ornl

NUREG/CR-4652
ORNL/TM-10059

OAK RIDGE NATIONAL LABORATORY

MARTIN MARIETTA

Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants

D. J. Naus

Prepared for the U.S. Nuclear Regulatory Commission
Office of Nuclear Regulatory Research
Under Interagency Agreement DOE 40-551-75

OPERATED BY
MARTIN MARIETTA ENERGY SYSTEMS, INC.
FOR THE UNITED STATES
DEPARTMENT OF ENERGY

Available from

Superintendent of Documents
U.S. Government Printing Office
Post Office Box 37082
Washington, D.C. 20013-7982

and

National Technical Information Service
Springfield, VA 22161

NUREG/CR-4652
ORNL/TM-10059
Dist. Category RV

Engineering Technology Division

CONCRETE COMPONENT AGING AND ITS SIGNIFICANCE RELATIVE TO LIFE EXTENSION OF NUCLEAR POWER PLANTS

D. J. Naus

Manuscript Completed — August 14, 1986
Date Published — September 1986

Prepared for the
U.S. Nuclear Regulatory Commission
Office of Nuclear Regulatory Research
Under Interagency Agreement DOE 40-551-75

NRC FIN No. B0828

Prepared by the
OAK RIDGE NATIONAL LABORATORY
Oak Ridge, Tennessee 37831
operated by
MARTIN MARIETTA ENERGY SYSTEMS, INC.
for the
U.S. DEPARTMENT OF ENERGY
Under Contract No. DE-AC05-84OR21400

CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

CONCRETE COMPONENT AGING AND ITS SIGNIFICANCE RELATIVE TO LIFE EXTENSION OF NUCLEAR POWER PLANTS*

D. J. Naus

ABSTRACT

The objectives of this study are to (1) expand upon the work that was initiated in the first two Electric Power Research Institute studies relative to longevity and life extension considerations of safety-related concrete components in light-water reactor (LWR) facilities and (2) provide background that will logically lead to subsequent development of a methodology for assessing and predicting the effects of aging on the performance of concrete-based materials and components. These objectives are consistent with Nuclear Plant Aging Research (NPAR) Program goals:† (1) to identify and characterize aging and service wear effects that, if unchecked, could cause degradation of structures, components, and systems and, thereby, impair plant safety; (2) to identify methods of inspection, surveillance, and monitoring or of evaluating residual life of structures, components, and systems that will ensure timely detection of significant aging effects before loss of safety function; and (3) to evaluate the effectiveness of storage, maintenance, repair, and replacement practices in mitigating the rate and extent of degradation caused by aging and service wear.

Applications of safety-related concrete components to LWR technology are identified, and pertinent components (containment buildings, containment base mats, biological shield walls and buildings, and auxiliary buildings), as well as the materials of which they are constructed (concrete, mild steel reinforcement, prestressing systems, embedments, and anchorages) are described. Historical performance of concrete components was established through information presented on concrete longevity, component behavior in both LWR and high-temperature gas-cooled reactor applications, and a review of problems with concrete components in both general civil engineering and nuclear power applications. The majority of the problems identified in conjunction with nuclear power applications were minor and involved either concrete cracking, concrete voids, or low concrete strengths at early ages. Five incidences involving LWR concrete containments considered significant are described

*Research funded by the Nuclear Regulatory Commission NPAR Program and conducted in conformance with its program goals.

†B. M. Morris and J. P. Vora, *Nuclear Plant Aging Research (NPAR) Program Plan*, NUREG-1144, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, Washington, D.C., July 1985.

in detail from occurrence and detection through remedial measures used to restore structural integrity or continuity. These incidences were related either to design, construction, or human error and involved two dome delaminations, voids under tendon-bearing plate, anchor head failures, and a breakdown in quality control and construction management.

Potential environmental stressors and aging factors to which LWR safety-related components could be subjected are identified and discussed in terms of durability factors related to the materials used to fabricate the components (e.g., concrete, mild steel reinforcement, prestressing systems, and embedments). The current technology for detection of concrete aging phenomena is also presented in terms of methods applicable to the particular material system that could experience deteriorating effects. Remedial measures for the repair or replacement of degraded concrete components are discussed, and examples of prerepair and postrepair structural performance are presented to indicate the effectiveness of these measures. Finally, considerations relative to development of a damage methodology for assessment of durability factor deterioration rates and prediction of structural reliability are discussed.

Conclusions and recommendations of the report note the need for (1) obtaining aging data from decommissioned plants, (2) using in-service inspection programs to provide aging trends, (3) developing a methodology to quantitatively and uniformly assess structural reliability as affected by aging or degradation of structural materials, and (4) performing research in support of all these needs. Although, as a group concrete structures have a history of reliability and durability, there is no standardized, widely accepted methodology for quantifying the condition and capacity of an individual structure. Such a means of evaluation needs to be developed if informed licensing decisions are to be made on an extension of licensed design life of nuclear power plant structures.

