified cementitious, and polymer-based material systems are used to effect patch repairs. Table 4.4 presents typical properties for these material systems. Coating systems have also been used effectively to increase the resistance to chloride penetration and carbonation, and reduce the concrete moisture content. Application and an indication of the relative protection provided by several coating systems is provided in Table 4.5. Future directions are stressing the importance of repairs to damaged or deteriorated concrete structures. This approach has only recently attained a degree of prominence in Europe. Also, there is a much greater awareness of potential problems that could result from either improper selection of repair materials or techniques, and a consensus is developing with regard to specific technical requirements for repair systems, as opposed to repair materials. Although many European countries have a range of national standards for design and construction of new buildings, and Eurocodes are in the course of preparation, standards and guidelines for repair of damaged structures are much less frequently available. At present, the most widely developed regulations are those developed by the German Committee on Reinforced Concrete (Ref. 4.21). 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. Within the next five years or so, European Standards are expected to be in place.

Wiss, Janney, Elstner Associates, Inc. (Northbrook, IL) has completed an overview of North American practices for repair of reinforced concrete structures. Objectives of this activity were to (1) describe repair materials and procedures commonly used, (2) establish criteria important to selection of a repair strategy, and (3) develop durability ratings for methods used to repair degradation that commonly occurs in reinforced concrete structures (e.g., dormant cracks, active cracks, spalls, and steel reinforcement corrosion). The distinction between this and the Taywood Engineering Ltd. activity is that North America repair activities have concentrated on infrastructure-related facilities such as road and bridge structures, while European repair practices have primarily (but not exclusively) addressed those for corrosion-damaged reinforced concrete in building and general civil engineering structures. A state-of-the-art manual (Ref. 4.22) has been prepared that discusses causes of distress, when a specific repair technique is applicable (e.g. specific crack sizes; how the techniques or materials are used — injection, routing, etc.), how to evaluate and test a repair, how to maintain the repair after it has been installed, the expected life of the repair technique, methods for determining when a repair has failed, and methods for re-repair. Table 4.6 lists typical causes of concrete cracking for active and dormant cracks. Flow diagrams to provide guidance in selection of a repair method for active and dormant cracks are presented in Figs 4.7 and 4.8, respectively. Typical crack, spall, and corroded steel repair methods and their perceived durability are summarized in Tables 4.7 – 4.9, respectively. Properties of rapid set patching materials, grouped by generic family, are presented in Table 4.10. Information specifically addressing repair of reinforced concrete structures in light-water reactor plants was assembled through responses to a questionnaire that was sent to U. S. utilities. In addition to a general description of the particular plant, each utility was requested to provide information on inspection procedures utilized (frequency, technique), types of deterioration that have occurred (voids, spalling, degradation of prestressing steel, etc.), the deterioration mechanism(s) (chemical attack, freeze-thaw, etc.), repair actions that have been undertaken (grout injection, dry pack, etc.), repair locations (basemat, vertical walls, etc.), research investigations on repair materials, and performance history for repair procedures that have been utilized. Responses were received from 29 sites representing 42 units (30 pressurized-water reactors and 12 boiling-water reactors), Fig. 4.9. Results provided indicate that the majority of the plants perform only a general visual inspection of the concrete structures in association with each integrated leak-rate test. Plants that have inspection programs for the concrete structures do so at intervals ranging from annual to five-years with visual methods being used primarily. In general, deterioration of the concrete structures has been minor due to the high initial quality of the construction and the relatively young age of the plants. Twenty-six of the sites reported that they had experienced some type of damage or deterioration to the concrete. Figure 4.10 summarizes the number of incidents of degradation by type and cause. The most common locations of deterioration in the pressurized-water reactor plants were in the containment dome, and in the walls and slabs of the auxiliary structures. Slabs, walls, and equipment supports (or pedestals) were the most common areas of damage in the boiling-water reactor plants. Twenty-seven of the sites reported that damaged concrete had been repaired, with most of the problems having occurred during construction. Dry packing has been used primarily to repair cracks and spalls. Other methods commonly used to repair cracks include epoxy injection, grout injection, flexible sealing, drilling and plugging, stitching, and routing and sealing. Little information was provided on materials that have been used for repair, repair procedures, or durability of repairs. When the performance of a repair was evaluated, visual inspection was used.

Corrpro Companies, Inc. (Medina, Ohio) has completed a review of corrosion of reinforced concrete structures, with an emphasis on stray electrical current-induced corrosion and use of cathodic protection to control the occurrence of corrosion in these

structures (Ref. 4.23). Four components are required for corrosion to occur: anode, cathode, electrolyte, and metallic path. Types of corrosion that can occur on metals embedded in concrete are shown schematically in Fig. 4.11. Conditions that affect the corrosion rate include oxygen level, conductivity of electrolyte, concentration of ions, anode/cathode area, temperature, corrosion resistance of metals, electrical relationship between metals, radiation (affects crystal structure of metal potentially making it more susceptible to localized corrosion), and bacteria. Methods available to detect corrosion include visual observations, half-cell potential measurements, delamination detection, electrolyte chemistry, corrosion monitors, acoustic emission, radiography, ultrasonics, magnetic perturbation, metallurgical properties, and electrical resistance. Remedial measures for corrosion and their advantages and disadvantages are summarized in Table 4.11. Stray electrical current is any current flowing in a path other than its intended circuit. Stray current will take all available conductive parallel paths back to its source and has a magnitude inversely proportional to the resistance of the path(s). With the exception of prestressing tendons, the portion of the structure picking up stray current will not experience corrosion damage. However, the area of discharge will result in metal dissolution. The amount of metal lost will be directly proportional to the current density and duration of exposure. In the case of prestressing tendons, hydrogen embrittlement corrosion may result where the structure is picking up the stray current. Potential sources of stray current at nuclear power plants are summarized in Table 4.12. Half-cell potential versus time study (Fig. 4.12), half-cell potential versus distance study (Fig. 4.13), and cooperative (interference) testing are techniques commonly used to detect stray current. Mitigation measures for stray current include prevention or elimination of the current source, installation of cathodic protection, draining the current from the affected source, and shielding the structure from the source (Fig. 4.14). Cathodic protection is not only a rehabilitation technique for corroding structures, but is also a corrosion prevention technique for steel that may lose its inherent passivity at a later time by contamination of the concrete or even due to occurrence of stray currents. It mitigates the corrosion reaction by imposing direct current flow between an anode placed on the concrete surface and the metal to be protected. Cathodic protection systems are of the impressed current or sacrificial anode type (Fig. 4.15). Impressed current systems use a DC power supply (rectifier) to force current flow from a relatively inert electrode (anode) through the concrete to the steel surface to be protected. In sacrificial systems, a metal that is more anodic (higher tendency to corrode) than the steel embedded in the concrete is used as the source of energy. The theory of cathodic protection is basically that all galvanic corrosion of steel is halted when the potential difference between the anodic and cathodic areas on the steel can be made equal to zero. Criteria commonly used for atmospherically exposed bridge structures to indicate when this condition is sufficiently met include 100 mV polarization decay, E (potential) log I (current) analysis, macrocell current reversal, fixed current density, voltage probe measurement, half-cell potential measurement, and rebar probe polarization decay. To be most effective, cathodic protection requires electrical continuity of all metallic components within a concrete structure. Cathodic protection systems should not be considered (1) for atmospherically-exposed steel such as embedments extending from concrete, (2) when a cathodic protection anode cannot be installed in an electrically continuous electrolyte, (3) if there is no or poor electrical continuity within the majority of steel reinforcement, or (4) when the system cannot be designed to avoid hydrogen embrittlement of the prestressing steel. Since cathodic protection systems are DC, they have the potential to cause stray current corrosion in other structures (Fig. 4.16), and application of cathodic protection to high-strength steel used in prestressing wires or strands may result in embrittlement due to generation of hydrogen at the cathode.

Howard University has developed an approach to categorize and rate safety-related concrete structures (subelements) in nuclear power plants in terms of their repair requirements (i.e., urgency) (Ref. 4.24). Figure 4.17 presents a flow diagram of the basic approach utilized. The existence of structural defects (e.g., cracks) and the environmental

exposure were considered to be the two primary parameters controlling repair urgency of reinforced concrete structures. In order to assess the damage state, the net effects of the occurrence of degradation on future performance must be determined. Because of the potential impacts of cracking on structural integrity and the fact that many of the potential degradation mechanisms manifest themselves in the form of cracks, the damage state was determined on the basis of cracking. Cracks are generally classified by direction, width, and depth and may be described as longitudinal, transverse, vertical, diagonal, or random. However, in the development of the methodology, the cracks were evaluated in terms of width only. Three basic categories of cracks were adapted: (1) fine – widths less than one mm, (2) medium — widths between one and two mm, and (3) large – widths greater than two mm. Four environmental exposure conditions were considered: (1) subterranean, (2) direct, (3) indirect, and (4) continuous fluid. These exposures were considered because of their potential to have synergistic effects with cracking to create damage that may result in limiting conditions relative to a structure's serviceability. Environmental exposure ratings are based on those provided in Ref. 4.7 which were developed based on several considerations: (1) historical environmental data, (2) exposure conditions for all surfaces of the structure or subelement, (3) accessibility of the structure's exposed surfaces for inspection, and (4) quantity/severity of the specific environmental conditions to which it is exposed. Typical ranges of environmental exposure ratings for subterranean, direct, indirect, and continuous fluid exposure were 8–10, 4–6, 2–4, and 4-7, respectively. Determination of the damage significance is based on the current damage state (e.g., crack width). Damage significance values ranged from two to ten, with ten representing the most limiting condition. The repair prioritization number, or rating, is established as follows

$$RPN = w_1(EE) + w_2(DS), \tag{4.4}$$

where

RPN	= repair priority number,
EE	= environmental exposure,
DS	= damage significance, and
w_1, w_2	= weighting factors.

Suggested values for w_1 and w_2, based on a sensitivity analysis (Ref. 4.24), are 6 and 14, respectively. Depending on the RPN value obtained, the structure (or subelement) will fall into one of three categories: (1) no or tolerable damage; (2) increased inspections, monitoring, or structural assessments required; or (3) repair required. Criteria to place a subelement into one of these three categories based on the RPN determined require development. When criteria have been established, results developed under this activity will provide a logical connection between the structural aging assessment methodology (Ref. 4.7) and the repair activities (Refs. 4.20 and 4.22).

4.5 PLANNED ACTIVITIES

The reports on potential structural issues related to aging of post-tensioning systems in nuclear power plant containments, and a recommended procedure that can be utilized to assess the current structural reliability of the safety-related concrete structures and to develop data for use in assessments of future performance will be finalized and issued.

4.6 REFERENCES

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4.7 MILESTONE STATEMENT AND SCHEDULE

The statement and schedule for Task S.3 milestones are given in the following charts.

MILESTONE STATEMENT AND SCHEDULE

TASK: S.3 STRUCTURAL COMPONENT ASSESSMENT/REPAIR TECHNOLOGY

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.3 STRUCTURAL COMPONENT ASSESSMENT/REPAIR TECHNOLOGY																							
S.3.1 LWR Critical Component Classification																							
(Activities Completed)																							
S.3.2 NDE/Sampling Inspection Technology																							
A. Issue Reports on Inservice Inspection and Structural Assessment Techniques for Concrete Materials/Structures.																							
(Activities Completed)																		a,b					
B. Issue Reports on Recommended In-Service Inspection Methodology.																							
a. Complete Draft Report on In-Service Inspection Guidelines for Concrete Structures.																							
b. Complete Draft Report Assessing Potential Post-Tensioning System Structural Safety Issues.																							

MILESTONE STATEMENT AND SCHEDULE

TASK: S.3 STRUCTURAL COMPONENT ASSESSMENT/REPAIR TECHNOLOGY

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.3.3 Remedial/Preventative Measures Considerations									b														
A. Issue Reports on Review of Techniques for Repair of Concrete Materials and Structures																							
a. Complete Draft Report Assessing European Repair Practices (Activity Completed)																							
b. Complete Draft Report Assessing North American Repair Practices and Presenting Results of Questionnaire on Concrete Performance in Nuclear Power Plants																							

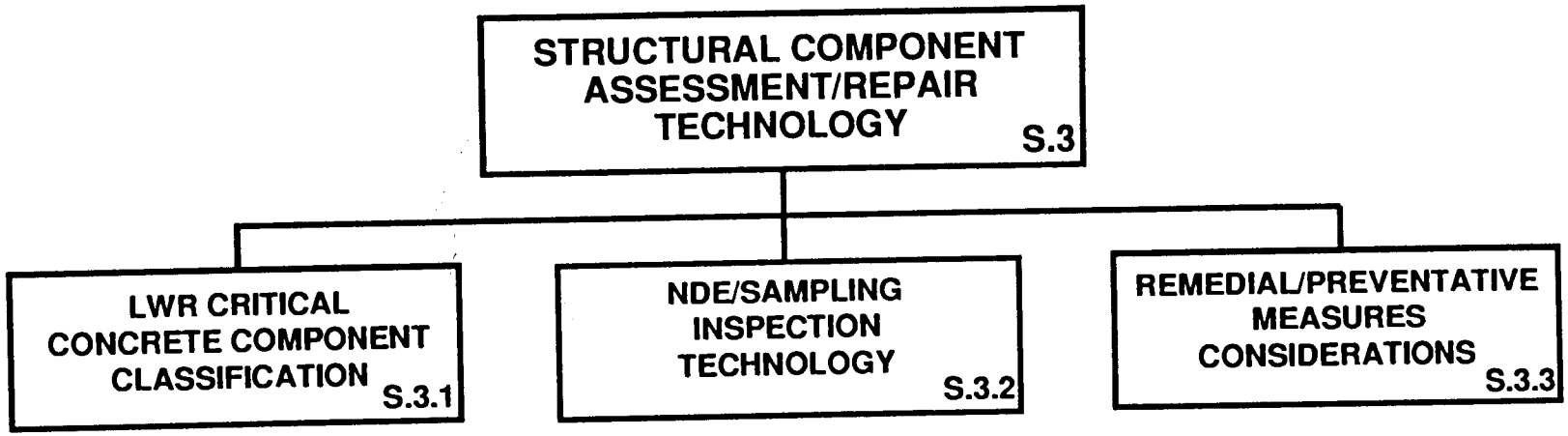


Fig. 4.1 Level 2 work breakdown structure for SAG Task S.3: Structural Component Assessment/Repair Technology.

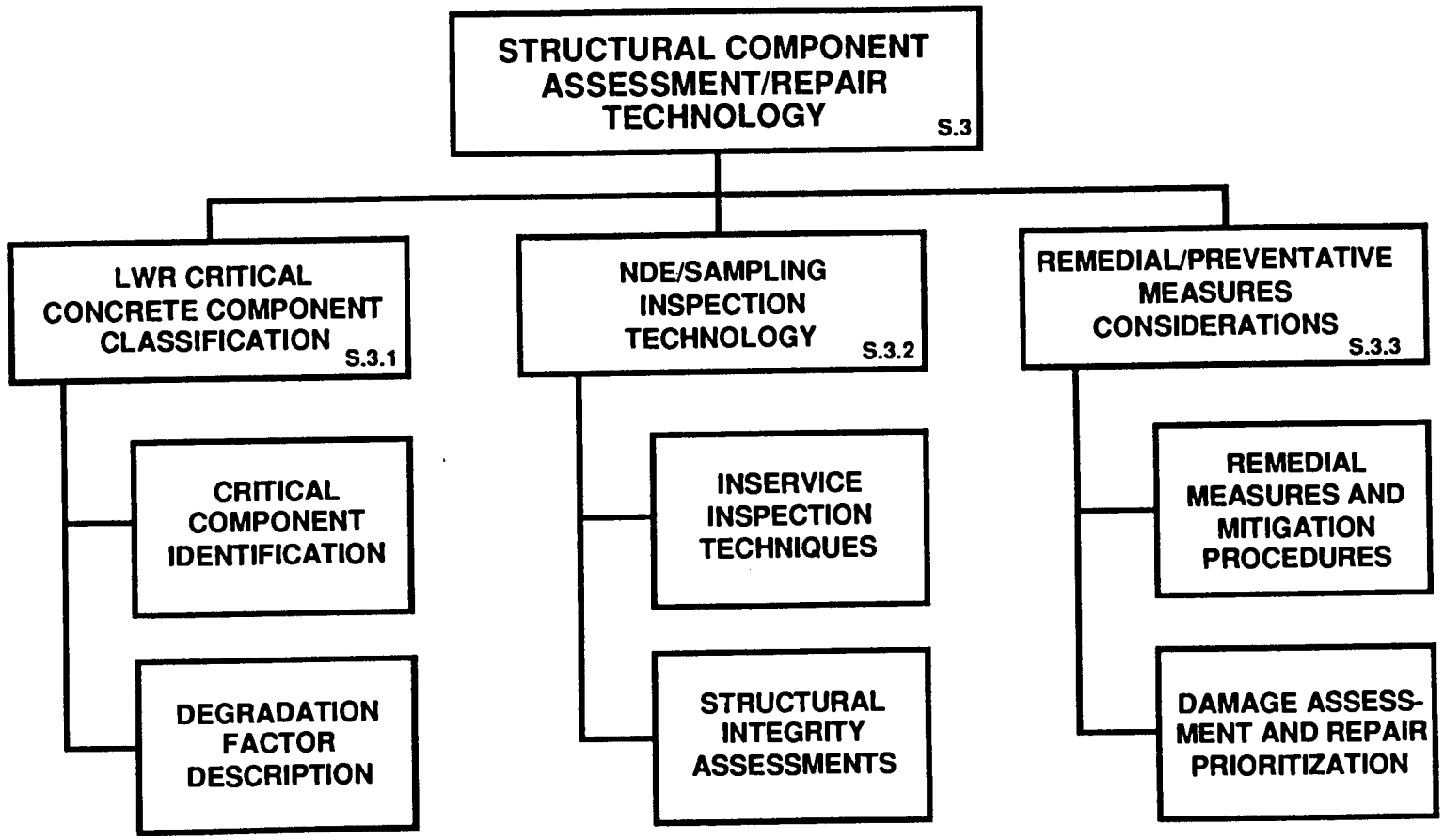


Fig. 4.2 Level 3 work breakdown structure for SAG Task S.3: Structural Component Assessment/Repair Technology.

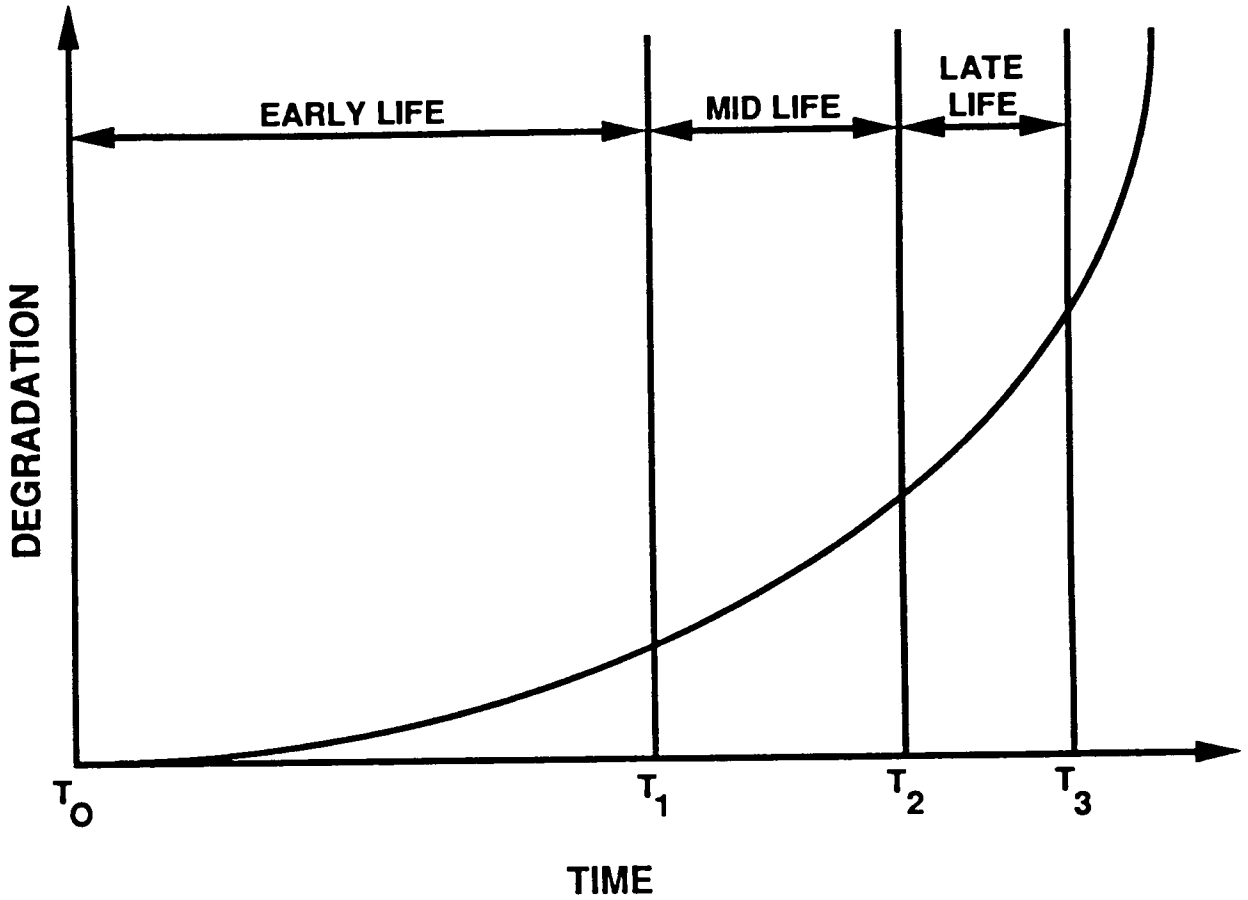


Fig. 4.3 General time rate of degradation curve for concrete structures.