---

1. INTRODUCTION

1.1 Background

Nuclear power currently supplies ~16% of the U.S. electricity requirements. This percentage is expected to rise to ~20% by 1990.* Despite the increasing role of nuclear energy in power production, the current trend is toward completion (or cancellation) of plants under

---

*As of August 1985, there were 95 licensed U.S. power reactors. Appendix A presents a listing.

construction, with no new nuclear plants having been ordered since the 1970s.*

Although the cessation of orders for nuclear power plants has resulted in a large degree from a slowdown of the growth in demand for electricity, a number of other factors have eroded the economic advantage nuclear power once had over many other forms of energy production. Escalation of material and labor costs, higher interest rates, etc., have resulted in a significant increase in the average duration of plant construction (see Fig. 1) and almost an order of magnitude increase in cost of generating capacity additions since the mid-1960s.[1] These factors have resulted in hesitancy on the part of utilities to consider the construction of new nuclear power plant facilities. In addition two other factors must be considered relative to the ability of the utility industry to meet the future energy requirements: design lifetime and shutdown costs of existing nuclear plants.

The basic laws that regulate the design (and construction) of nuclear plants are contained in Title 10 of the *Code of Federal Regulations*

---

*As of August 1985, 30 plants [21 pressurized-water reactors (PWR), 9 boiling-water reactors (BWR)] were under construction, 90 plants (60 PWRs, 30 BWRs) were canceled or indefinitely deferred, and 2 plants (PWRs) are planned.

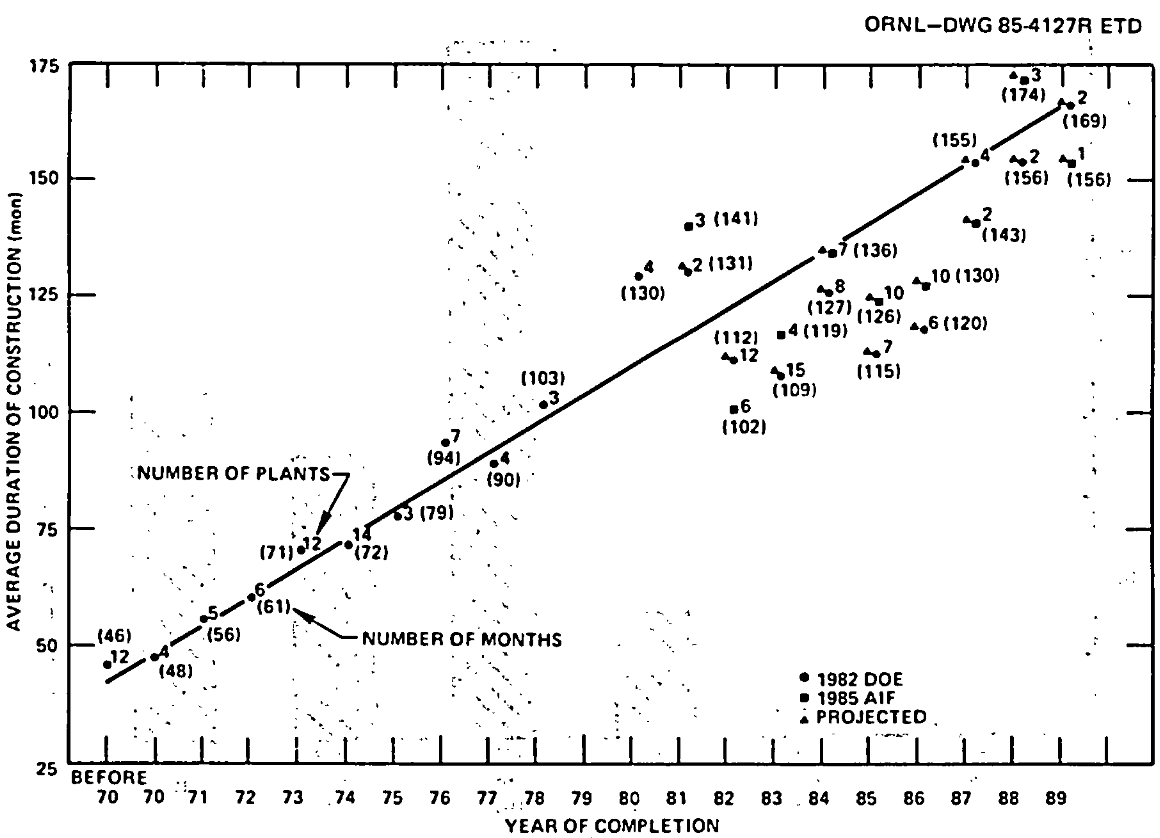


Fig. 1. Average construction time for U.S. nuclear plants.

(10 CFR),[2] which is clarified by Regulatory Guides, NUREG reports, Standard Review Plans, etc. The design lifetime of nuclear plants is somewhat unique because the operating license for a plant has a specific expiration date, usually 40 years from the date of the construction permit issuance. Figure 2, which presents a histogram of light-water reactor (LWR) plants listed in Appendix A as a function of years since an operating license was granted, indicates that plants will start to reach termination of their operating licenses in the next 15 to 20 years. The potential impact of the expiration of operating permits is further clarified in Ref. 3, where it is noted that under the present situation* the United States could experience a loss of electric generating capacity on the order of 150 GW† during the time period 2005 to 2020.

*Assumes no life extension of facilities.

†A more recent estimate of the potential loss of electric generating capacity indicates that the loss is on the order of 50 to 60 GW.

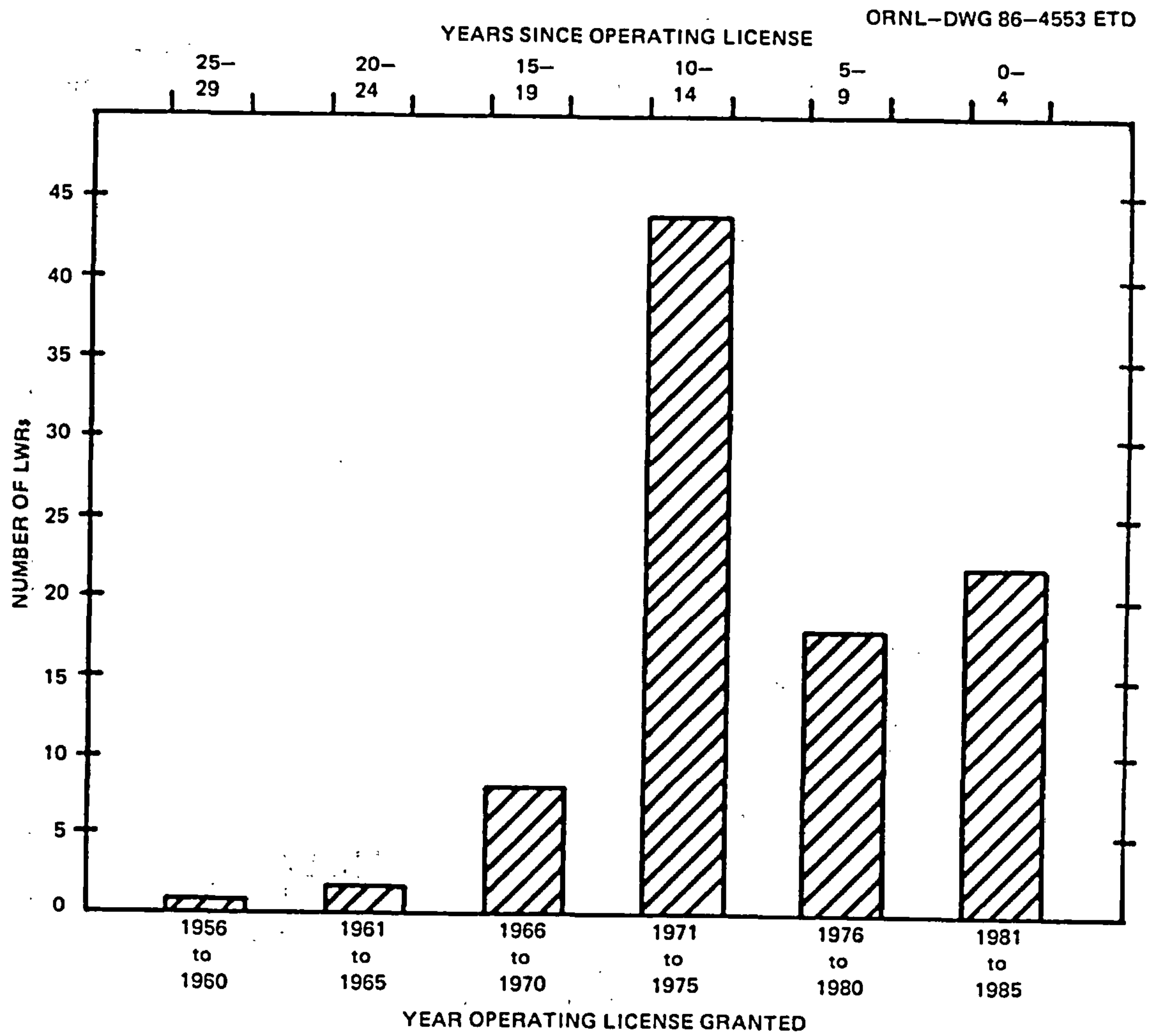


Fig. 2. LWRs licensed during 5-year time intervals.

Final shutdown and decommissioning costs are other important considerations of the utilities. As noted in Ref. 4, estimated dismantlement costs in terms of 1983 dollars range from $14.8 million for Prairie Island 2 to $333 million for Calvert Cliffs 1. Although these costs are small when compared with the initial and lifetime costs of a plant, they must be built into the rate structure based on an agreement between the principal utility owner and the state regulatory commission.

A potential timely and cost-effective solution to the problem of meeting future energy demand is to extend the service life of the nuclear plants. Refurbishment and life extension activities have worked well for non-nuclear generating plants, with some fossil-fueled plants having been in service for 50 to 60 years. Hydroelectric plants are expected to operate for significantly more than 40 years[3]. Refurbishment and life extension should work equally well for nuclear plants, especially because many of the plants may have only been in operation 25 to 30 years prior to expiration of their operating license.

Two Electric Power Research Institute (EPRI) studies[3,5] have already addressed this topic. In the first study[3] the concept of operating a LWR power plant beyond its initial license life was studied from both economic and technological aspects. Conclusions of the study were that (1) extending the life of LWRs was economically beneficial if, up to the first decade of the 21st century, the nuclear fuel costs remain low relative to other feasible baseload power generation technologies (allowable downtime for refurbishment can be several years and cost on the order of several 100 million 1979 dollars before economic feasibility becomes borderline); (2) even in situations where a large piece of equipment such as a reactor pressure vessel or steam generator required replacement, case studies have shown that replacement is feasible; and (3) major repairs or replacement of the concrete structure in the base mat, containment walls, or biological shield within the containment could result in a significant cost. The second EPRI-funded study[5] considered the feasibility of extending the life of existing nuclear power plants and concluded that power reactors should have useful service lives substantially in excess of the licensed 40 years from the date of construction and that a generic method for verifying the continued integrity of concrete structures should be developed.