Source: Adaptation of diagram showing typical rate of deterioration of concrete subjected to sulfate attack or alkali-silica reaction presented in J. R. Clifton and L. I. Knab, *Service Life of Concrete*, NUREG/CR-5466, National Institute of Standards and Technology, Gaithersburg, MD, November 1989.

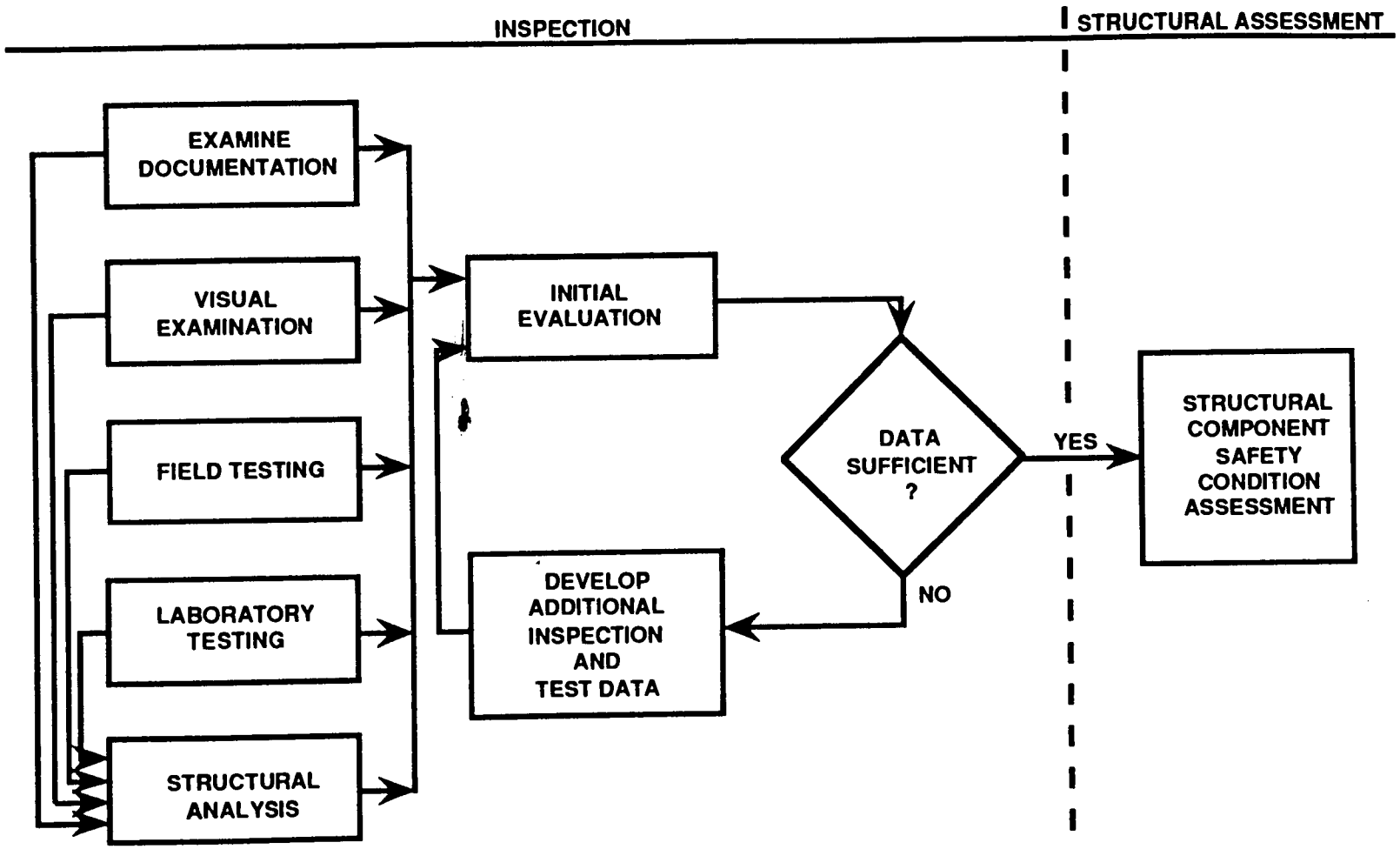


Fig. 4.4 Concrete component current condition assessment methodology.

Source: Adaptation of a procedure presented in T. L. Rewerts, "Safety Requirements and Evaluation of Existing Buildings," *Concr. Int'l.* 7(4), American Concrete Institute, Detroit, April 1985.

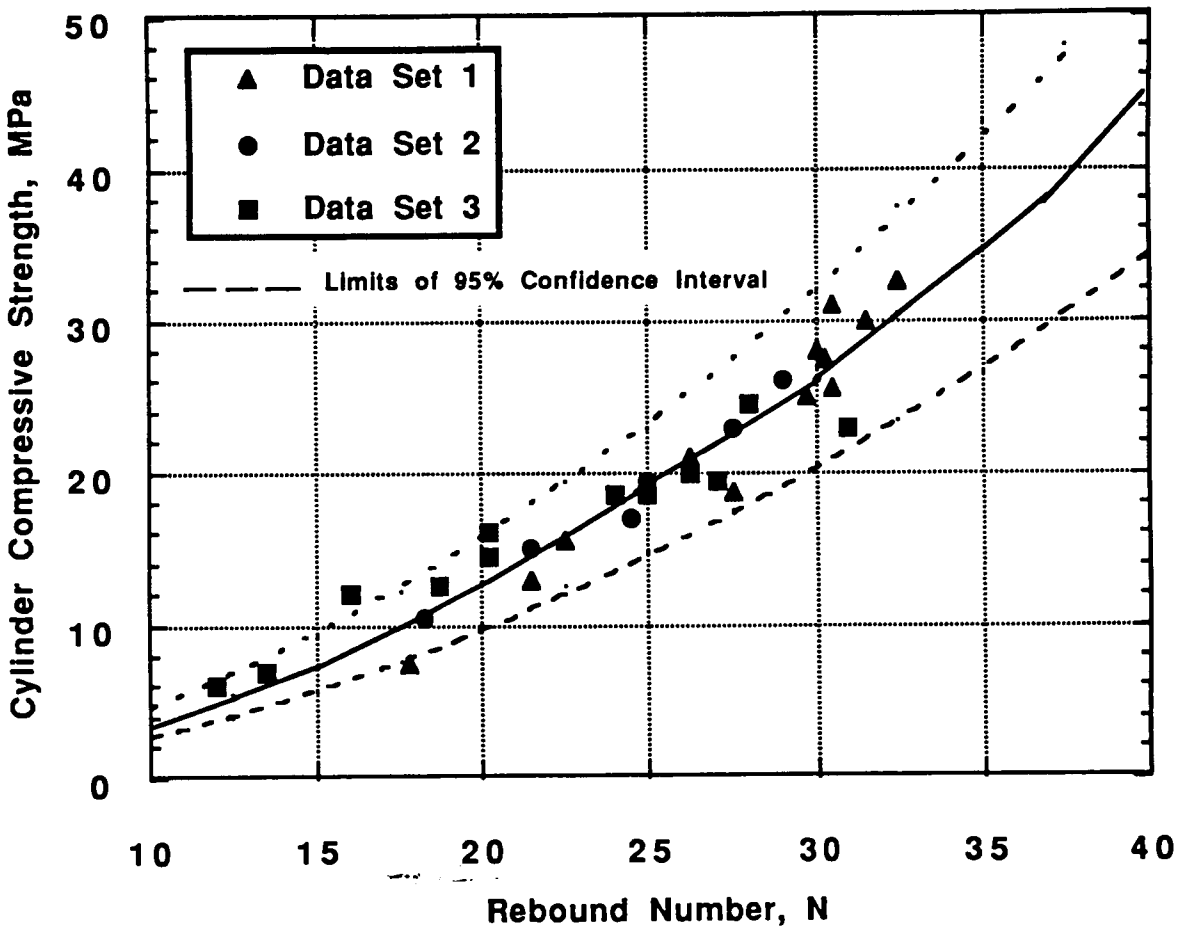


Fig. 4.5 Relationship between cylinder compressive strength and rebound number for selected gravel concretes.

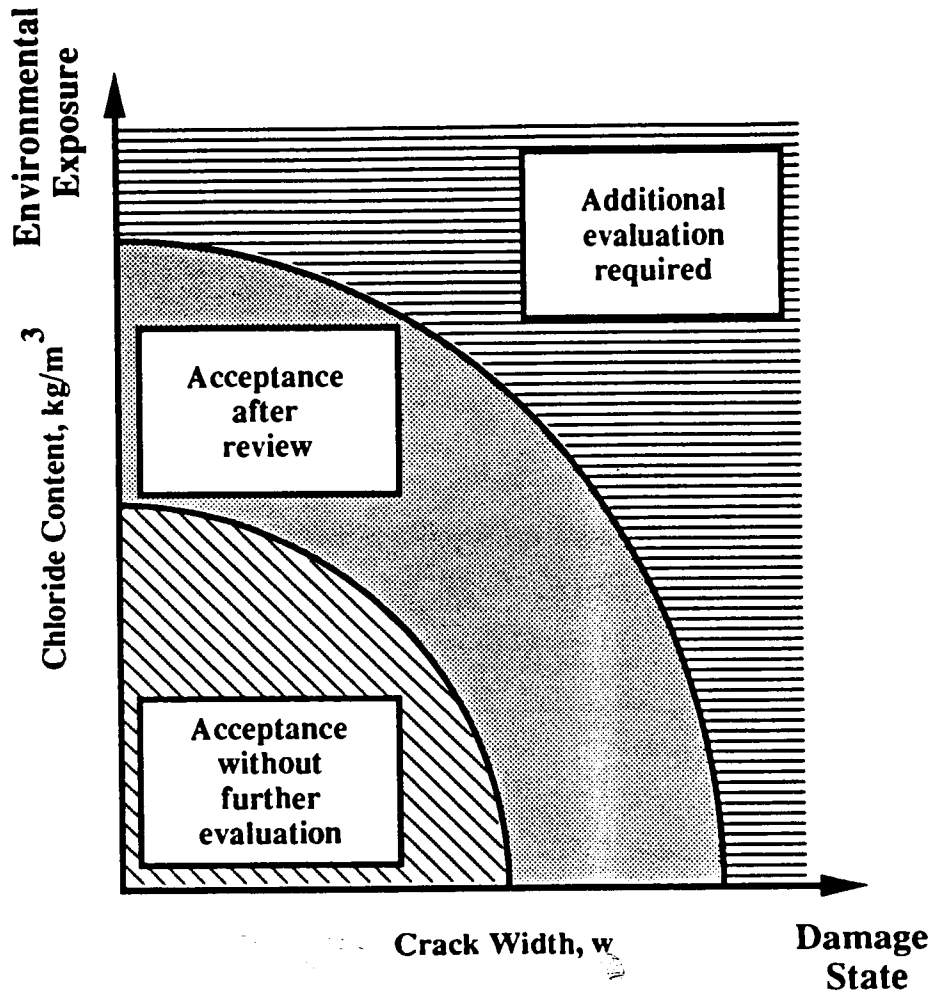


Fig. 4.6 "Three-tiered" hierarchy relating damage state and environmental exposure.

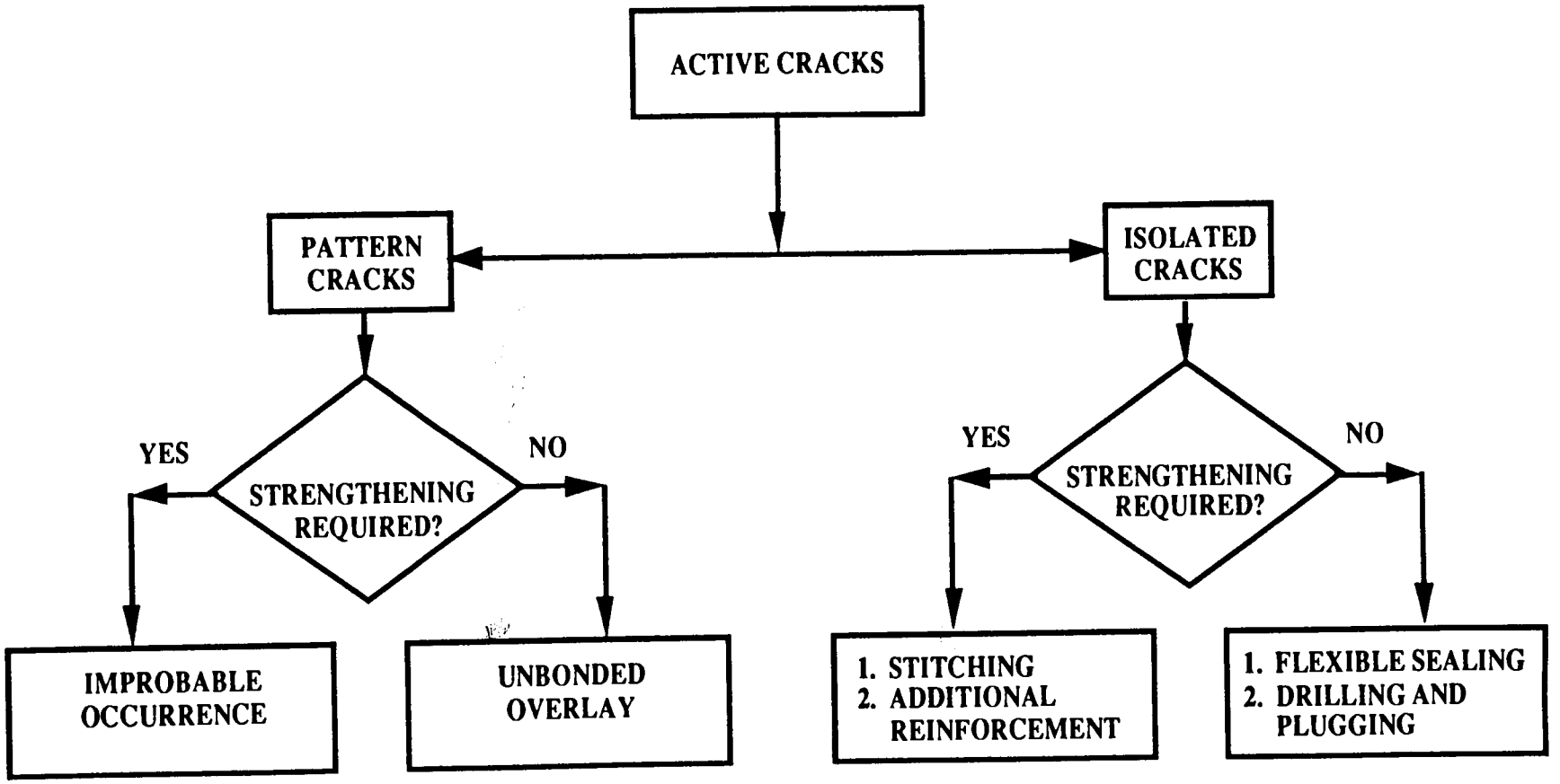


Fig. 4.7 Selection of repair method for active cracks.

Source: "Evaluation and Repair of Concrete Structures," EM 1110 - 2 - 2003, U.S. Army Corps of Engineers, Washington, DC, July 1986.

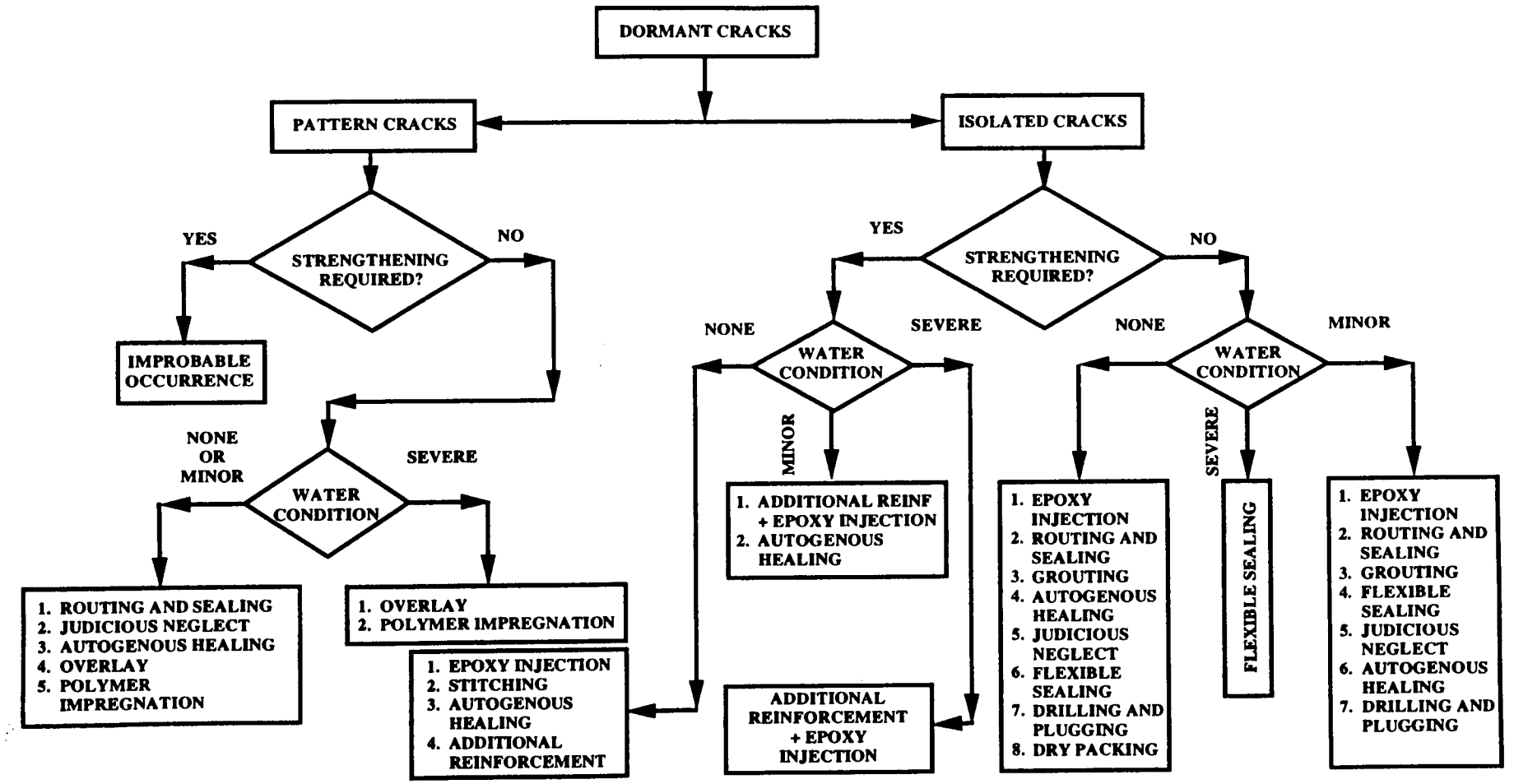


Fig. 4.8 Selection of repair method for dormant cracks.

Source: "Evaluation and Repair of Concrete Structures," EM 1110 - 2 - 2003, U.S. Army Corps of Engineers, Washington, DC, July 1986.

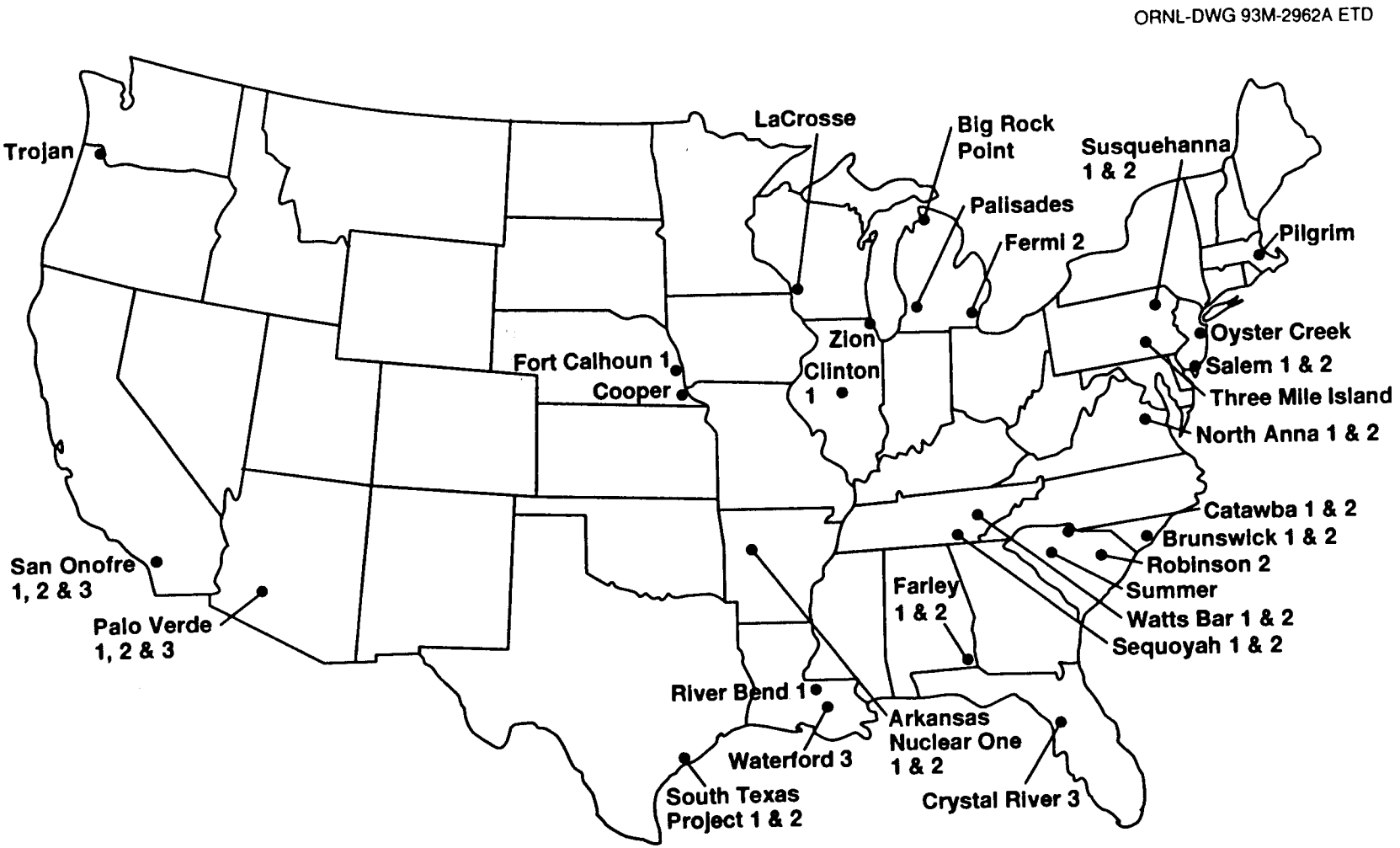
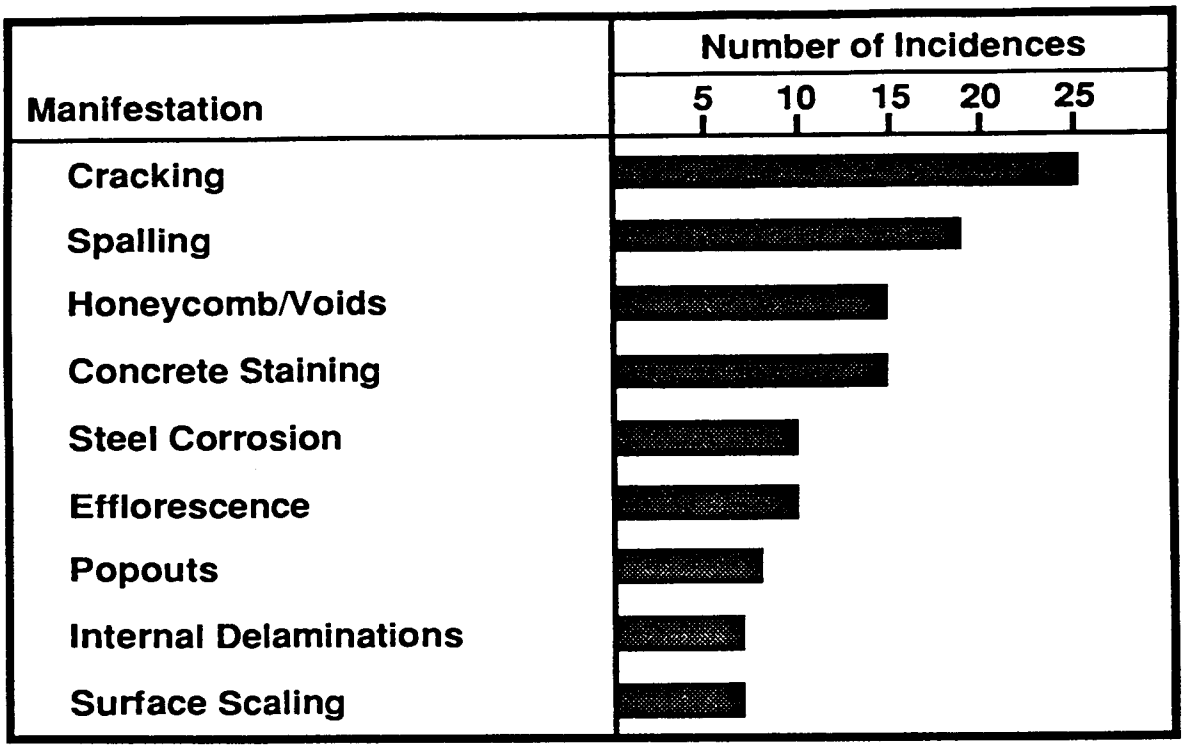
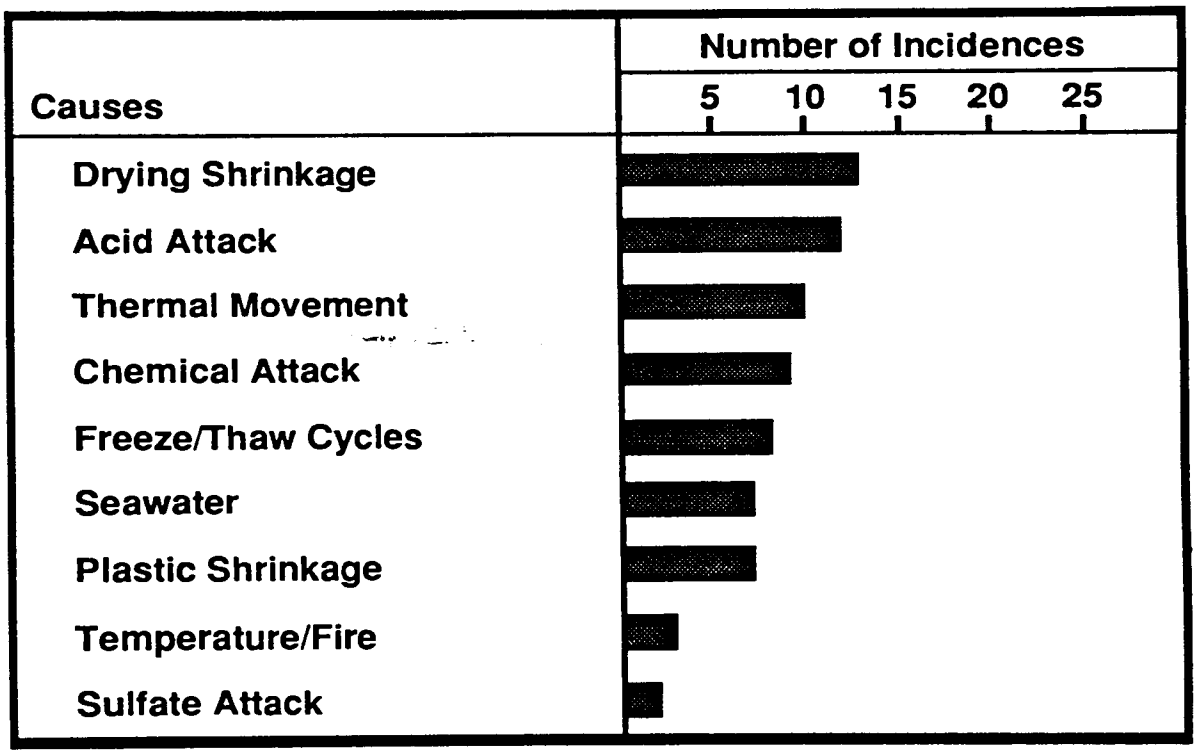


Fig. 4.9 Twenty-nine sites having forty-two units responded to the questionnaire addressing performance of nuclear power plant concrete structures.



a. Type of degradation.



a. Cause of degradation.

Fig. 4.10 Summary of survey responses related to degradation of light-water reactor concrete structures grouped according to (a) type and (b) cause.

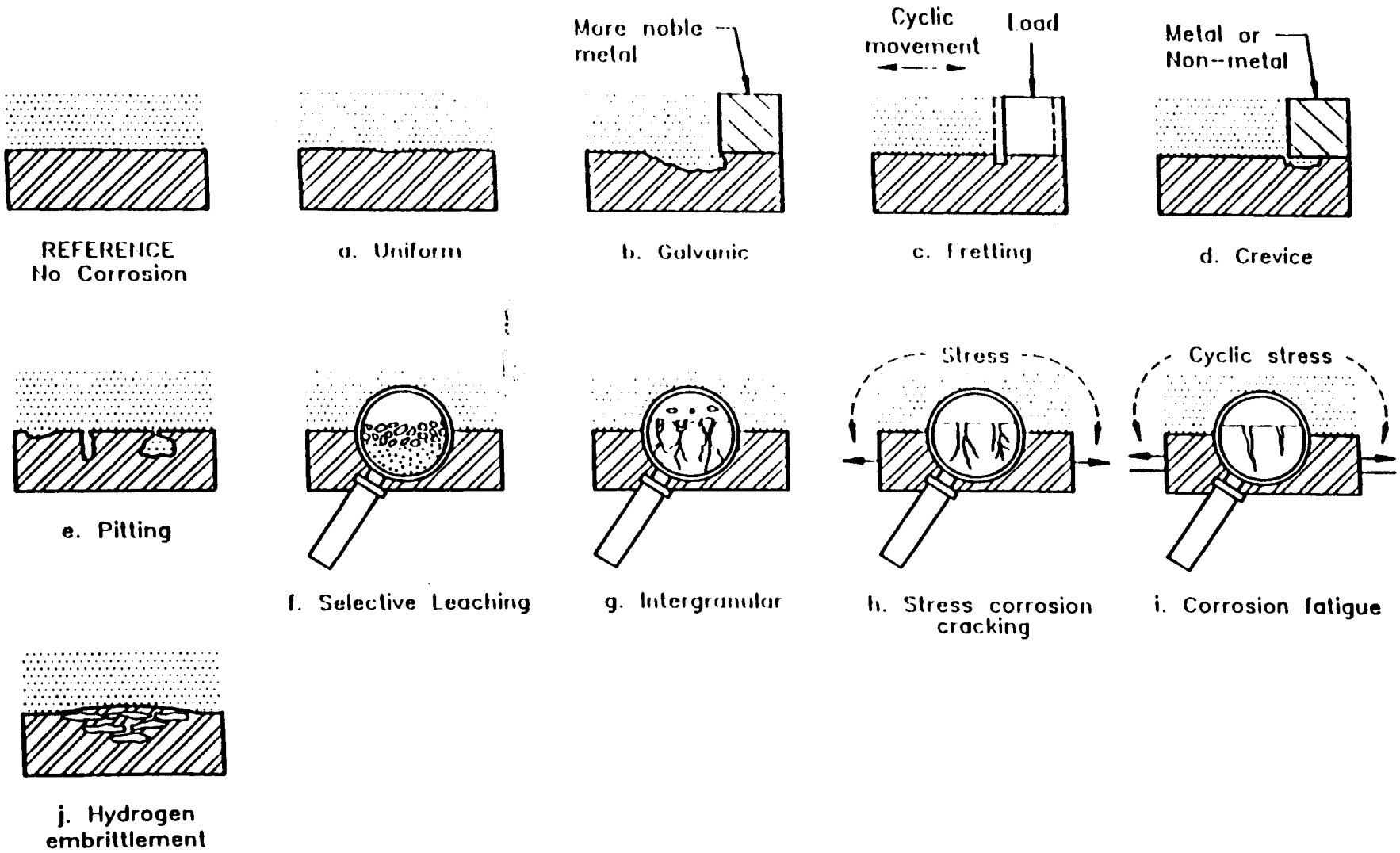


Fig. 4.11 Forms of corrosion that can be found on metals embedded in concrete.

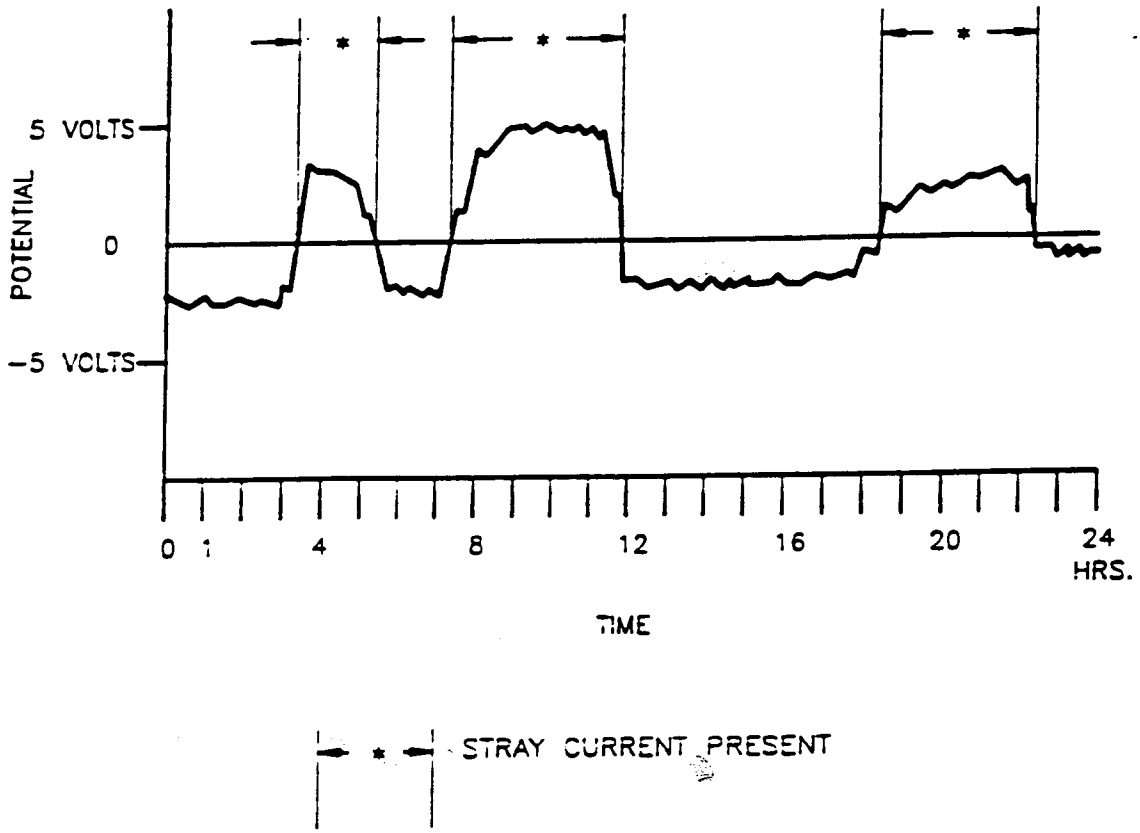
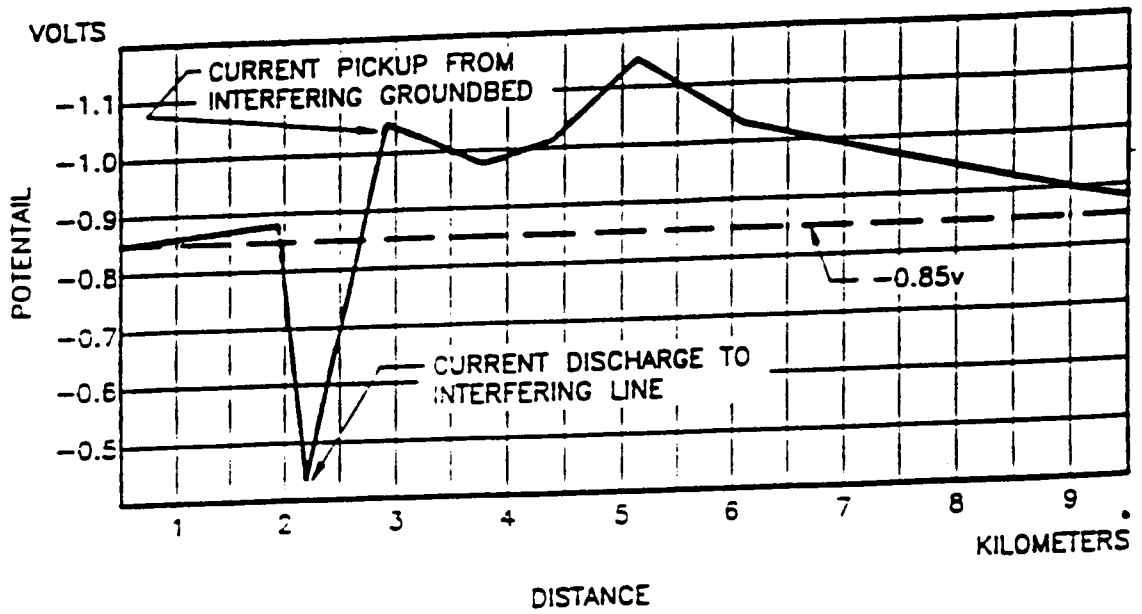


Fig. 4.12 Structure to soil measurements of half-cell potential versus time indicating presence of dynamic stray current.



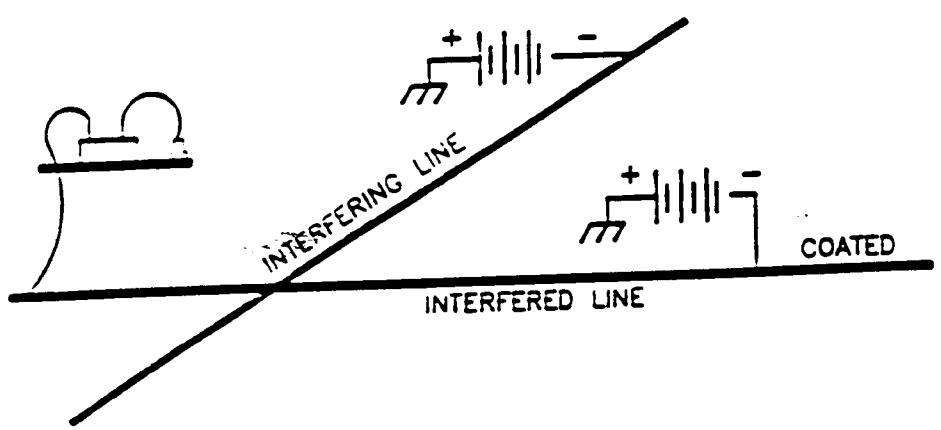


Fig. 4.13 Structure to soil potential measurements over distance indicating presence of static stray current.

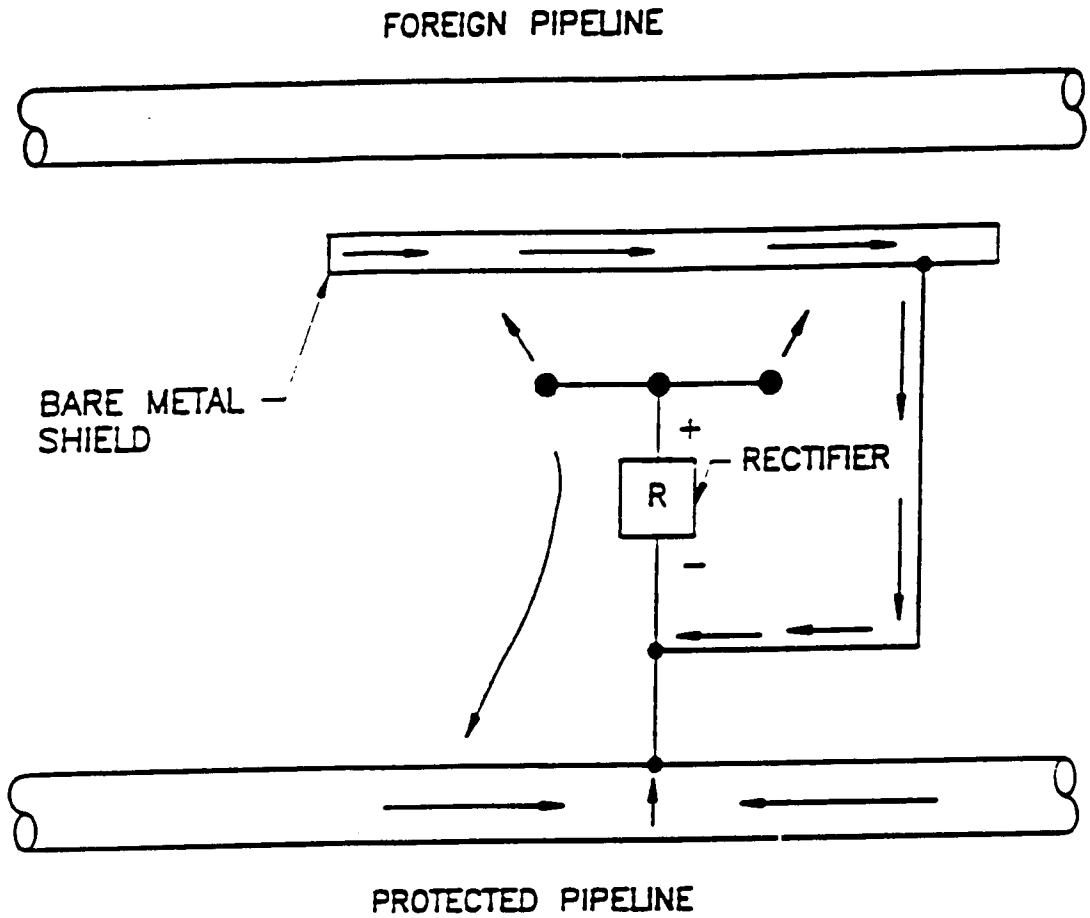


Fig. 4.14 Example of stray current mitigation through installation of a metallic shield.