1.2 Objective

The objective of this study is twofold: (1) to expand upon the work that was initiated in the first two EPRI studies[3,5] relative to the longevity and life extension considerations of safety-related concrete components in LWR facilities and (2) to provide a background that will logically lead to subsequent development of a methodology for assessing and predicting the effects of aging* on the performance of the concrete-based materials and components.

---

*For definition of aging refer to NUREG-1144.[6]

1.3 Approach

Information in the EPRI studies[3,5] indicated that concrete durability (aging) under the influence of either material interactions, aggressive environments (freeze-thaw, wetting-drying, or chemical), or exposure to extreme environments (elevated temperature, irradiation, or seismic) is one of the key issues in nuclear plant life extension. Although operating plants have reported little difficulty with concrete materials, an evaluation of the long-term effects of the environmental challenges to which these structures are subjected has not been adequately addressed.

The approach to be followed in accordance with the Nuclear Plant Aging Research (NPAR) strategy to evaluate the long-term environmental challenges of LWR concrete facilities and thus provide the background material to meet the previously stated objectives will consist of six parts: (1) description of primary safety-related concrete components in LWRs; (2) review of the performance of concrete components in both nuclear and non-nuclear applications; (3) identification and discussion of potential environmental stressors and aging factors to which concrete safety-related components may be subjected in an LWR environment; (4) review of the current state of the art for in-service inspection, surveillance, and detection of concrete aging phenomena and assessment of structural adequacy; (5) discussion of remedial measures for the repair or replacement of degraded concrete components; and (6) remarks concerning correlations between damage assessment and life extension considerations.

References

1. R. J. Christensen, "An Architect-Engineer Perspective," pp. 25–40 in *Proceedings of a Conference on Construction of Power Generation Facilities*, J. Willenbrock, ed., The Pennsylvania State University, University Park, Pa., September 16–18, 1981.

2. 10 CFR.

3. C. A. Negin et al., *Extended Life Operation of Light Water Reactors: Economic and Technological Review*, EPRI NP-2418, vols. 1 and 2, Electric Power Research Institute, Palo Alto, Calif., June 1982.

4. "Public Interest Effort Surveys Decommissioning Tactics and Funding," *Nucleonics Week* 26(17) (April 25, 1985).

5. I. Spiewak and R. S. Livingston, *The Longevity of Nuclear Power Systems*, EPRI NP-4208, Electric Power Research Institute, Palo Alto, Calif., August 1985.

6. B. M. Morris and J. P. Vora, *Nuclear Plant Aging Research (NPAR) Program Plan*, NUREG-1144, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, Washington, D.C., July 1985.

2. DESCRIPTION OF SAFETY-RELATED CONCRETE COMPONENTS IN LWRs

2.1 Design Considerations

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 *10 CFR 50*, "Licensing of Production and Utilization Facilities,"[1] require, in part, that structures, systems, and components important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the safety functions to be performed and that they be designed to withstand the effects of postulated accidents and environmental conditions associated with normal operating conditions.

Initially, existing building codes such as American Concrete Institute (ACI) Standard 318-71, *Building Code Requirements for Reinforced Concrete* (ANSI A89.1-1972),[2] were used in the nuclear industry as the basis for the design of concrete structural members. However, because the existing building codes did not cover the entire spectrum of design requirements and because they were not always considered adequate, the Nuclear Regulatory Commission (NRC) developed its own criteria for the design of Category I structures.* In particular, definitions of load combinations for both operating and accident conditions were provided, as well as a list of tornado-borne missiles and a description of the characteristics of tornados for different regions of the United States.

Using ACI 318-71 as a basis, with modifications to accommodate the unique performance requirements of nuclear plants, ACI Committee 349 developed and published in October 1976 ACI 349-76, *Code Requirements for Nuclear Safety Related Structures*.[3] The procedures and requirements described in this document are generally acceptable to the NRC staff and provide an adequate basis for complying with the general design criteria for structures other than reactor vessels and containments.† Conditions for applying the procedures and requirements in ACI 349 are presented in Ref. 4, and additional information on the design of seismic Category I structures, which are required to remain functional if the Safe Shutdown Earthquake (SSE) occurs, are contained in Ref. 5. Reference 6 presents a good comparison between ACI 318 and ACI 349.

Requirements for the design of concrete reactor vessels and containments are presented in ACI 359-77, *ASME Section III — Division 2, Code*

---

*Category I structures are those essential to the function of the safety class systems and components, or that house, support, or protect safety class systems or components, and whose failure could lead to loss of function of the safety class system and components housed, supported, or protected.