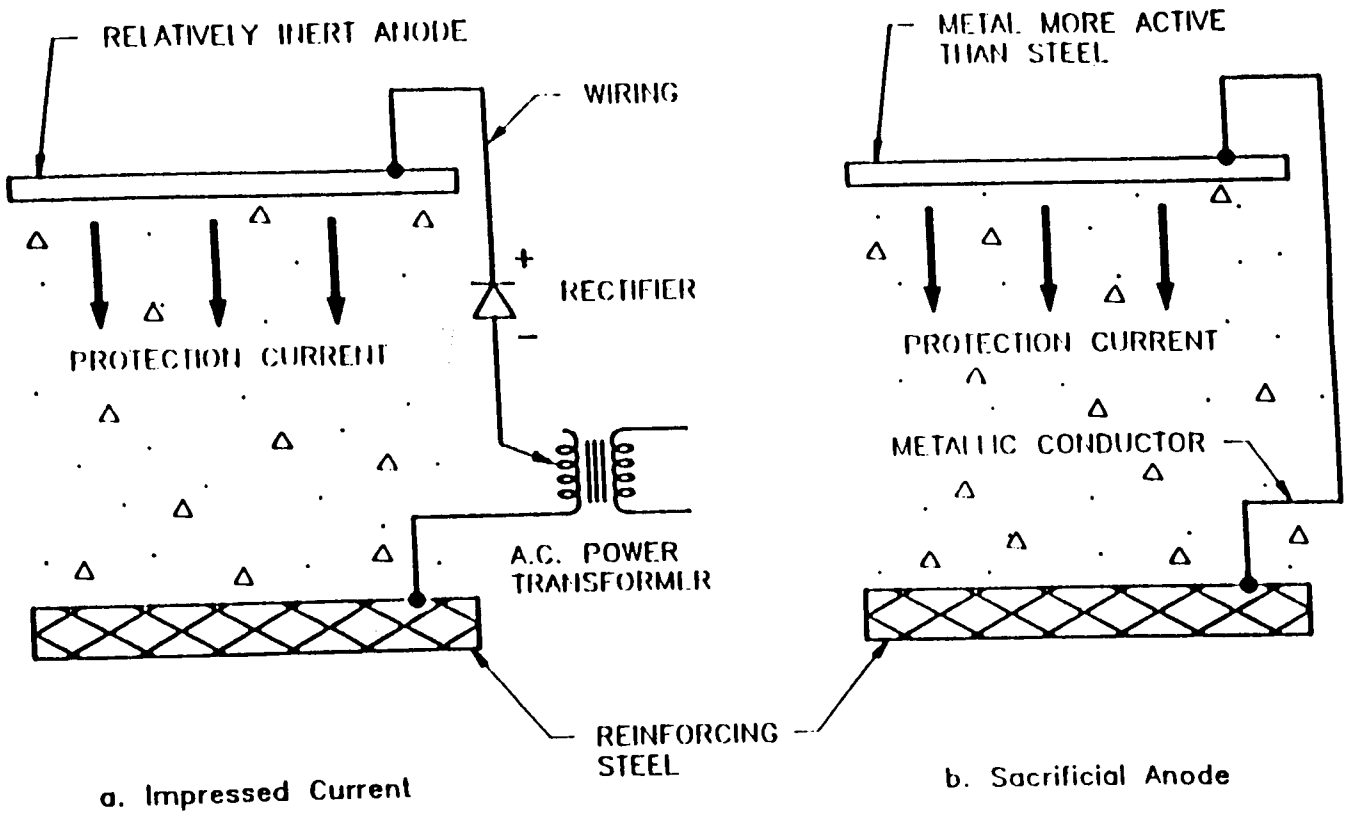


Fig. 4.15 Schematic diagram showing mechanism of impressed current and sacrificial anode cathodic protection systems.

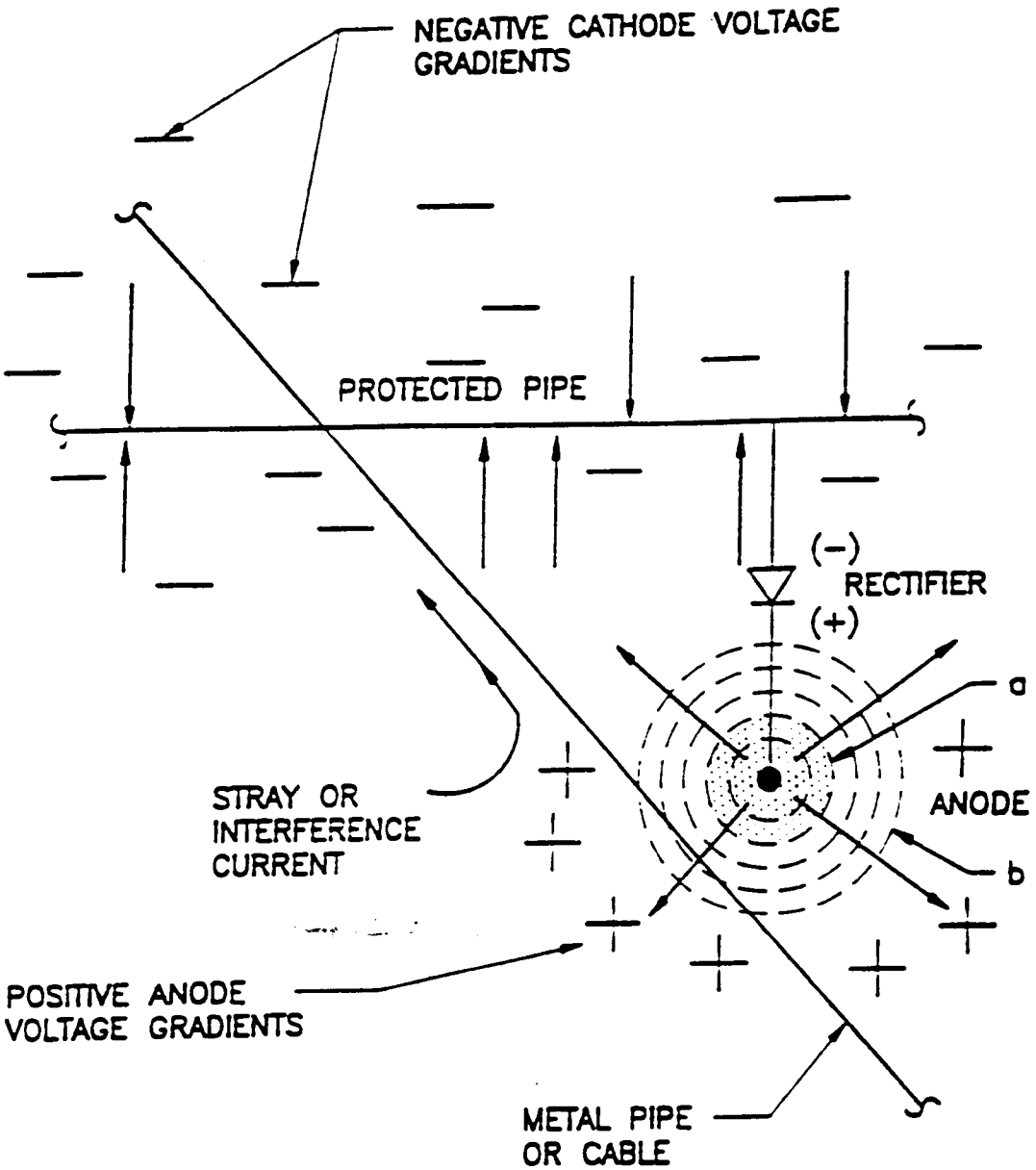
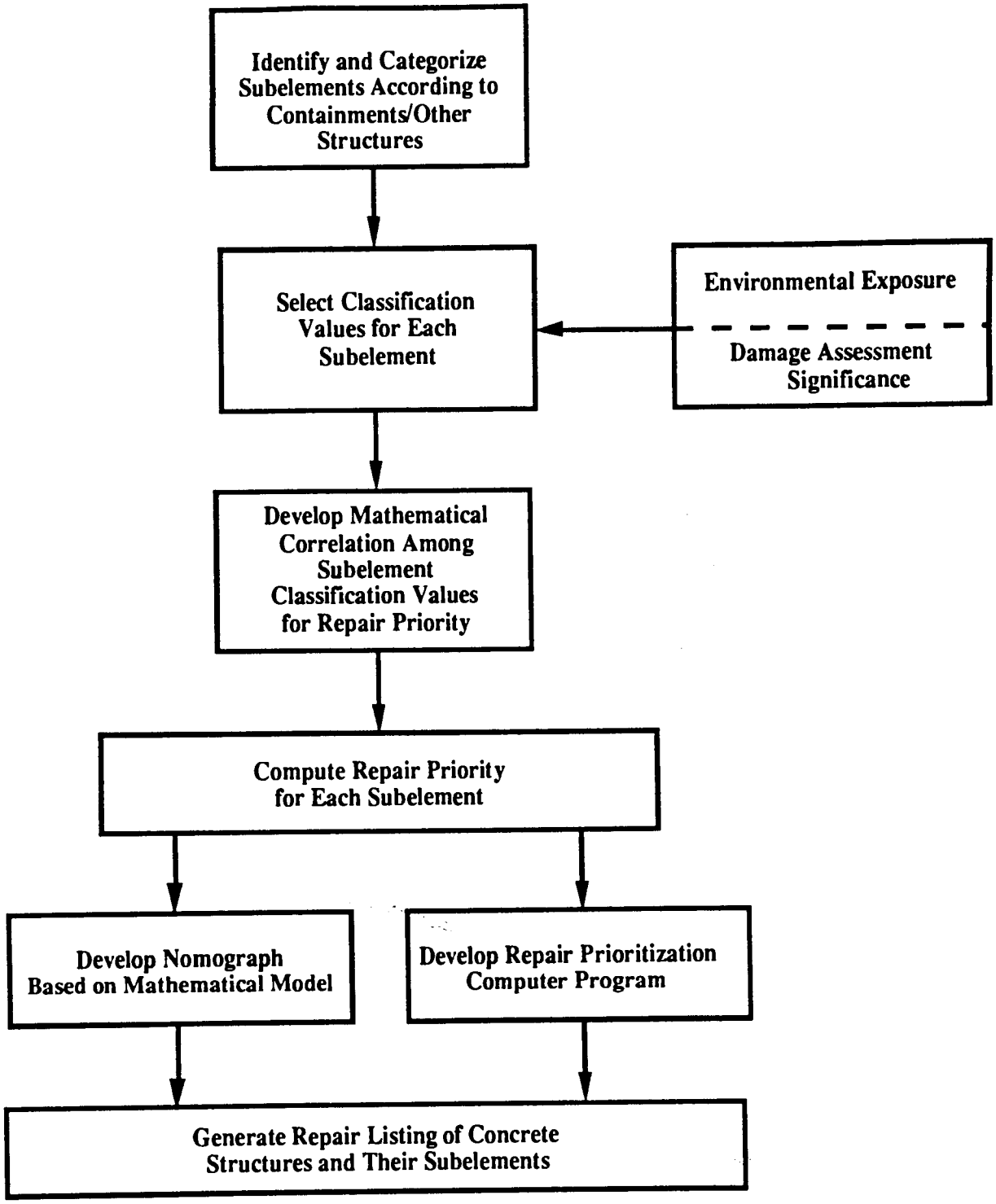


Fig. 4.16 Illustration of impact of voltage gradient created by operation of an impressed current cathodic protection system at (a) low and (b) high voltage gradients.



F.g. 4.17 Flow diagram of repair prioritization methodology approach

Table 4.1. Nondestructive evaluation methods for inspection of concrete materials

Material and characteristic	Available methods of detection	
Primary	Secondary	
Concrete		
General quality	Ultrasonic pulse	Ultrasonic pulse echo
velocity	Gamma radiography[a]	
Rebound hammer		
Penetrating probe		
Cracking/voids	Visual inspection	Ultrasonic pulse echo
Ultrasonic pulse velocity	Gamma radiography[a]	
Acoustic impact		
Strength	Penetrating probe	Breakoff methods
Rebound hammer	Surface hardness methods	
Pullout methods		
Mild steel reinforcing		
Location/size	Pachometer	Ultrasonic pulse echo
Gamma radiography[a]	Penetrating radar	
Corrosion	Visual inspection[b]	Rate of corrosion probes
Electrical potential measurements		
Prestressing tendons		
Loads	Tendon liftoff tests	Load cells
Corrosion	Visual inspections	Corrosion inhibitor
Mechanical property tests	analysis	
Tendon load vs elongation tests		
Concrete embedments		
Visual inspections		
Mechanical testing		

[a]Limited to concrete thickness $\leq$450 mm.

[b]Reflected through cracking and staining observed at concrete surface.

Table 4.2 Summary of applications for testing methods: nondestructive.

TEST METHOD	PRINCIPLE	MAIN APPLICATION	ADVANTAGES	LIMITATIONS
Visual	Includes detailed visual examination of observed distress.	To obtain general information regarding concrete distress.	Provides valuable information as to cause(s) of distress and extent of damage.	Provides information on the condition of the exposed surface only. Additional testing methods are required.
Audio Method	Utilizes the difference in sounds to distinguish between delaminated and nondelaminated areas of the test structure.	To locate delaminations and voids.	Quick and inexpensive method. No extensive training is required.	Very subjective to the person performing the test.
Electrical Method	Uses the resistance and potential difference measurements of a structure to determine the moisture content and rate of corrosion of the structure.	To determine the rate of corrosion of a structure.	Quick and inexpensive method. No extensive training is required.	Provides only a potential rate of corrosion and not the actual amount of corrosion present. It is also affected by moisture content.
Impulse Radar	Uses the principle of transmitted and reflected waveforms to locate objects in the structure tested.	To locate voids, embedded reinforcement, delaminations, flaws in concrete, tanks, and utilities embedded in the ground.	Quick, portable, and accurate in locating objects. No damage to concrete.	Affected by moisture. Skills are required in analysis of results.
Infrared Thermography	Uses the principle that all objects emit infrared rays. The infrared camera receives these rays and displays them on a color monitor.	To locate voids.	Quick and portable. No damage to concrete.	Affected by moisture. Skill is required in the analysis of the results. Temperature dependent.
Magnetic Method	Generates a magnetic field and determines the intensity of the magnetic field.	To determine depth and location of reinforcement.	Quick and inexpensive method. No extensive training is required.	Temperature dependent. Ineffective in heavily reinforced area.
Microscopic Refraction	Estimates time traveled from the point of impact to the receiver.	To locate cracks, voids, and assess quality of concrete.	Quick and causes no damage to the concrete.	Influenced by the method of impact used.

Table 4.2 (con't.) Summary of applications for testing methods: nondestructive.

TEST METHOD	PRINCIPLE	MAIN APPLICATION	ADVANTAGES	LIMITATIONS
Modal Analysis	Dynamic test based on vibrations induced to a structure.	Determines vibrational response of a structure.	Provides information about nature of structure when subjected to a dynamic load.	Relatively slow and costly process.
Nuclear Method	Emits gamma rays and indicates the amount returned.	To determine the density of hardened concrete.	Has the ability to determine moisture present as a function of depth.	Expensive, heavy, slow, and needs a skilled operator. The density found is only for the top portion of the concrete.
Radiography	Gamma radiation attenuates when passing through the concrete. Extent of attenuation is controlled by density and thickness of concrete.	Locating internal cracks, voids, and variations in density and composition of concrete. Locating embedded reinforcing steel and voids in concrete.	Portable and relatively inexpensive compared to X-ray. Internal defects can de detected. No damage is done to the concrete.	Radiation intensity cannot be adjusted. Qualified technician is required to operate the instruments because of the radiation source. Two opposite surfaces of component must be accessible.
Rebound Hammer	Measure surface hardness. Spring driven hammer strikes the surface of concrete and rebound distance is noted on scale.	Estimation of compressive strength, uniformity, and quality of concrete.	Inexpensive. Large amount of data can be quickly obtained. Good for determining uniformity of concrete. No damage to concrete tested.	Results are affected by the condition of the concrete surface tested. Does not give precise strength predictions. Results are dependent on the test location.
Ultrasonic Pulse Velocity	Measures the transit time of an induced-pulsed compressional wave propagating through the concrete.	Estimation of the quality and uniformity of concrete. Locates voids, cracks, and estimates depth of rebars.	Test can be performed very quickly. It can also locate voids, cracks, and determine the depth of the reinforcement. No damage to the structure.	Does not give precise estimation of strength. Skills are required in analysis of results. Moisture variation and presence of rebar can affect results.

Table 4.3 Summary of applications for testing methods: destructive.

TEST METHOD	PRINCIPLE	MAIN APPLICATION	ADVANTAGES	LIMITATIONS
Air Permeability	Determine the rate of recovery of air in a test hole after evacuation.	Insitu assessment of the resistance of concrete to carbonation and to penetration of aggressive ions.	Locates corrosion and voids in grouted structural members.	Only a research model has been built.
Break-Off Test	Measures the lateral force required at the top to break off the core at the bottom.	Estimation of strength of concrete.	Inexpensive and quick.	Minor repairs needed.
Chemical Method	Determines chemical characteristics of the concrete through different tests.	To identify chemical characteristics and determine chemical contents in concrete.	Provides information that may assist in determining cause(s) of distress.	Destructive and slow test to perform.
Cores	Physical measurement of actual condition using standard ASTM test methods.	To supplement and/or verify NDT results.	Very informative.	Destructive and slow test.
Probe Penetration (Windsor Probe test)	Measures the depth of penetration into the concrete. Surface and sub-surface hardness can be measured.	Estimation of compressive strength, uniformity, and quality of concrete.	Equipment is simple and durable. Good for determining quality of surface concrete.	Damages small areas. Does not give precise prediction of strength. Results are dependent upon firing mechanism.
Pullout	Measures the force required to pull out a steel rod with an enlarged head cast into the concrete.	Estimates the compressive and tensile strength of concrete.	Directly measuresthe inplace strength of concrete.	Pull-out devices must be inserted during construction or placed by drilling into hardened concrete. Minor repairs are needed. Correlation to compressive strength is still questionable.

Table 4.4 Typical properties of patch repair materials.

Material Property	Patch Repair Material			
	Concrete	Mortar	Polymer Modified Mortar	Epoxy Mortar
Compressive Strength, MPa	20 - 50	20 - 50	10 - 55	55 - 100
Elastic Modulus, GPa	20 - 40	15 - 35	10 - 20	0.5 - 20
Thermal Expansion Coefficient (10^{-6} mm/mm/°C)	7 - 13	8 - 20	8 - 20	25 - 30
Maximum Service Temperature Under Load (°C)	>300	>300	100 - 300	40 - 80

Table 4.5 Relative performance for several coating systems for concrete materials.

Coating	Application*				Protection*			
	Damp Conditions	Alkali Resistance	Easy 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.

Table 4.6 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 which 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 4.7 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, $C\ell$ 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	5	
	Additional reinforcing	4	
	Strengthening	3	
Active cracks	Penetrating sealer	3	Cracks less than 0.5 mm (0.020 in.)
	Flexible sealing	3	Requires maintenance
	Rout 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 4.8 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 (1 1/2 in.) thickness
	Polymer overlays	2	25 to 50 mm (1 to 2 in.) thickness
	Latex-modified concrete overlays	2	Greater than 30 mm (1 1/4 in.) 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
	Shotcrete	3	Good for large areas

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

Table 4.9 General guide to repair options for corroded reinforcing steel

Repair Options	Perceived durability rating (1–5*)	Commentary
Two coats of epoxy on steel	2	Need near white steel
Polymer-modified bar coatings	2	Need near white steel
Zinc rich primer	3	Need near white steel
Calcium nitrate patch	2	Incorporates corrosion inhibitor
Polymer concrete patch	3	Reduces corrosion, high strength
Low w/c PCC	3	Corrosion outside patch accelerated
Cathodic protection	2	Expensive, also has maintenance costs

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

Table 4.10 Typical properties of rapid set patching materials by generic family.