†ACI 349-76 is endorsed by *U.S. Nuclear Regulatory Guide 1.142*, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)."[4]

for Concrete Reactor Vessel and Containments.*[7] Supplemental load combination criteria are presented in Sect. 3.8.1 of the *NRC Standard Review Plan* [NUREG-0800].[8]

2.2 Seismic Category I Structures

A myriad of concrete-based structures are contained as a part of an LWR system. Although the particular components may vary somewhat according to the selection of nuclear steam supply system (NSSS) and containment concept, the structures can be grouped into four primary categories for discussion: reactor containment buildings, containment base mats, biological shield walls and buildings, and auxiliary buildings (balance-of-plant structures).

2.2.1 Reactor containment buildings

2.2.1.1 Background. From a safety standpoint the containment building is probably the most important structure of a nuclear power plant facility because it serves as the final barrier against the release of radioactive fission products to the environment under postulated design basis accident (DBA) conditions. Containment design is based on pressure and temperature loadings associated with a loss-of-coolant accident (LOCA), resulting from a double-ended rupture of the largest size pipe in the reactor coolant system. The containment is also designed to retain its integrity under low probability ($<10^{-4}$) environmental loadings such as those generated by earthquake, tornado, or other site-specific environmental events such as floods, seiche, or tsunami. Additionally, the containment is required to provide biological shielding under both normal and accident conditions and is required to protect the internal equipment from external missiles, such as tornado- or turbine-generated missiles and aircraft impact (where postulated). Design pressures and temperatures are dependent on containment-free volume and presence of either heat sinks or pressure suppression systems.

2.2.1.2 Evolution. Prior to 1965, installed capacity of nuclear power plants in the 50- to 400-MW(e) range utilized steel containments of various configurations, for example, spherical, cylindrical with elliptical bottom and hemispherical top, and cylindrical with hemispherical dome and flat slab. Their designs conformed to the American Society of Mechanical Engineers (ASME) *Unfired Pressure Vessel Code*,[10] with the shells fabricated from welded steel plates up to 38 mm in thickness. Support for the reactor vessel and shielding requirements is provided by reinforced concrete. As the plant sizes were increased to 800 MW(e), shielding requirements increased, and the practical limit for fabrication of steel containments without requiring postweld heat treatment were exceeded. At this time it also seemed prudent to combine the containment and shielding functions into a composite steel-lined reinforced concrete structure.

---

*ACI 359-77 is endorsed by *U.S. Nuclear Regulatory Guide 1.136*, "Material for Concrete Containments."[9]

The first concrete containments were built in the mid-1960s and typically consisted of an ~1.4-m-thick cylindrical reinforced concrete wall with an ~1.1-m-thick hemispherical dome and a flat base slab. Leak tightness was provided by a steel liner, which generally ranged from 6.35 to 12.7 mm in thickness depending on its location. Grade 60 (Nos. 11, 14, and 18) reinforcing bars were normally utilized to resist hoop, axial, seismic, and shear loadings. Concrete compressive strengths ranged from 20.7 to 34.5 MPa. Later the concrete was partially prestressed in the vertical direction only with mechanically spliced reinforcing steel in the hoop direction and dome (Ginna, Robinson 2).

Fully prestressed containments were first built in the late 1960s. The first generation of fully prestressed containments typically consisted of an ~1.1-m-thick cylindrical wall, an ~0.8-m-thick shallow (ellipsoidal) dome, a large ring girder at the intersection of the dome and wall, six buttresses, and a flat reinforced concrete base slab. The wall was prestressed by hoop tendons, anchored at two buttresses and spanning 120°, and vertical tendons placed with equal spacings near the inside and outside faces. Dome prestressing was provided by three groups of tendons, with each group at 120° with respect to the other two groups. Tendons consisted of ninety 6.35-mm-diam steel wires posttensioned to ~1.65 GPa. Grade 40 or Grade 60 rebars (Nos. 9, 10, 11, and 18) were used to provide light reinforcement. Concrete strengths ranged from 27.6 to 34.5 MPa.

As a consequence of the quantity of tendons (>900), which required a very labor-intensive activity to fabricate, position, tension, and corrosion proof and an increase in plant size, the second generation of fully prestressed concrete containments was developed. In the second generation containments the number of buttresses was reduced to three, with the hoop tendons spanning 240° between buttresses. This resulted in both a reduction in installation time and an improvement in the radial force distribution on the shell.[11] Another change was that the capacity of the prestressing tendons was approximately doubled, which was reflected in a reduction of up to 510 tendons (depending on containment height, diameter, and design pressure) relative to first generation design requirements.[11] Tendon systems utilized were composed of either one hundred eighty 6.35-mm-diam steel wires or fifty-five 12.7-mm-diam seven-wire strands. Concrete and reinforcing steel requirements and the necessity for a ring girder remained essentially unchanged from the first generation of prestressed concrete containments.

Third-generation prestressed concrete containments replaced the ellipsoidal dome with a hemispherical dome, thus permitting a simpler posttensioning tendon layout. Through the use of inverted U-shaped vertical tendons, the ring girder was eliminated. The inverted U-shaped tendons were divided into two tendon sets oriented 90° to each other in the dome array, with all tendons in each set parallel to each other in the dome and the midtendon in each set located in a diametrical plane of the dome and cylinder. Hoop tendons were still anchored in a three-buttress arrangement with each tendon spanning 240°. Concrete, reinforcing steel, and tendon systems were essentially the same as for the second-generation containments.

2.2.1.3 <u>Summary description of containment types utilized for LWRs in the United States.</u> As of April 30, 1985, there were 95 licensed U.S. nuclear power reactors.[12] Table 1 presents a summary distribution of

Table 1. Summary of containments for U.S. power reactors

Containment construction	Reactor type	Containment description	
		Type	Number
Steel	PWR	Dry	9
	PWR	Ice condenser	5
	BWR	MKI	21
	BWR	MKII	1
	BWR	pre-MK	4
Reinforced concrete	PWR	Subatmospheric	6
	PWR	Dry	6
	PWR	Ice condenser	2
	BWR	MKI	2
	BWR	MKII	3
	BWR	MKIII	1
Prestressed concrete	HTGR		1
	PWR	Dry	32
	BWR	MKII	2

containment concepts that have been used for both the PWR and BWR systems.* As noted in the table, a variety of containment designs have been utilized. Rather than present a summary description for each design, only representative PWR and BWR containment designs for each major type of containment construction have been selected to identify major components and indicate design parameters. Containment designs selected include: steel (PWR ice condenser and BWR MKI),† reinforced concrete (PWR subatmospheric, PWR dry, BWR MKII, and BWR MKIII), and prestressed concrete (PWR shallow dome — six buttresses and PWR hemispherical dome — three buttresses). Table 2 presents representative design parameters for each of these containment types for which a reference plant has been selected as an example.

PWR ice condenser. The containment for each of the Sequoyah reactors consists of a free-standing steel containment vessel, ice condenser, internal reinforced concrete structure, and reinforced concrete shield building.[13] Figure 3 presents the containment configuration. The shield building is a reinforced concrete cylinder 0.9 m thick, with a 2.7-m-thick slab on rock and a 0.6-m-thick dome. Concrete strengths are

*See Appendix A for a more detailed description for each reactor plant.

†Steel containment designs have been included to define and indicate positioning of concrete components associated with the containment configuration.

Table 2. Summary of design parameters for selected containment systems

LWR reactor type	Containment type	Reference plant (Doc. No.)	Allowable leak rate (vol %/d)	Containment[a] free volume (10^3 m^3)	Pressure (kPa)		Temperature (°C)	
					Design	Accident	Design	Accident
PWR ice condenser	Steel cylinder — hemispherical dome	Sequoyah (50-327)	0.25	34.0	Atmospheric	82.7	15.6–48.9	104.4
BWR MKI	Steel — pressure suppression	Peach Bottom (50-277)	0.50	4.5 (D) 3.4 (PSC) 3.9 (PCP)	Atmospheric	427.5	<57.2	138.3
PWR subatmospheric	Reinforced concrete — hemispherical dome	Surry (50-280)	0.10	51.0	62.1–75.8	310.3	15.6–40.6	65.6
PWR dry	Reinforced concrete — hemispherical dome	Indian Point 3 (50-286)	0.10	73.9	Atmospheric	324.1	<48.9	
BWR MKII	Reinforced concrete — pressure suppression	Limerick (50-352)	0.50	7.1 (D) 4.2–4.6 (PSC) 3.3–3.6 (PCP)	Atmospheric	379.2	<57.2	171.1 (D) 104.4 (PSC)
BWR MKIII	Reinforced concrete — pressure suppression	Grand Gulf (50-416)	0.40	39.6 7.6 (D) 3.9 (PCP)	Atmospheric	103.4 206.8 (D)	<57.2	85.0 165.6 (D)
PWR shallow dome	Prestressed concrete — six buttresses	Zion (50-295)	0.10	73.6	Atmospheric	324.1	<54.4	132.8
PWR hemispherical dome	Prestressed concrete — three buttresses	Trojan (50-344)	0.20	63.0	Atmospheric	413.7	<48.9	138.3

[a]D = drywell
PSC = pressure—suppression chamber
PCP = pressure—suppression chamber pool water.