	Approx. working time (min.)	Approx. time to traffic (min.)	Compressive strength (MPa)		Abrasion loss (g)	Flexural strength (MPa)	Bond strength (MPa)		E (10^3 MPa)	α (10^{-6}/°C)	Linear shrinkage (%)
MATERIAL	@ 22°C	@ 22°C	@ 3 hr	@ 24 hr	@ 24 hr	@ 24 hr	@ 24 hr Dry PCC	@ 24 hr Wet PCC			
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 - 1.15*
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 4.11 Remedial methods for corrosion

METHOD	ADVANTAGES	DISADVANTAGES
Repair damage.	Relatively easy.	Repair must be compatible. Can accelerate corrosion.
Cathodic protection (impressed current).	Adjustable to control level of protection. New and post-construction use.	Requires maintenance. Can cause stray current corrosion.
Cathodic protection (sacrificial).	Limited control. No maintenance. Effective on corroding structure.	Limited use. Experimental use for atmospherically exposed concrete structures.
Corrosion inhibitors.	No maintenance.	Limited life. Must be applied during construction. Post construction use is experimental.
Chloride removal.	Removes chloride away from steel.	Experimental. Limited life. Not for prestressing steel.
Concrete membranes and sealants.	Effective on non-contaminated structures.	Not effective on corroding structure.
Shielding (stray current).	No maintenance. Passive.	Local effect only. Must identify source.
Dielectric isolation.	No maintenance. Passive.	Limited to bimetallic corrosion control.
Metal coating (barrier).	Isolates metal from electrolyte. Passive. Can be applied to atmospherically exposed structures.	Must be applied during construction for embedded steel. Can accelerate corrosion at depths.
Metal coating (sacrificial)	Some protection at defects. Can be applied to atmospherically exposed steel surfaces.	Limited life. Must be applied during construction of embedded steel.
Structural modification.	Passive. Permanent.	Limited suitability.
Material selection.	Passive. Permanent.	Limited suitability.
Environmental modification.	Passive. Permanent.	Limited suitability.

Table 4.12 Potential sources for stray current at nuclear power plants.

SOURCE	TYPE OF STRAY CURRENT
Electrical railway and mass transit systems.	Dynamic.
Impressed current cathodic protection systems.	Static.
High voltage direct current (HVDC) systems.	Dynamic/Static.
DC welding operation.	Dynamic.
Electrical grounding system.	Dynamic/Static.
Battery power supplies and battery recharging stations.	Dynamic.
DC motors such as elevators, cranes, remote controlled valves, etc.	Dynamic.
Industrial machinery.	Dynamic.
Electroplating operations.	Dynamic.
Telephone systems with very old technology.	Dynamic.
Electronic and instrumentation and control equipment.	Dynamic.
Railroad train switch signals.	Dynamic.
Geophysical effects such as lightning strikes, sun spots related electromagnetic interferences, telluric currents.	Dynamic.

5. TASK S.4 – QUANTITATIVE METHODOLOGY FOR CONTINUED SERVICE DETERMINATIONS

5.1 OBJECTIVE

The goal of this task is to develop a methodology to facilitate quantitative assessments of current and future structural reliability and performance of concrete structures in nuclear power plants, taking into account those effects that might diminish the ability of these structures to withstand future operating, extreme environmental or accident conditions. Specific objectives associated with accomplishing this goal are to (1) identify models to evaluate changes in strength of concrete structures over time in terms of initial conditions, service load history, and aggressive environmental factors; and (2) formulate a methodology to predict structural reliability of existing concrete structures during future operating periods from a knowledge of initial conditions of the structure, service history, aging, nondestructive condition assessment techniques, and inspection/maintenance strategies. When completed, the condition assessment methodology will be in a form such as summarized in Fig. 5.1 and will provide answers to the following questions, or at least frame the questions in a logical manner.

1. Which aging factors are particularly significant for concrete structures in terms of their future reliability?

2. Has the original strength of the structure degraded over time as a result of environmental stressors or aging factors?

3. What is the residual life of a structure and how would it respond to a design-basis event?

4. Which NDE techniques or in-situ strength measurements are most useful for demonstrating reliability of an existing structure?

5. What inspection procedures should be required and how frequently should they be conducted?

6. Which repair techniques are most effective in extending usable life and enhancing reliability?

7. What documentation (analysis or data) regarding condition assessment of Category I concrete structures should be provided to support an application to continue operation of a nuclear power plant past its original design period?

Figure 5.2 presents the Level 2 work breakdown structure for the quantitative methodology for continued service determinations task.

5.2 BACKGROUND

Once it has been established that a component has been subjected to environmental stressors or aging factors that may have resulted in deteriorating influences,* the effects of these influences must be related to a condition or structural reliability assessment, especially where the component is being considered for a continuation of its service past its initial licensing period. In order to continue the operation of NPPs, evidence should be provided

* Even a structure that exhibited no obvious signs of deteriorating influences might have to be evaluated for continued service considerations.

that the critical safety-related concrete structures in these facilities in their current condition are able to withstand potential future design events (over the continued service period) with a level of reliability adequate to meet requirements for protecting the public health and safety. Barriers to prediction of the future performance of a structure or component are listed in Ref. 5.1 and include the availability of (1) a systematic approach or methodology for treating the problem, (2) an effective mechanism for obtaining and reporting data on the actual in-service performance of materials, (3) knowledge of the mechanisms of degradation, (4) knowledge of the environmental factors causing degradation, (5) the ability to simulate or account for the synergism between degradation factors, and (6) mathematical models describing the material behavior in specific environments or applications. With the exception of the systematic approach or methodology, each of these barriers has been addressed previously.

A methodology for conducting such an assessment did not exist prior to initiation of this activity (Refs. 5.2 and 5.3). Advancements in structural reliability techniques have reached the point where it should be possible to make a quantitative evaluation of the durability (residual life) of a concrete structure based on knowledge of the condition of the structure when it was built, its service history, its present condition, and projected use during a time interval past its initial licensing period (Ref. 5.4). Such techniques would provide guidance to utilities and regulators alike regarding the technical data to be submitted in support of an application for continued service, current structures condition, and the need, if any, for future inspections or regular maintenance as a condition for granting a license for continued operation.

5.3 PLAN OF ACTION

This task will implement results obtained under Tasks S.2 (Chapter 3) and S.3 (Chapter 4) to develop a reliability-based evaluation methodology for concrete structures that will enable the factors that affect structural durability to be taken into account (Fig. 5.3). The methodology will integrate information on degradation and damage accumulation, environmental factors, and load history into a decision tool that will enable a quantitative measure of structural reliability and performance under projected future service conditions based on a condition assessment of the existing structure. The interaction of this task with the condition assessment methodology of Task S.3 (Fig. 4.4) is illustrated schematically in Fig. 5.4. The Level 3 work breakdown structure for this task is presented in Fig. 5.5.

5.3.1 Predictive Model Development (Subtask S.4.1)

Predictive or damage models identified or developed are being utilized in the formulation of procedures for assessing the current condition as well as the development of reliability-based future condition assessment of critical safety-related concrete components. The models enable the change in strength of concrete structures over time to be evaluated in terms of initial conditions, applied load history, and a parameterization of environmental stressors and aging factors. Since models developed under this subtask deal only with a subset of the parameters needed to predict future structural reliability, many of these models may have to be synthesized to encompass all significant factors and to account for any synergistic effects.

5.3.2 Data Assemblage/Implementation (Subtask S.4.2)

Pertinent data for use in establishing functional and performance requirements for critical components and in development of the predictive models are being assembled. Requirements used in the design, fabrication, erection, and testing of the critical components are identified through documents such as General Design Criteria 1, "Quality

Standards and Records;" 2, "Design Bases for Protection Against Natural Phenomena;" and 4, "Environmental and Missile Design Bases" of Appendix A, "General Design Criteria for Nuclear Plants" to 10CFR50, "Licensing of Production and Utilization Facilities" (Ref. 5.5). Additional information is obtained from codes such as American Concrete Institute (ACI) Standard 318, "Building Code Requirements for Reinforced Concrete" (Ref. 5.6), ACI 349, "Code Requirements for Nuclear Safety Related Structures" (Ref 5.7), and ACI 359, "ASME Section III—Division 2, Code for Concrete Reactor Vessels and Containments" (Ref. 5.8). Quantitative performance data on factors that can impact structural durability are being identified, including age, operating conditions, environmental stressors, design bases, material quality, and construction methods. Uncertainties associated with this data are being identified for use in the reliability-based methodologies in Task S.4.3. In-situ strength determinations by destructive or nondestructive evaluation methods are being assessed, with particular attention to quantifying measurement uncertainties associated with various condition assessment methods. The role of in-service inspection and maintenance in maintaining the reliability of structures is being addressed.

5.3.3 Methodology Development (Subtask S.4.3)

Stochastic models of the variables in the deterministic prediction models that have been identified and refined in Subtasks S.4.1 and S.4.2 are being developed, as well as techniques for analyzing current or future reliability of concrete structures. Structural loads arising from service, extreme environmental, and accident conditions are modeled as stochastic processes, as well as the environmental conditions under which the nuclear power plant must operate. The models of structural behavior used to predict response to various postulated conditions or strength changes over time also are random in nature (Ref. 5.9). Uncertainties in life prediction models, in strength, and in the ability of nondestructive evaluation methods to predict strength are being included in development of the methodology. The methodology being developed to predict future strength, or continued service period, reflects the stochastic characterization of the plant environment to determine probability distributions of structural capacity at some future time. Conversely, results being developed under this subtask make it possible to determine a probability distribution of additional usable life associated with maintaining a minimal structural capacity.

5.4 SUMMARY OF ACCOMPLISHMENTS

During the reporting period covered by this document, five journal articles (Ref. 5.10 –5.14), a NUREG report (Ref. 5.15), an ORNL/NRC letter report (Ref. 5.16), and two papers in conference proceedings (Ref. 5.17–5.18) were published. Prior activities have developed a probability-based methodology to estimate the strength degradation of a component and to evaluate the effect of periodic maintenance from a reliability point of view (Ref. 5.19). This methodology has been extended to consider cases where several defects or zones of damage may contribute to the reduction in strength of a structural member. A summary of recent activities is provided below.

5.4.1 Introduction

Methodologies have been developed for making condition assessments and service life predictions of new or existing reinforced concrete structures in nuclear power plants and hazardous waste depositories (Refs. 5.20). These methodologies integrate information on design requirements, material and structural degradation, damage accumulation, environmental factors, and nondestructive evaluation (NDE) technology into a decision tool

that provides a quantitative measure of structural reliability under projected future service conditions. This research has highlighted the need for quantitative modeling of strength degradation and the impact of NDE on in-service condition assessment.

5.4.2 Time-Dependent Reliability Analysis

Structural loads, engineering material properties, and strength degradation mechanisms are random in nature. Time-dependent reliability analysis methods provide a framework for performing condition assessments of existing structures and for determining whether in-service inspection and maintenance are required to maintain reliability and performance at the desired regulatory level.

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

$$M(t) = R(t) - S(t). \tag{5.1}$$

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

$$P_f(t) = P[M(t) < 0] = \int_0^\infty F_R(x)\, f_S(x)\, dx. \tag{5.2}$$

in which $F_R(x)$ and $f_S(x)$ are the probability distribution function of R and density function of S. Equation 5.2 provides one quantitative measure of structural reliability and performance, provided that P_f can be estimated and validated. The numerical evaluation of Eq. 5.2 and the development of supporting statistical data remain research challenges. However, significant progress has been made in this regard during the past several years.

For service life prediction and reliability assessment, one is more interested in the probability of satisfactory performance over some period of time, say (0,t), than in the snapshot of the reliability of the structure at a particular time provided by Eq. 5.2. Indeed, it is difficult to use reliability analysis for engineering decision analysis without having some time period (say, an in-service maintenance interval) in mind. The probability that a structure survives during interval of time (0,t) is defined by a reliability function, L(0,t). If, for example, n discrete loads $S_1, S_2, \ldots, S_n$ occur at times $t_1, t_2, \ldots, t_n$ during (0,t), the reliability function becomes,

$$L(0,t) = P[R(t_1) > S_1 \ldots, R(t_n) > S_n]. \tag{5.3}$$

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

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

$$h(t) = -d \ln L(0,t)/dt. \tag{5.4}$$

Solving for L(0,t) yields,

$$L(0,t) = \exp\left[-\int_0^t h(x)dx\right]. \tag{5.5}$$

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

$$P\left[T_f \le t + \Delta t \mid T_f > t\right) = 1 - \exp\left[-\int_t^{t+\Delta t} h(x)dx\right]. \tag{5.6}$$

The hazard function for pure chance failures (case 1 in Section 5.4.3) is constant. When structural aging occurs and strength deteriorates, h(t) characteristically increases with time. In-service inspection and maintenance impact the hazard function, causing it to change discontinuously at the time that in-service inspection is performed. The main difference between time-dependent reliability of undegrading and degrading structural components can be characterized by their hazard functions. Much of the challenge in structural reliability analysis involving deteriorating structures lies in relating the hazard function to specific degradation mechanisms, such as corrosion.

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

$$L(0,t) = \int_0^{\infty} \exp\left(-\lambda t\left[1 - t^{-1}\int_0^t F_S(rg)\,dt\right]\right) f_R(r)\,dr. \tag{5.7}$$

in which $f_R(r)$ is the probability density function of initial strength, R(0), and g(t) equals the mean of R(t)/R(0), a function describing the degradation of strength in time (see Fig. 5.6). The limit state probability, or probability of failure during (0,t), can be determined as F(t) = 1 - L(0,t); F(t) is not the same as $P_f(t)$ in Eq. 5.2.

5.4.3 Service Life Predictions for Reinforced Concrete Slab

Time-dependent reliability concepts are illustrated with a simple example of a concrete slab drawn from recent research on aging of concrete structures in nuclear plants (Refs. 5.13 and 5.14). This slab was design using the requirements for flexure strength found in ACI Standard 318 (Ref. 5.6):

$$0.9\,R_n = 1.4\,D_n + 1.7\,L_n. \tag{5.8}$$

in which R_n is the nominal or code resistance, and D_n and L_n are the code-specified dead and live loads, respectively. The strength of the slab changes in time, initially increasing as the concrete matures and then decreasing due to (unspecified) environmental attack. This situation is illustrated conceptually by the sample functions r(t) and s(t) for strength and load in Fig. 5.6. The behavior of the resistance over time must be obtained from mathematical models describing the degradation mechanism(s) present.

Figure 5.7 presents a comparison of limit state probabilities for intervals (0,t) for t ranging up to 60 years. Three cases are presented (see Fig. 5.6): (1) no degradation in strength, i.e., R(t) = R(0), a random variable [this case is analogous to what has been done in probability-based code work to date (Ref. 5.21)]; (2) R(t) initially increasing with concrete maturity and then degrading; and (3) R(t) degrading linearly over time to 90% of its initial strength at 40 years. The statistics used in the illustrations that follow are

summarized in Table 5.1. The basis for these statistics is given elsewhere (Ref. 5.12). Neglecting strength degradation entirely in a time-dependent reliability assessment can be quite unconservative, depending on the time-dependent characteristics of strength.

5.4.4 Service Life Predictions for Reinforced Concrete Shear Wall

As shown above, the failure probability of a structural component or system under stationary random loading can be evaluated as a function of time if the strength degradation and the probabilistic characteristics of the initial strength are known. In previous papers (Refs. 5.13 and 5.14) a probability-based method to estimate the strength degradation of a component and to evaluate the effect of periodic maintenance from a reliability point of view was provided. In the method developed, 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 combined effects of expansive aggregate reactions at several points along a given cross section of the wall. The evaluation of the (random) residual strength of the wall requires that the cumulative effect of defects in a cross section be considered. Recent research has provided a method whereby the impact of randomly occurring multiple defects on structural capacity can be considered (Ref. 5.16). Some results are summarized in the following.

The wall considered is a low-rise wall with $h_w/\ell_w = 1$ and is subjected to vertical load, D, which is uniformly distributed on the top of the wall and in-plane lateral load, V, which is concentrated at the top of the wall. The shear strength of concrete walls can be estimated from empirical models (Refs. 5.6 and 5.22). 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 (Refs. 5.23–5.25) determines the ultimate shear strength as the sum of the forces sustained by a truss mechanism, V_t, and by an arch mechanism, V_a. It is assumed that the wall fails if all the reinforcing bars yield in tension and the concrete arch crushes in compression. According to the lower bound theorem of plasticity (Ref. 5.26), this approach provides a conservative estimate of the shear strength. These models have been modified for the reliability analysis of a degrading concrete low-rise shear wall (Ref. 5.16). Figure 5.8 shows that the strength predicted by this method compares well to experimental tests of low-rise shear walls.

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

$$G(t) = \frac{V_t + V_a(t)}{V_{u0}} = \frac{V_t + G_a(t) \cdot V_a(0)}{V_{u0}}, \tag{5.9}$$

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

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

$$M_u = T_s\left(\frac{b}{2} - d_c\right) + C_c\left(\frac{b}{2} - k_2 c_u\right) + C_s\left(\frac{b}{2} - d_c\right) \tag{5.10}$$

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

Illustration. For illustration, assume that

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

- The wall is designed for in-plane shear based on the current design requirement (Ref. 5.7).

$$0.9R_n = E_{ss} \tag{5.11}$$

  in which R_n is the nominal shear strength and E_{ss} is the structural action due to safe-shutdown earthquake. The statistical characteristics of the shear strength and the earthquake load are shown in Table 5.1. It is assumed that $E_{ss} = 3D =$ 3.21MN.

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

- The defect intensity is modeled as,

$$Y(t) = C(t - T_I)^2 \tag{5.12}$$

  in which C is a time-invariant random variable described by a lognormal distribution with mean value, μ_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 (Ref. 5.27). The specified yield strength of the reinforcement is 414 MPa and the mean is 465 MPa.

- Compressive strength of the concrete increases during the first 10 years but does not change thereafter. The mean compressive strength (in units of MPa) at time t is evaluated by (Ref. 5.28)

$$E[f_c(t)] = \begin{cases} 15.51 + 3.95 \ln t, & t < 10 \text{ years} \\ 47.91, & t \geq 10 \text{ years} \end{cases} \tag{5.13}$$

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

The mean degradation in shear strength of the wall in which expansive aggregate reactions occur in the concrete is illustrated in Fig. 5.9, assuming $v_u = 0.1/\text{m}^2/\text{year}$. 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. 5.9 are presented in Figs. 5.10 and 5.11, respectively. The increase in failure probability due to the strength degradation is small because of the large variability in earthquake load intensity (Ref. 5.29). 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. 5.12. 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.

5.4.5 Condition Assessment and In-Service Inspection

Forecasts of reliability of the type illustrated in Fig. 5.7 enable the analyst to determine the time period beyond which the desired reliability of the structure cannot be ensured. At such a time, the structure should be inspected. Intervals of inspection and maintenance that may be required as a condition for continued operation can be determined from the time-dependent reliability analysis. In-service inspection and maintenance are a routine part of managing aging and deterioration in many engineered facilities; work already has been initiated to develop policies for offshore platforms (Ref. 5.30) and aircraft (Ref. 5.31) using probabilistic methods.

When a structure is inspected and/or repaired, something is learned about its in-service condition that enables the probability distribution of strength to be updated. The density function of strength, based on prior knowledge of the materials in the structure, construction and standard methods of analysis, is indicated by $f_R(r)$. Scheduled inspection, maintenance and repair cause the characteristics of strength to change; this is denoted by the (conditional) density $f_R(r|B)$, in which B is an event dependent on in-service inspection. The information gained from inspection usually involves several structural variables including dimensions, defects, and perhaps an indirect measure of strength or stiffness. If these variables can be related through event B, then the updated density of R following in-service inspection is,

$$f_R(r|B) = P[r < R \le r + dr, B]/P[B] = c\,K(r)\,f_R(r), \tag{5.14}$$

in which $f_R(r)$ is termed the prior density of strength, K(r) is denoted the likelihood function, and c is a normalizing constant. The time-dependent reliability analysis then is re-initialized following in-service inspection/repair using the updated $f_R(r|B)$ in place of $f_R(r)$. The updating causes the hazard function (e.g., Fig. 5.11) to be discontinuous.

Uncertainties in methods of in-service inspection/repair affect the density $f_R(r|B)$. Using a combination of methods usually is more effective from a reliability point of view than using one method. When there are limited resources, it often is most effective to select a few safety-critical elements and concentrate on them (Refs. 5.12 and 5.32). Optimal intervals of inspection and repair for maintaining a desired level of reliability can be determined based on minimum life cycle expected cost considerations. Preliminary investigations of such policies have found that they are sensitive to relative costs of inspection, maintenance, and failure (Ref. 5.29). If the cost of failure is an order (or more) of magnitude larger than inspection and maintenance costs, the optimal policy is to inspect at nearly uniform intervals of time. However, additional research is required before such policies can be finalized as part of an aging management plan.

5.5 PLANNED ACTIVITIES

An evaluation of the importance of aging of concrete structures on seismic probabilistic risk assessment or seismic margin analysis of an existing nuclear power plant will be completed. This activity is intended to provide some initial data to answer the question of whether structural aging has a significant impact on overall plant risk during a projected service life or service life extension. The idea of using a seismic probabilistic risk assessment for this purpose derives from the observation that structural components generally play a negligible role in an internal events risk assessment, and thus structural aging would have little impact on plant risk from internal initiating events. On the other hand, structural performance is relatively more important in an external events analysis, and may impact several plant safety systems simultaneously (common-cause failures). The results of this assessment will be provided in a report.

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5.7 MILESTONE STATEMENT AND SCHEDULE

The statement and schedule for the milestones in Task S.4 are given in the following charts.

MILESTONE STATEMENT AND SCHEDULE

TASK: S.4 QUANTITATIVE METHODOLOGY FOR CONTINUED SERVICE DETERMINATIONS

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.4 QUANTITATIVE METHODOLOGY FOR CONTINUED SERVICE DETERMINATIONS																							
S.4.1 Predictive Model Development																							
A. Complete Implementation of Strength Degradation Models and Evaluate Impact on Component Reliability	▲																						
B. Complete Evaluation of Impact of System reliability Considerations on Time-Dependent Structural Reliability	▲																						

MILESTONE STATEMENT AND SCHEDULE

TASK: S.4 QUANTITATIVE METHODOLOGY FOR CONTINUED SERVICE DETERMINATIONS

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.4.2 Data Assemblage/Implementation																							
A. Complete Assemblage of Data Related to Design Requirements, Material Behavior, Construction Parameters, etc.																							
(activity completed)																							
B. Complete Assemblage of Data Related to Service History Effects																							
(activity completed)		b																					
C. Complete Evaluation of Data Uncertainities and the Impact on the Reliability Analysis																							
a. Complete Assessment of Effectiveness of Destructive and Nondestructive In Situ Strength Evaluations in Enhancing Time-Dependent Reliability and Performance																							
(activity completed)																							
b. Complete Initial Evaluation of How Managed Aging (ISI and Maintenance) in Engineered Facilities Affects Forecasted Time-Dependent Reliability																							

MILESTONE STATEMENT AND SCHEDULE

TASK: S.4 QUANTITATIVE METHODOLOGY FOR CONTINUED SERVICE DETERMINATIONS

SUBTASK/MILESTONE	FY 93				FY 1994												FY 95				FY 96	FY 97	BEYOND FY 97
	1	2	3	4	O	N	D	J	F	M	A	M	J	J	A	S	1	2	3	4			
S.4.3 Methodology Development																							
A. Complete Paper on Probabilistic Methods for Condition Assessment and Life Prediction of Concrete Structures in Nuclear Power Plants		a,b										c						d					
(activity completed)																							
B. Issue Reports on Reliability-Based Condition Assessments of Nuclear Power Plant Concrete Structures																							
a. Issue Report on Condition Assessment and Reliability-Based Life Prediction																							
(activity completed)																							
b. Complete Report on Probability-Based In-Service Inspection and Condition Assessments																							
c. Complete Report on Methodology for Making Reliability-Based Future Condition Assessments of Degrading Shear Walls																							
d. Complete Draft Report on Reliability Basis for Evaluation of Current Condition and Future Performance																							

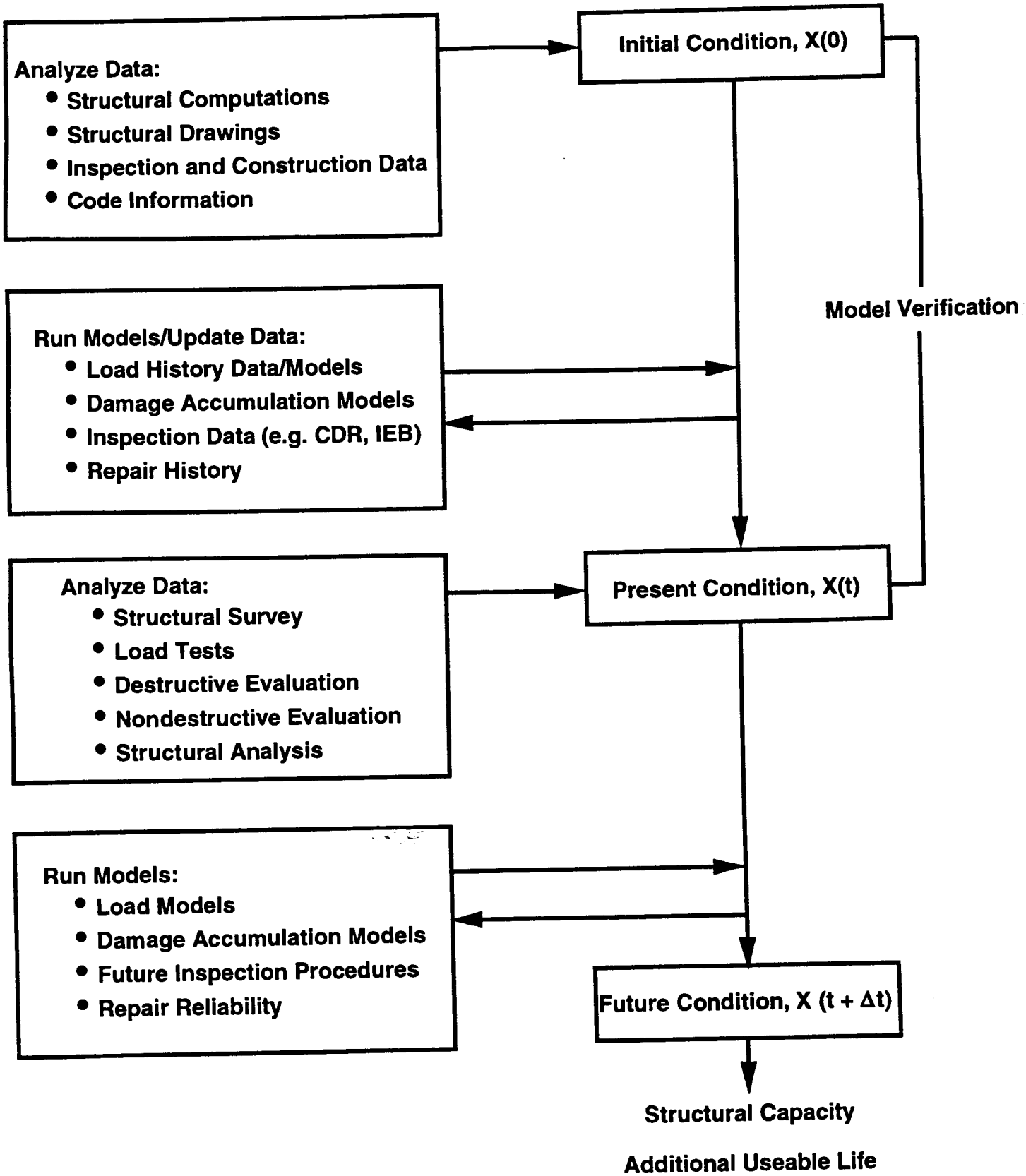

Fig. 5.1. System reliability approach to condition assessment.

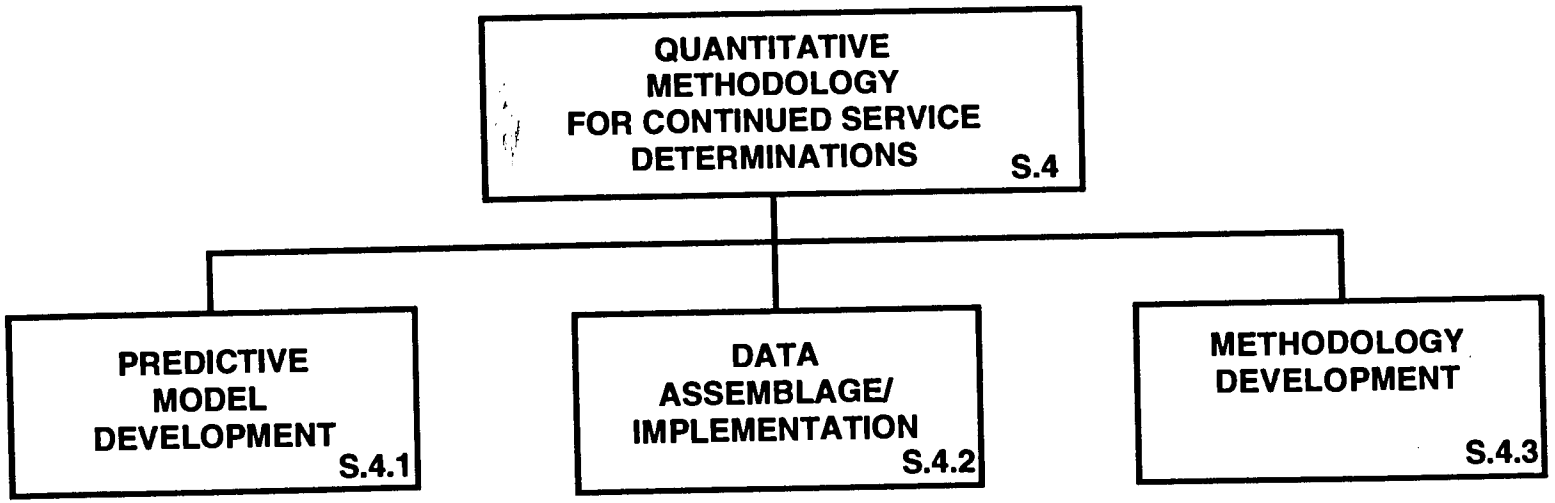

Fig. 5.2. Level 2 work breakdown structure for SAG Task S.4: Quantitative Methodology for Continued Service Determinations.

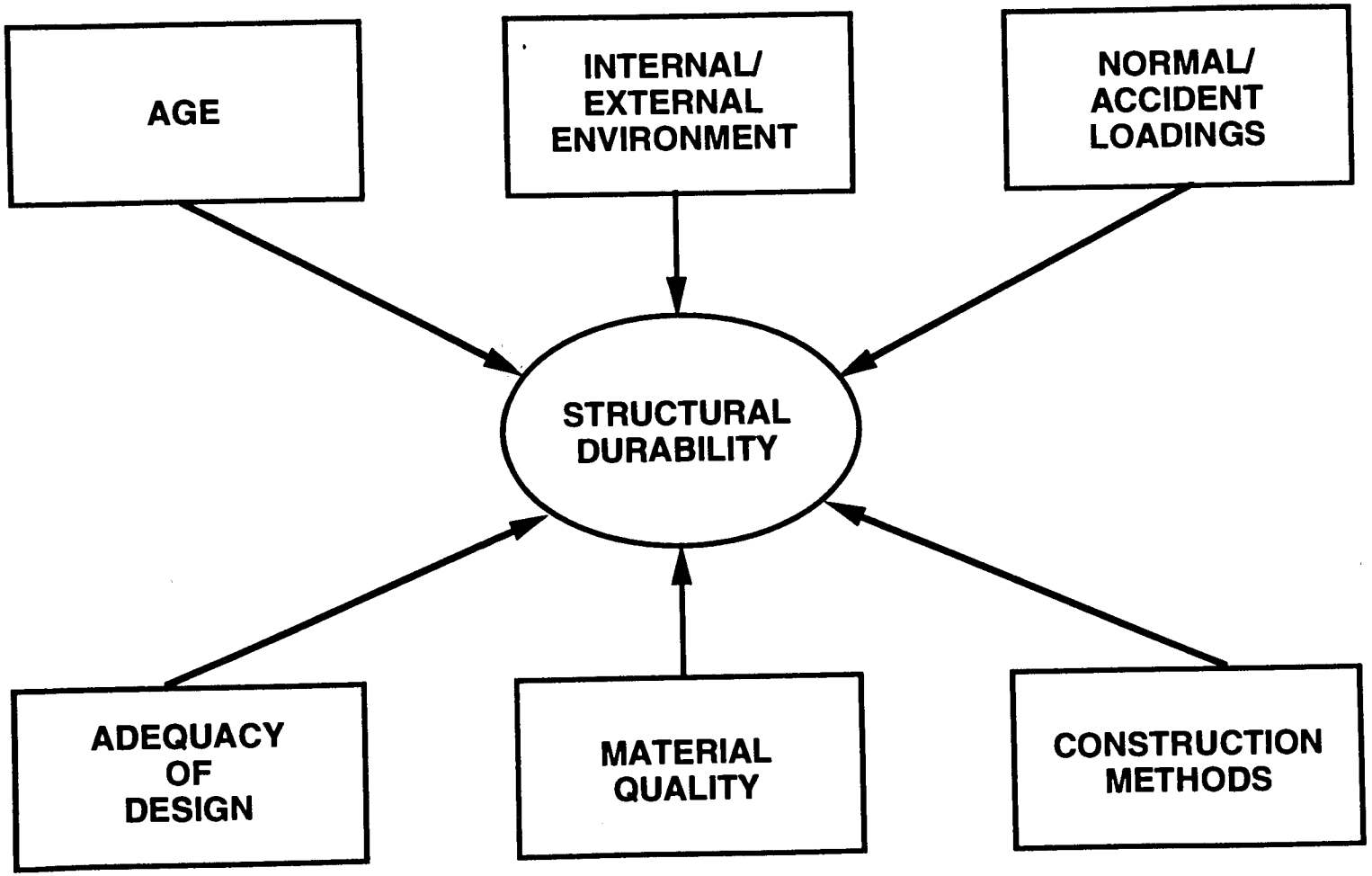

Fig. 5.3. Structural durability is based on a systematic evaluation of several factors, including synergism of the various factors.

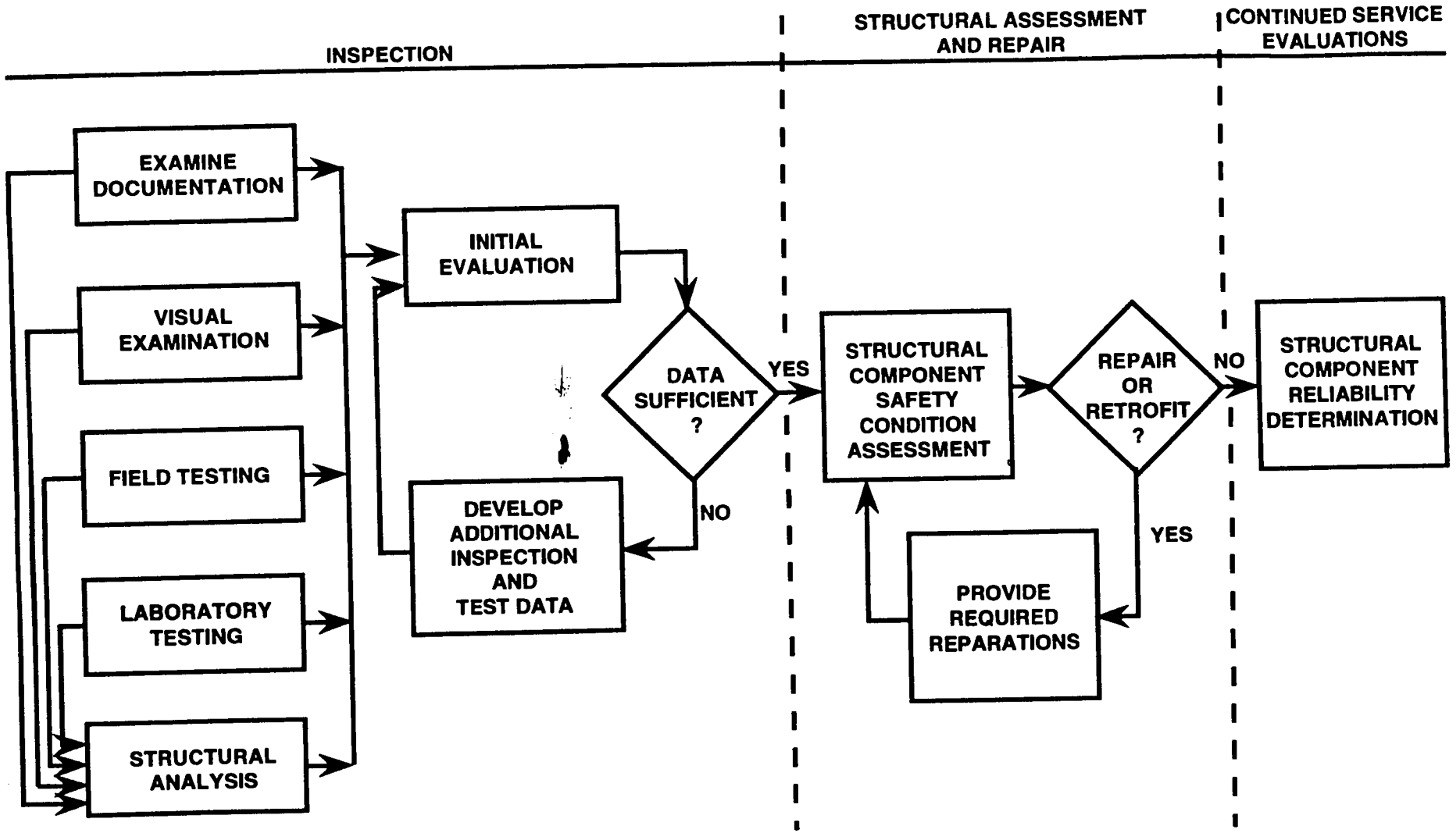

Fig. 5.4 LWR concrete component evaluation methodology. Source: Adaptation of a procedure presented in T. L. Rewerts, "Safety Requirements and Evaluation of Existing Buildings," *Concr. Int'l.* 7(4), American Concrete Institute, Detroit, April 1985.

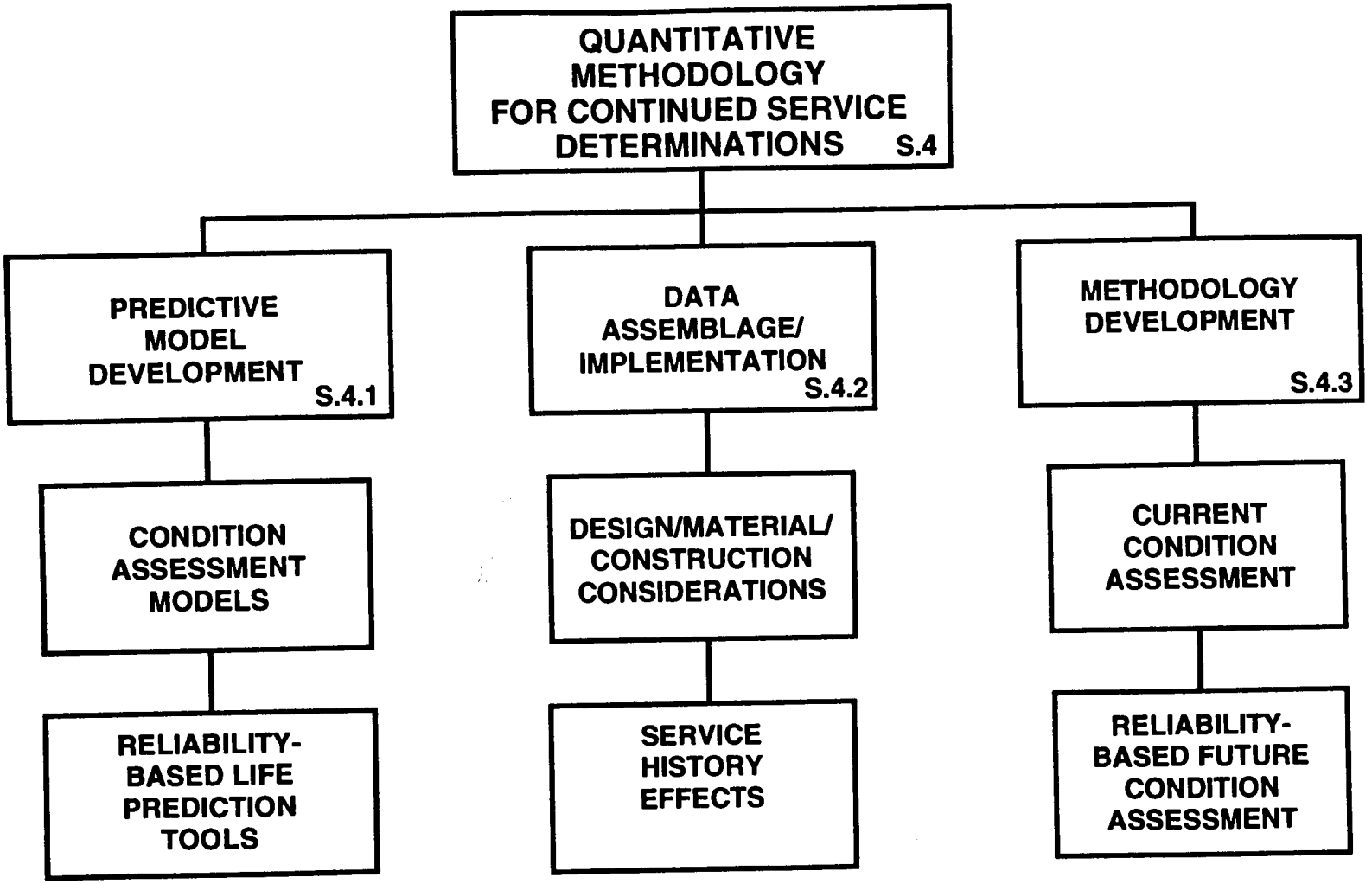

Fig. 5.5. Level 3 work breakdown structure for Task S.4: Quantitative Methodology for Continued Service Determinations.