ORNL–DWG 86–4554 ETD

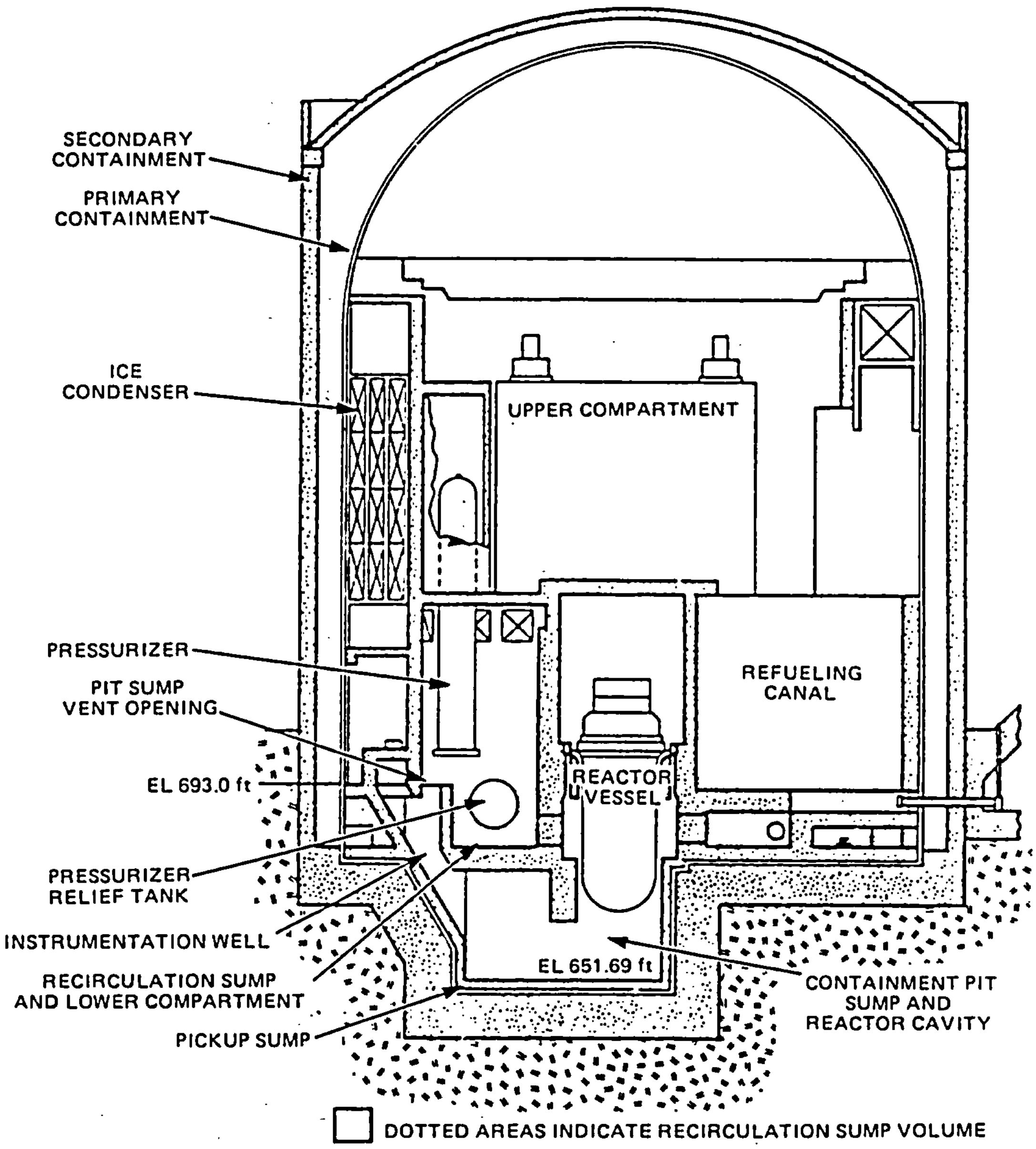


Fig. 3. PWR ice condenser containment configuration. *Source: Containment Performance Working Group Report, Draft Report for Comment*, NUREG-1037, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.

27.6 MPa* for the cylinder dome and walls and 20.7 MPa for the base slab. A 5.2-m-ID circular reinforced concrete wall 2.59 m thick (primarily for biological shielding) supports and encloses the 6.23-MN reactor vessel above the lower reactor cavity. Other concrete components include a variety of walls, divider barriers, floor slabs, and columns.

BWR Mark I. The containment at Peach Bottom and Browns Ferry[14,15] is a pressure—suppression system that consists of a drywell, a pressure—suppression chamber (torus) that stores a large volume of water, and a connecting vent system between the drywell and water pool (Fig. 4). The drywell is a lightbulb-shaped steel pressure vessel with a spherical lower portion and a cylindrical upper portion. The suppression chamber is a steel pressure vessel in the shape of a torus, which is located below and encircles the drywell. Eight circular vent pipes connect the suppression chamber with the drywell. The drywell is enclosed in a reinforced concrete structure for shielding purposes. In areas where it backs up the drywell shell, the reinforced concrete provides additional resistance to deformation of the containment shell. Shielding over the

*Concrete strengths are presented as 28-d design values. Actual strength levels of the concrete in the structures in all likelihood significantly exceeded these values.

ORNL–DWG 86–4555 ETD

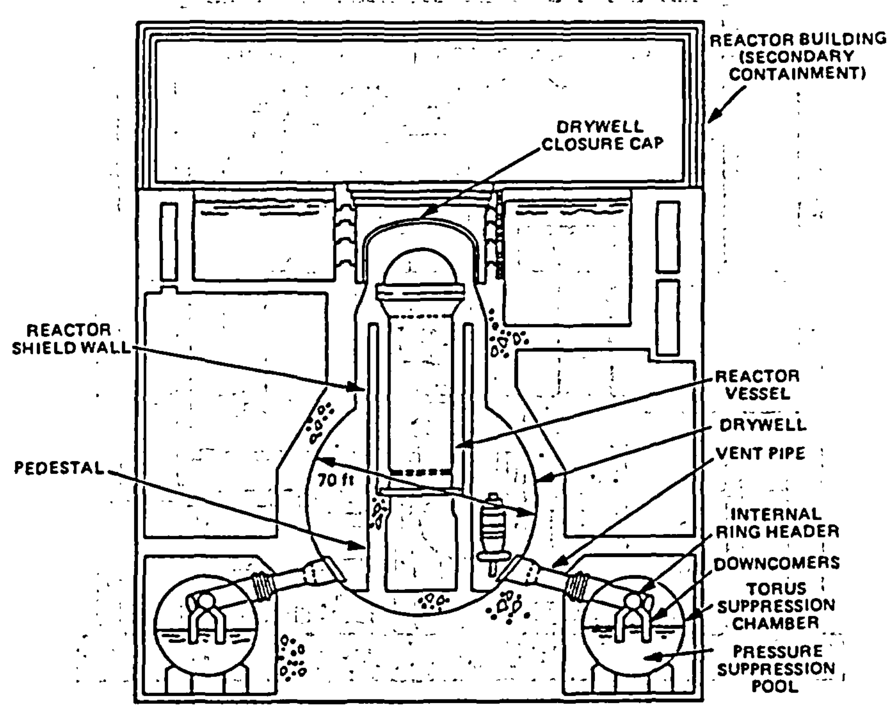


Fig. 4. BWR Mark I containment configuration. *Source: Containment Performance Working Group Report, Draft Report for Comment*, NUREG-1037, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.