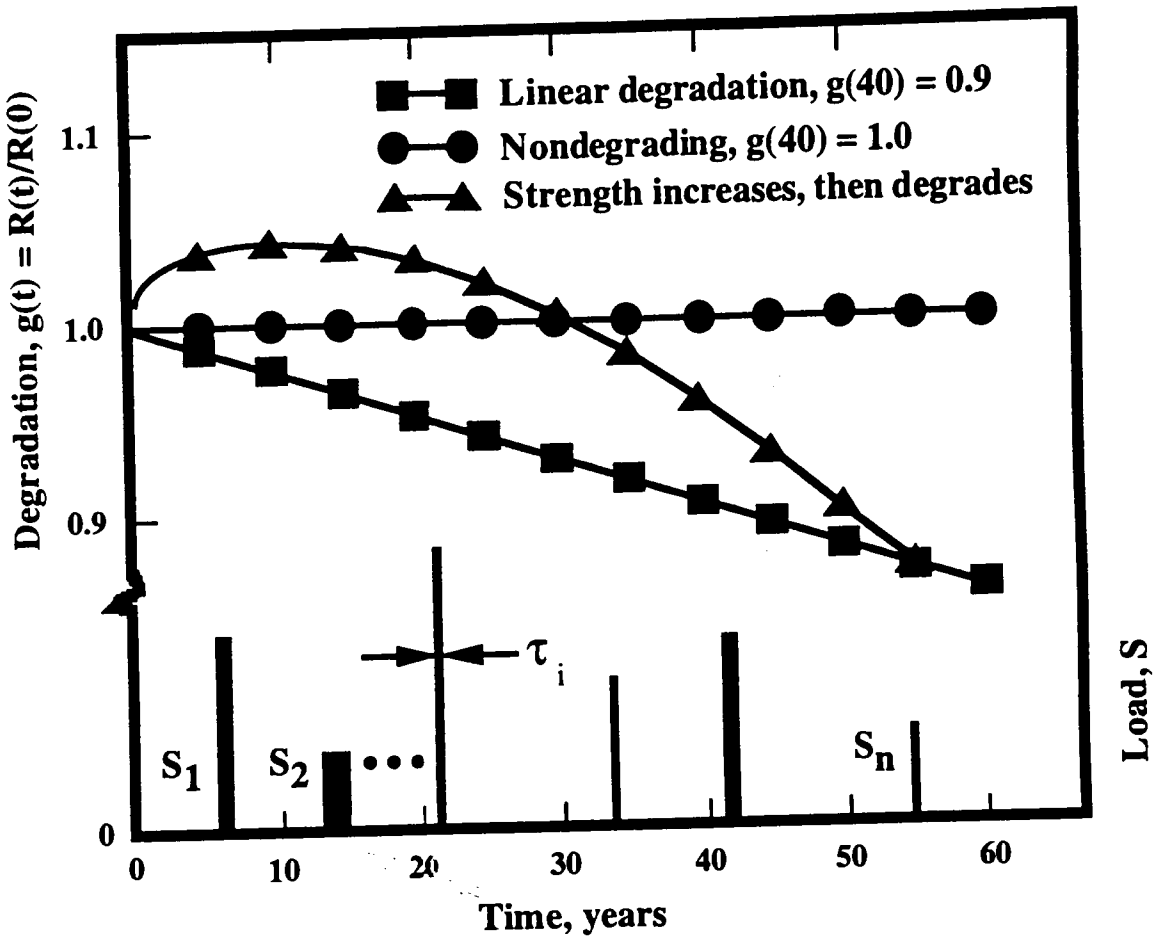

Fig. 5.6 Mean degradation functions of one-way slab.

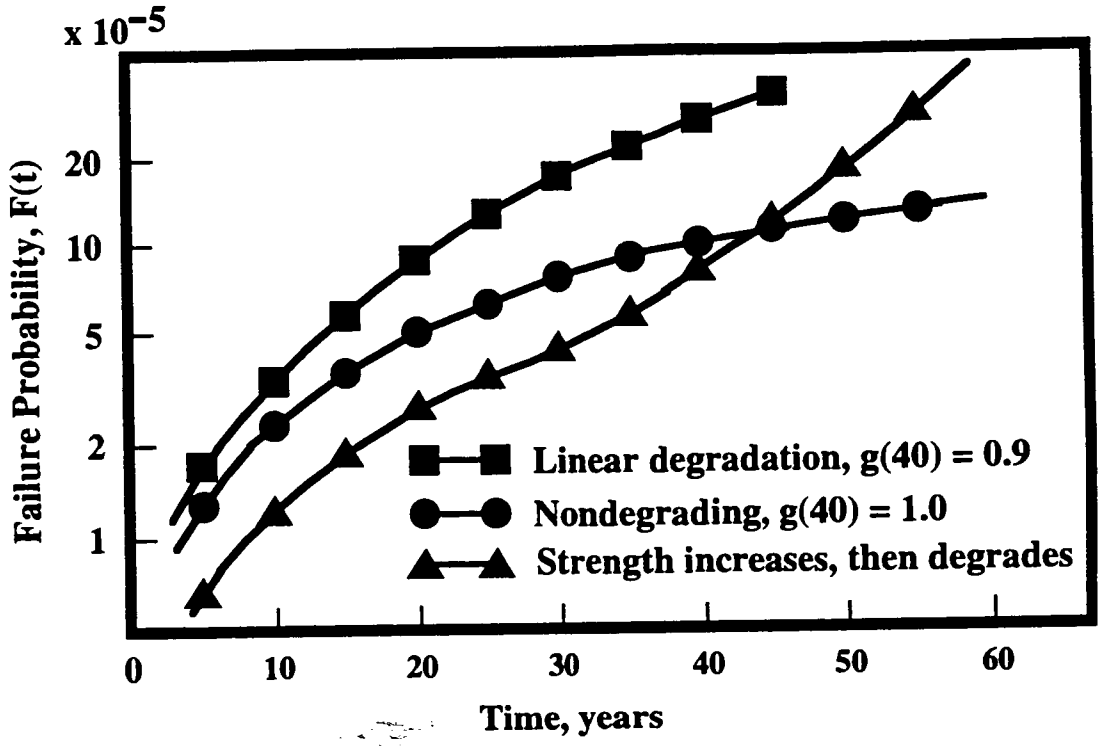

Fig. 5.7 Failure probability of a one-way slab.

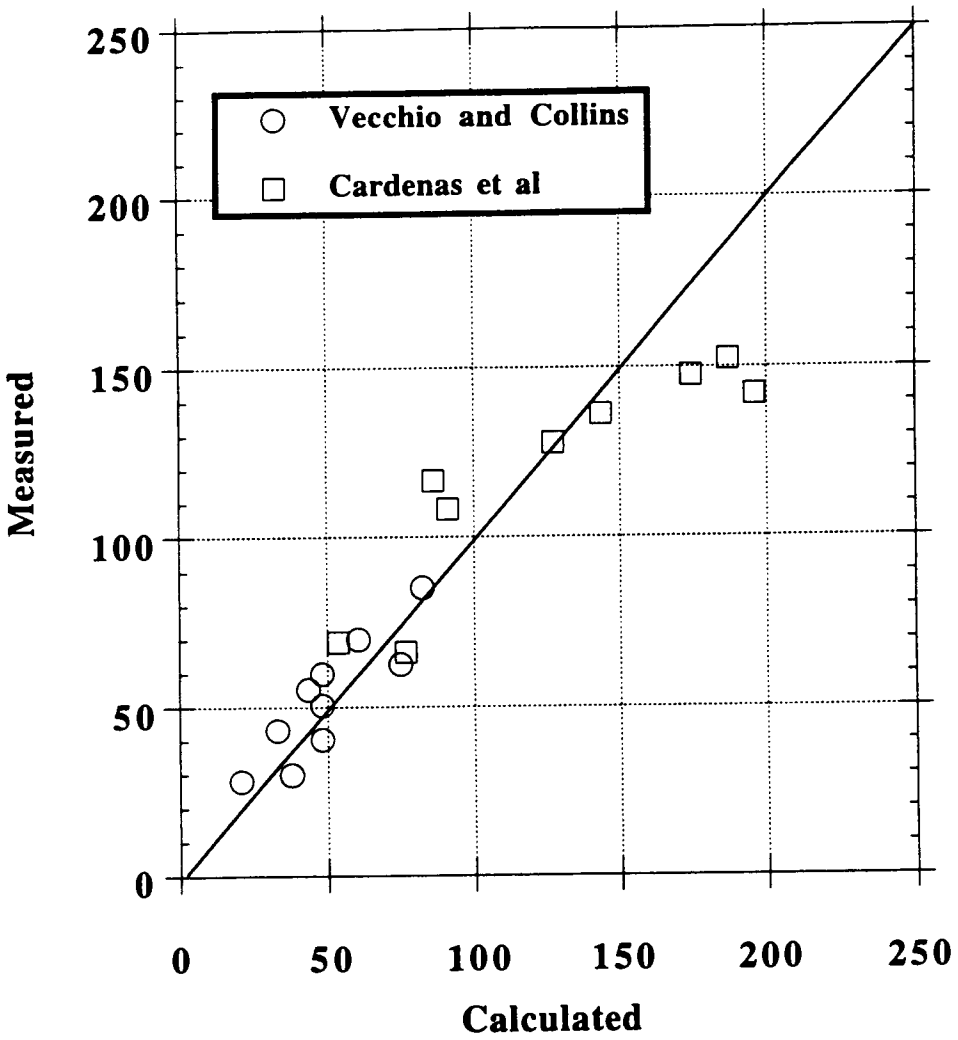

Fig. 5.8 Comparison of measured and calculated shear strength of low-rise reinforced concrete wall.

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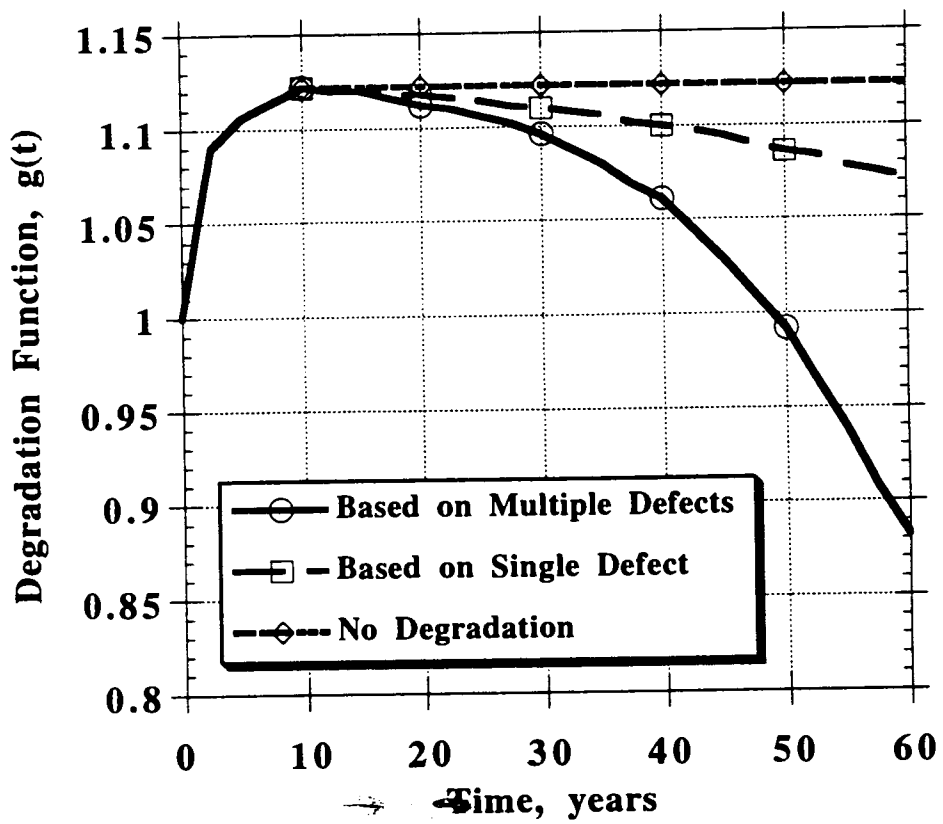

Fig. 5.9 Mean degradation function of wall in shear without repair ($\nu_u = 0.1/m^2/yr$).

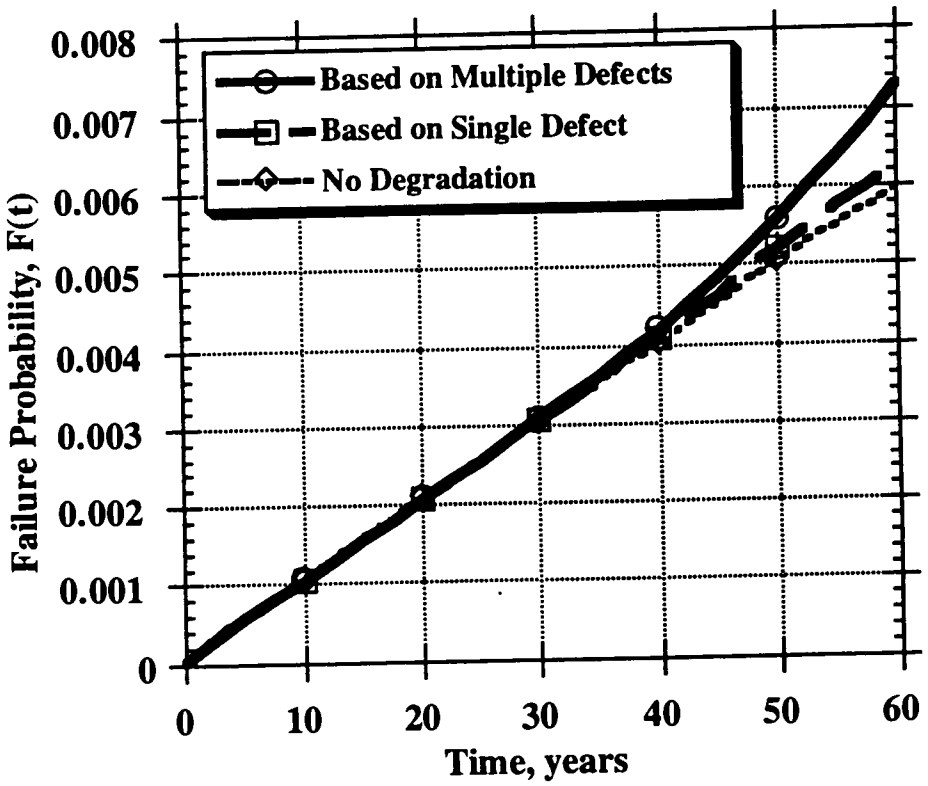

Fig. 5.10 Failure probability of wall in shear without repair ($\nu_u = 0.1/m^2/yr$).

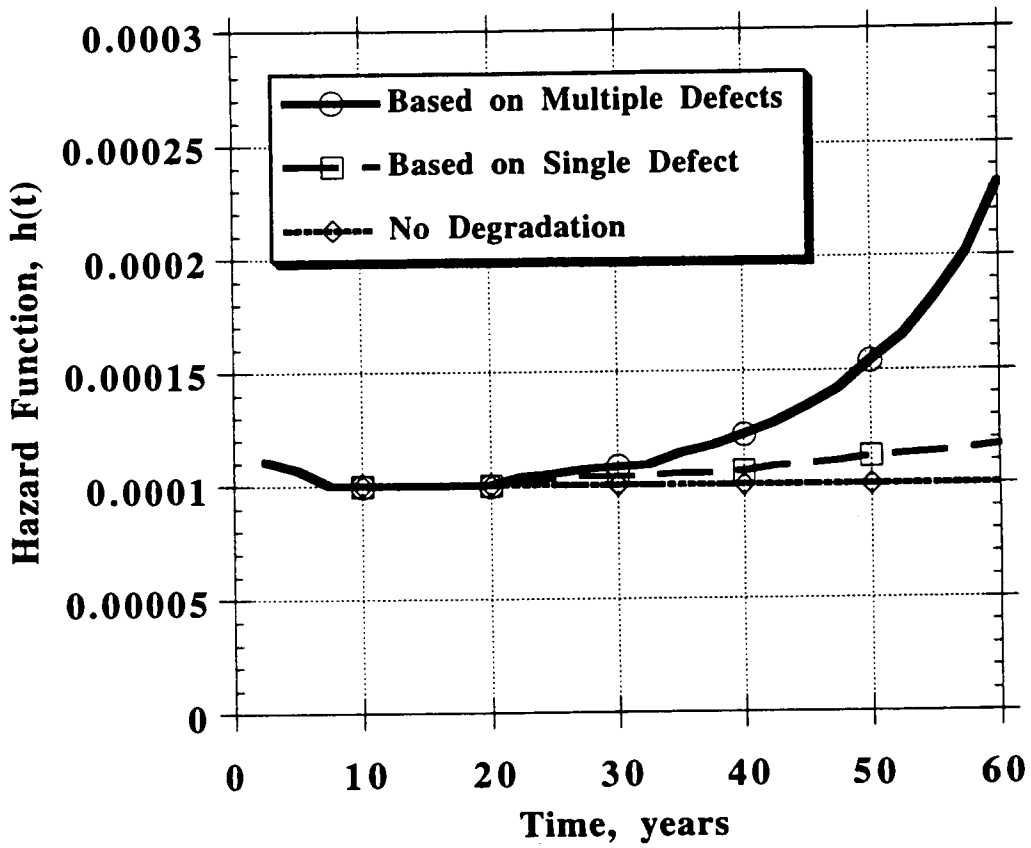

Fig. 5.11 Hazard function of wall in shear without repair.

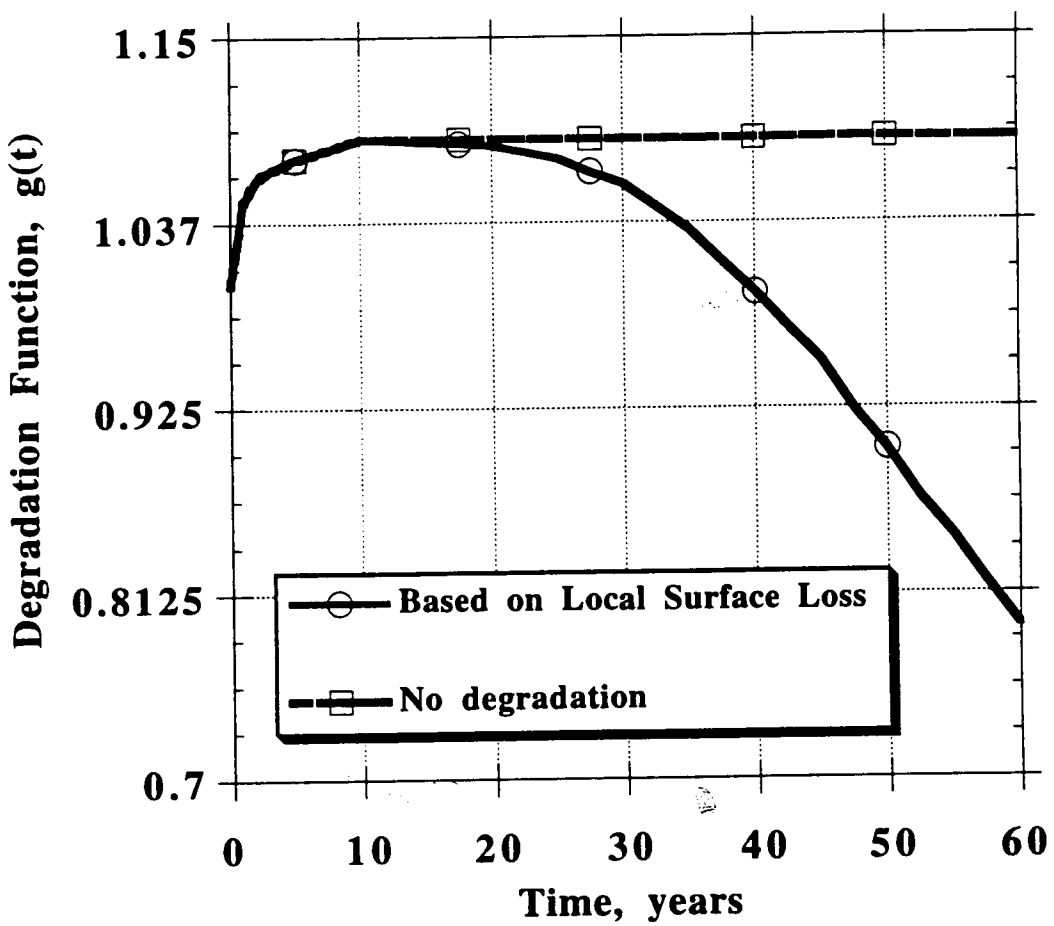

Fig. 5.12 Mean degradation function of wall in flexure/compression without repair, ($\nu_u = 0.1/m^2/yr$).

Table 5.1 Statistical properties of strength and load.

Parameter	Rate of Occurrence	Duration	Mean	Coefficient of Variation	Probability Density Function
Flexure Strength	—	—	1.12 M_n	0.14	Lognormal
Shear Strength	—	—	1.7 V_n	0.18	Lognormal
Dead Load	—	—	1.0 D_n	0.07	Normal
Live Load	0.5/yr	3 mo.	0.4 L_n	0.50	Type I
Earthquake Load	0.05/yr	30 sec.	0.08 E_{sse}	0.85	Type II

APPENDIX A

SUMMARY OF ACCOMPLISHMENTS

Task S.1 – Program Management

- Program Planning and Resource Allocation – Subtask S.1.1
  - Issued two five-year program plans (ORNL/NRC/LTR-90/1, ORNL/NRC/LTR-92/3)
  - Participated in nine program briefings with NRC personnel
  - Implemented thirteen subcontracts (ten completed)
  - Participated in ANS topical meetings on Plant License Renewal (Orlando) and Nuclear Power Plant Life Extension (Snowbird), Second International Seminar on Containment of Nuclear Reactors (Lausanne), Fourth (Alexandria) and Fifth (Washington, D.C.) Workshops on Containment Integrity, American Power Conference (Chicago), ASME Power Plant Systems/Components Aging Management and Life Extension (San Diego, New Orleans, and Denver), 3rd Int'l Seminar on Containment of Nuclear Reactors (UCLA), NRC Nuclear Power Plant License Renewal Workshop (Reston), SMiRT Conferences (Lausanne, Anaheim and Stuttgart), National Concrete Engineering Conference (Chicago), ASCE Materials Engineering Conference (Atlanta), U.S. Army Corps of Engineers Workshop on Deterioration and Condition Assessment of Concrete Navigational Structures (Washington, D.C.), National Research Council Transportation Research Board 71st Annual Meeting (Washington, D.C.), U.S. National Standard for the Exchange of Product Model Data Conference (Gaithersburg), Symposia on Building Databases (Gaithersburg), Symposium on Integrated Knowledge Systems for High Performance Construction Materials and Systems (Gaithersburg), Workshop on Integrated Knowledge Systems for High Performance Concretes (Gaithersburg), and Workshop on Concrete Performance Modeling for Low-Level Waste Disposal (Gaithersburg).
  - Participated in short courses on nuclear power reactor safety (MIT), containment in-service inspection and testing (ASME), General Electric Boiling Water Reactor Technology (USNRC), and Westinghouse Pressurized Water Reactor Technology (USNRC)
  - Completed investigative trip to Fort Calhoun Nuclear Power Plant to participate in assessment of impact on structural integrity of containment building of occurrence of post-tensioning system grease leakage

Task S.1 – Program Management (cont.)

- Program Monitoring and Control – Subtask S.1.2
  - Presented program overview papers at Sixteenth, Eighteenth, Nineteenth, Twentieth, and Twenty-First WRSMs
  - Presented program overview paper at NRC Aging Research Information Meeting
  - Completed technical reviews and provided comments to NRC on NUMARC license renewal technical reports on PWR containments and Class 1 Structures, and on Chap. 4.0 "Civil Structures" of SRP – License Renewal
  - Implemented annual technical reporting system and issued four reports
  - Presented program overviews to ASME Advisory Committee on Technical Issues (Nashville), New Production Reactors Working Group on Standards (Washington, D. C.) and Atomic Safety Licensing Board (Gatlinburg)
  - Completed technical reviews of INEL reports "Life Assessment Procedures for Metallic Containments (Chapter 5)" (NUREG/CR-5314), and "Quantification of Degradation Damage in Light Water Reactor Concrete Containments" (INEL-12-6002, Rev. 0)

Task S.1 – Program Management (cont.)

- Documentation and Technology Transfer – Subtask S.1.3
  - Coordinated activities with other government agencies-related programs
    - Low-level radioactive waste program (NRC)
    - Modular high-temperature gas-cooled reactor (NRC)
    - Nuclear plant aging research program (NRC)
    - New production reactors working group (DOE)
  - Twenty-nine technical presentations at national or international meetings of which eighteen were program overviews
  - Twenty-four letter or NUREG reports issued (three in final review)
  - Thirty-one papers prepared for domestic and international conferences
  - Ten journal articles published with four related to program overviews
  - Ten foreign trip reports issued
  - Established informal technology exchange with four international technical organizations
    - Federation Internationale de la Precontrainte (FIP)
    - Int'l Union of Testing and Research Laboratories for Materials and Structures (RILEM)
    - Int'l Council for Building Research Studies and Documentation (CIB)
    - Commission of European Communities (CEC)
  - Contacted 114 foreign organizations in 27 countries to pursue informal exchange of technology
  - Participated in technical society committee activities
    - American Concrete Institute (Service Life Prediction, Radioactive and Hazardous Waste Management, Concrete Materials Property Data Base, Fracture Mechanics, Composite Concrete/Steel High-Pressure Vessels)
    - American Society of Mechanical Engineers (Section XI Working Group on Plant Life Extension, Working Group on Concrete Pressure Components, Subgroup on Containments)
    - RILEM (Damage Classification of Concrete Structures, Techniques for Prediction of Service Life)
    - FIP (Commission on Prestressed Concrete Pressure Vessels and Containments: Working Group 2 - Containment Structures and Working Group 3 - Non-nuclear PCPVs)
  - Presented program overviews to seven domestic and twelve international organizations
  - Contacted 113 domestic organizations: governmental (24), universities (18), national organizations (21), consulting/engineering groups (36), and utility organizations/utilities (14) to pursue exchange of information
  - Completed ten trips to foreign research facilities to coordinate activities and exchange technology
  - Presented lectures on "Concrete Material Systems" and "Containments for Nuclear Power Plants" at IAEA Interregional Training Course on Safety Aspects of Aging and Maintenance in Nuclear Power Plant Operation
  - Participated in IAEA co-ordinated research program on Management of Aging of Concrete Containment Building
  - Developed new RILEM Technical Committee "Methodology for Life Prediction of Concrete Structures in Nuclear Power Plants"

Task S.2 – Materials Property Data Base

- Structural Materials Information Center Formulation/Implementation – Subtask S.2.1
  - Presented papers at Eighteenth WRSM and NRC Aging Research Information Meeting
  - Completed review and assessment of domestic and foreign materials property data bases (ORNL/NRC/LTR-89/3)
  - Completed plan for development of Structural Materials Information Center (SMIC) (ORNL/NRC/LTR-89/8)
  - Implemented plan and provided first issue of SMIC (ORNL/NRC/LTR-90/22)
    - Selected software for electronic data base and formatted to handle composite material systems, i.e., concrete
    - Developed format for Structural Materials Handbook and Structural Materials Electronic Data Base
    - Established data and information requirements for concrete, metallic reinforcement, prestressing tendon, and structural steel materials
  - Developed handbook pages and electronic data base files for 143 concrete-related materials (128 portland cement concretes, 12 metallic reinforcements, 1 prestressing tendon, and 2 structural steels) and entered into the Structural Materials Information Center
  - Expanded SMIC to include nonconcrete materials (rubber, 1 entry)
  - Completed report on SMIC and its potential applications (ORNL/NRC/LTR-92/8)
  - Completed draft report summarizing materials contained in the SMIC
  - Completed draft report addressing formulation of a customized data base for structural materials
  - Technical presentations at five national and one international conference describing SMIC
  - Published one journal article describing SMIC

- Data Collection – Subtask S.2.2
  - Completed procurement and testing of aged concrete samples from ten nuclear facilities
  - Completed report presenting results of five-year compressive strength testing of variable fly ash content high-strength concretes (ORNL/NRC/LTR-89/12)
  - Completed subcontracts with Construction Technology Laboratories, Inc., to provide 30-year data from long-term study of performance of cement in concrete (ORNL/NRC/LTR-91/26) and testing additional specimens ($t > 35$ years) (ORNL/NRC/LTR-92/26)
  - Completed subcontract with Sargent & Lundy Engineers to provide baseline material property information on concrete materials utilized in Commonwealth Edison Companies (CECo) nuclear power plants and to assist in setting up a material sampling program with CECo
  - Completed report presenting results obtained from testing aged concretes obtained from several United Kingdom nuclear power stations (Taywood Engineering Subcontract Report 1303/92/6037)
  - Completed report presenting results obtained from surveillance data for several United Kingdom nuclear power stations (Taywood Engineering Subcontract Report 1302/92/5957)

Task S.2 – Materials Property Data Base (cont.)

- Material Behavior Modeling – Subtask S.2.3
  - Exchanged technology with National Institute of Standards and Technology (NIST) on modeling of concrete material behavior and permeability studies
  - Exchanged information with Technical Research Center of Finland, Taywood Engineering Laboratories, British Cement Association, British Research Establishment, AEA Technology, CEB, RILEM, University of Toronto, and Danish Road Directorate on concrete modeling and service life prediction
  - Exchanged information with CIB/RILEM and ACI committees on damage classification of concrete structures
  - Participated in CANMET/ACI Conference on Durability of Concrete (Montreal) and Fifth International Conference on Durability of Building Materials and Components (Brighton)
  - Published NIST report evaluating models for use in predicting remaining service life of concrete and reviewing accelerated aging techniques and tests (NISTIR 4712)
  - Completed report presenting results of structural concrete condition surveys conducted at several United Kingdom nuclear power stations (Taywood Engineering Subcontract Report 1303/92/6163)

Task S.3 – Structural Component Assessment/Repair Technology

- LWR Critical Concrete Component Classification – Subtask S.3.1
  - Completed development of concrete component classification and rating system
  - Completed development of degradation factor classification and rating system
  - Published report on structural aging assessment methodology for concrete structures in nuclear power plants (ORNL/NRC/LTR-90/17)

- NDE/Sampling Inspection Technology – Subtask S.3.2
  - Published report reviewing and assessing in-service inspection techniques and methodologies for application to concrete structures in nuclear power plants (ORNL/NRC/LTR-90/29)
  - Completed report reviewing and assessing nondestructive methods relative to provision of statistical data for use with condition assessment and reliability-based life prediction models (NISTIR 4874)
  - Participated in government- and industry-sponsored short courses and seminars: Nondestructive Testing of Concrete Materials and Structures (University of Wisconsin - Milwaukee), Workshop on Nondestructive Evaluation for Performance of Civil Structures (NSF), CBT Symposium on Facility Diagnostics (NIST), Nondestructive Testing of Civil Structures and Materials (NSF), Society for Experimental Mechanics International Conference on Nondestructive Testing of Concrete in the Infrastructure (Detroit), and Diagnosis of Concrete Structures (Bratislava)

- Remedial/Preventative Measures Considerations – Subtask S.3.3
  - Participated in short courses and seminars: Structural Repair of Concrete and Masonry (ASCE), and Concrete Durability and Repair (Michigan State University)
  - Contacted 10 domestic and 11 foreign research establishments to obtain information on repair procedures and techniques to evaluate the performance of repair materials
  - Completed survey on performance of concrete materials and structures in U.S. light-water reactor plants
  - Completed subcontract with Wiss, Janney, Elstner and Associates to provide an assessment of repair procedures that have been utilized for concrete materials and structures (ORNL/NRC/LTR-93/28)
  - Completed subcontract with Howard University to develop damage assessment and repair prioritization methodologies
  - One presentation at international conference describing repair prioritization methodology approach
  - Completed report on assessment of European repair practices for corrosion damaged reinforced concrete (Taywood Engineering Subcontract Report 1303/91/5823)
  - Completed report on corrosion of metals either embedded or in contact with concrete, potential for stray electrical current-induced corrosion in nuclear power plants, and assessment of performance of cathodic protection systems (ORNL/NRC/LTR-93/2)
  - Implemented subcontract to identify potential structural safety issues related to aging of post-tensioning systems in nuclear power plant containments
  - Implemented subcontract to develop a recommended procedure that can be utilized to assess the current structural reliability and to develop data for use in assessments of future performance of the safety-related concrete structures in nuclear power plants

Task S.4 – Quantitative Methodology for Continued Service Considerations

- Completed discussions with recognized experts on subject of reliability-based techniques for current and future condition assessments of concrete structures

- Implemented subcontract with the Johns Hopkins University to develop a methodology for use in performing current condition assessments and reliability-based life predictions for concrete structures in nuclear power plants

  - Presented papers at Eighteenth WRSM and NRC Aging Research Information Meeting
  - Completed review of literature on methods for time-dependent reliability analysis
  - Developed probabilistic approaches for assessing time-dependent reliability and deterioration of reinforced concrete structural components subjected to stochastic load events
  - Completed framework for a probability-based method to evaluate time-dependent reliability of a single or series of structural components
  - Completed report describing development of a probabilistic method for condition assessment and reliability-based life prediction of concrete structures (ORNL/NRC/LTR-92/4)
  - Completed report presenting methodology to facilitate quantitative assessments of current and future structural reliability and performance of concrete structures in nuclear power plants and applied it to provide a basis for selecting appropriate periods for continued service and determining optimum intervals and extent of inspection and maintenance (NUREG/CR-6052)
  - Completed report addressing reliability assessment of degrading concrete shear walls (ORNL/NRC/LTR-94/6)
  - Presented papers at three international and one national conference describing quantitative methodology
  - Published five journal articles

- Completed two peer review board meetings

APPENDIX B

LIST OF TECHNICAL 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, 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*, Snowbird, Utah, Session 2, Paper 1, American Nuclear Society, July 31–August 3, 1988.

5. D. J. Naus, M. F. Marchbanks, C. B. Oland, and E. G. Arndt, *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, *Structural and Seismic Engineering*, **Vol. 3**, March 1989.

8. D. J. Naus, M. F. Marchbanks, C. B. Oland, and E. G. Arndt, "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.

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* 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, M. F. Marchbanks, C. B. Oland, E. G. Arndt, and T. M. Brown, "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, C. B. Oland, M. F. Marchbanks, and E. G. Arndt, *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 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 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 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 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 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 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 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," *Proceedings of United States Nuclear Regulatory Commission Eighteenth Water Reactor Safety Information Meeting held at Holiday Inn – Crowne Plaza, Rockville, Maryland,* NUREG/CP-0114, pp. 527–552 in *Structural and Seismic Engineering,* **Vol. 1,** April 1991.

21. C. B. Oland and D. J. Naus, "Development of the Structural Materials Information Center," *Proceedings of United States Nuclear Regulatory Commission Eighteenth Water Reactor Safety Information Meeting held at Holiday Inn – Crowne Plaza, Rockville, Maryland,* NUREG/CP-0114, pp. 579–596 in *Structural and Seismic Engineering,* **Vol. 1,** April 1991.

22. B. R. Ellingwood and Y. Mori, "Probabilistic Methods for Condition Assessment and Life Prediction of Concrete Structures in Nuclear Power Plants, *Proceedings of United States Nuclear Regulatory Commission Eighteenth Water Reactor Safety Information Meeting held at Holiday Inn – Crowne Plaza, Rockville, Maryland,* NUREG/CP-0114, pp. 553–577 in *Structural and Seismic Engineering,* **Vol. 1,** April 1991.

23. D. J. Naus, C. B. Oland, and E. G. Arndt, "Aging Management of Safety-Related Concrete Structures in Nuclear Power Plants," *Technical Sessions on Nuclear Plant Systems/Components Aging Management and Life Extension,* ASME Pressure Vessel and Piping Conference 1991, pp. 1–7 in *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 *J. 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 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 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, C. B. Oland, B. R. Ellingwood, Y. Mori, and E. G. Arndt, "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, C. B. Oland, B. R. Ellingwood, Y. Mori, and E. G. Arndt, "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,* **Vol. 228**, Technical Session 16 on Nuclear Plant Systems/Components Aging Management and Life Extension, New Orleans, Louisianna, June 21–25, 1992.

39. D. J. Naus, C. B. Oland, B. R. Ellingwood, Y. Mori, and E. G. Arndt, "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 *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, C. B. Oland, B. R. Ellingwood, Y. Mori, and E. G. Arndt, "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, Technion City Haifa, Israel, I. Soroka editor, 1992.

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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.

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53. W. J. Swiat, W. Young, J. Pajak, M. Funahashi, D. Burke, and J. Wagner, *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.

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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.

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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.

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66. 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, 1993.

67. D. J. Naus, C. B. Oland, E. G. Arndt, B. Ellingwood, and Y. Mori, "An Overview of the ORNL/NRC Program to Address Aging of Concrete Structures in Nuclear Power Plants," pp. 327–329 in *Nuclear Engineering and Design,* **142**, 1993.

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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 *Nuclear Engineering and Design,* **142,** 1993.

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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.

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APPENDIX C

LIST OF TECHNICAL PRESENTATIONS UNDER THE SAG PROGRAM

1. D. J. Naus, "Structural Aging Considerations in Nuclear Power Plant Life Extension," Engineering Technology Division Information Meeting, Oak Ridge National. Laboratory, Oak Ridge, Tennessee, September 10, 1986.

2. D. J. Naus, "Aging of Concrete Components and Its Significance Relative to Life Extension of Nuclear Power Plants," *9th International Conference on Structural Mechanics in Reactor Technology,* Lausanne, Switzerland, August 20, 1987.

3. D. J. Naus, "Structural Life Extension Program," presented to D. F. Ross, W. Houston, and J. Hulman, Division of Engineering, Office of Nuclear Regulatory Research, NRC/RES Program Review, Oak Ridge National Laboratory, Oak Ridge, Tennessee, October 8, 1987.

4. D. J. Naus, "Structural Aging Program," presented to G. A. Arlotto, Director, Division of Engineering, Office of Nuclear Regulatory Research, NRC/RES Program Review, Oak Ridge National Laboratory, Oak Ridge, Tennessee, April 20, 1988.

5. D. J. Naus, "Structural Aging Program," presented to H. A. Dopchie (VINCOTTE), F. F. Motte (C.E.N./S.C.K.), H. M. L. Dreese (INTERCOM), and J. P. Poncelet (Ministry of Economic Affairs), Oak Ridge National Laboratory, Oak Ridge, Tennessee, May 3, 1988.

6. D. J. Naus, "Structural Aging Program," presented to E. S. Beckjord, Director, Office of Nuclear Regulatory Research, NRC/RES Program Review, Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 8, 1988.

7. D. J. Naus, "Structural Aging Program," presented to Dr. J. G. Tyror, Director, Safety and Reliability Directorate, United Kingdom Atomic Energy Authority, Oak Ridge National Laboratory, Oak Ridge, Tennessee, June 6, 1989.

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11. D. J. Naus, "Considerations in the Evaluation of Concrete Structures for Continued Service in Aged Nuclear Power Plants," *American Power Conference 51st Meeting,* Chicago, Illinois, April 25, 1989.

12. D. J. Naus, "Concrete Technology Program at ORNL," presented to A. M. Gill, Office of Nuclear Reactor Regulation, Planning, Program & Management Branch, Oak Ridge National Laboratory, Oak Ridge, Tennessee, May 10, 1989.

13. D. J. Naus, "Structural Aging Program," presented to L. C. Shao, Director, Division of Engineering, Office of Nuclear Regulatory Research, NRC/RES Program Review, Oak Ridge National Laboratory, Oak Ridge, Tennessee, July 27, 1989.

14. D. J. Naus, "Structural Aging Program to Assess the Adequacy of Critical Concrete Components in Nuclear Power Plants," presented at the *10th International Conference on Structural Mechanics in Reactor Technology*, Anaheim, California, August 16, 1989.

15. D. J. Naus, "Aging of Concrete Structures," Atomic Safety Licensing Board Panel FY 1989 Annual Meeting, Gatlinburg, Tennessee, October 31, 1989.

16. D. J. Naus, "ORNL Structural Aging Program," New Production Reactors Working Group on Standards, Washington, D.C., November 16, 1989.

17. D. J. Naus, "Assessment of the Significance of Aging of Concrete Structures in Nuclear Power Plants," *119th Annual Meeting of the Minerals, Metals & Materials Society*, Anaheim, California, February 22, 1990.

18. D. J. Naus, "ORNL Concrete Structural Aging Program," Japanese Nuclear Plant Aging Team of JAERI, Oak Ridge, Tennessee, March 14, 1990.

19. C. B. Oland, "Overview of the Structural Aging Program," Ontario Hydro Research Division, Toronto, Ontario, Canada, March 26, 1990.

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27. D. J. Naus, "Structural Aging Program (BO845)," NRC/RES Laboratory Performance Review Meeting, Oak Ridge National Laboratory, Oak Ridge, Tennessee, November 28, 1990.

28. D. J. Naus, "ORNL Concrete Structural Aging Program," Boston Edison, Oak Ridge, Tennessee, November 29, 1990.

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