top of the drywell is provided by removable, segmented, reinforced concrete plugs.

PWR subatmospheric. The containment vessel at Surry[14,16] is a steel-lined reinforced concrete structure with an ~1.4-m-thick vertical cylindrical wall, an ~0.8-m-thick hemispherical dome, and an ~3-m-thick flat base slab (Fig. 5). The steel liner for the wall is 9.5 mm thick. Over the base mat the steel liner consists of 6.35- and 19.1-mm plates. Approximately 0.6 m of concrete is placed on top of the mat liner to protect it from thermal loadings and internal missiles. The steel liner for

ORNL–DWG 86–4556 ETD

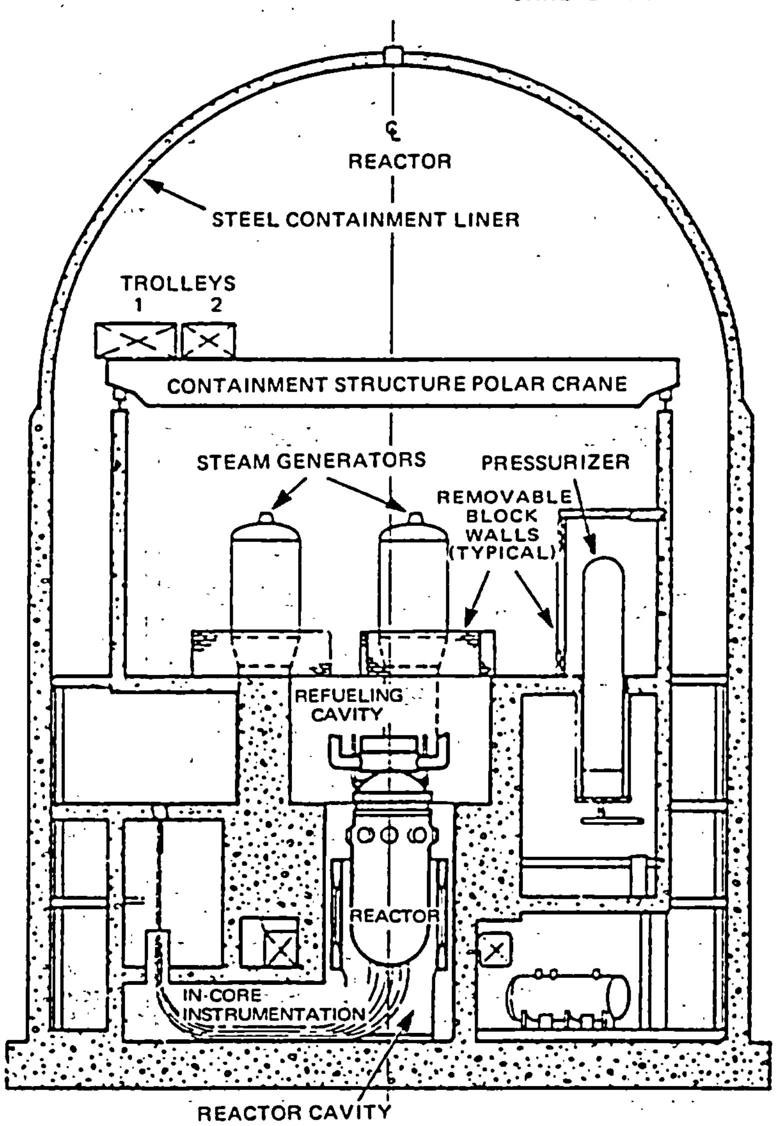


Fig. 5. PWR subatmospheric containment building. *Source: Containment Performance Working Group Report, Draft Report for Comment*, NUREG-1037, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.

the dome is 12.7 mm thick. The containment is fabricated using 20.7- and 27.6-MPa concretes. Primary reinforcement is provided by bars placed circumferentially and axially in the wall. Seismic reinforcement consists of bars placed helically at an angle of 45° with the horizontal in both directions in the wall of the cylinder. Stirrups or diagonal bars are provided in the lower portion of the cylinder to resist radial shear. Internally, reinforced concrete is used for biological shielding, reactor vessel support, barriers, floors, and walls.

PWR dry. The containment building at Indian Point 3[14,17] (Fig. 6) is a reinforced concrete structure consisting of an ~2.7-m-thick base mat, an ~1.4-m-thick cylindrical wall, and an ~1.1-m-thick hemispherical dome. Concrete strengths are on the order of 21 to 28 MPa. Leak tightness is provided by a ductile steel liner whose thickness is 6.35 mm over

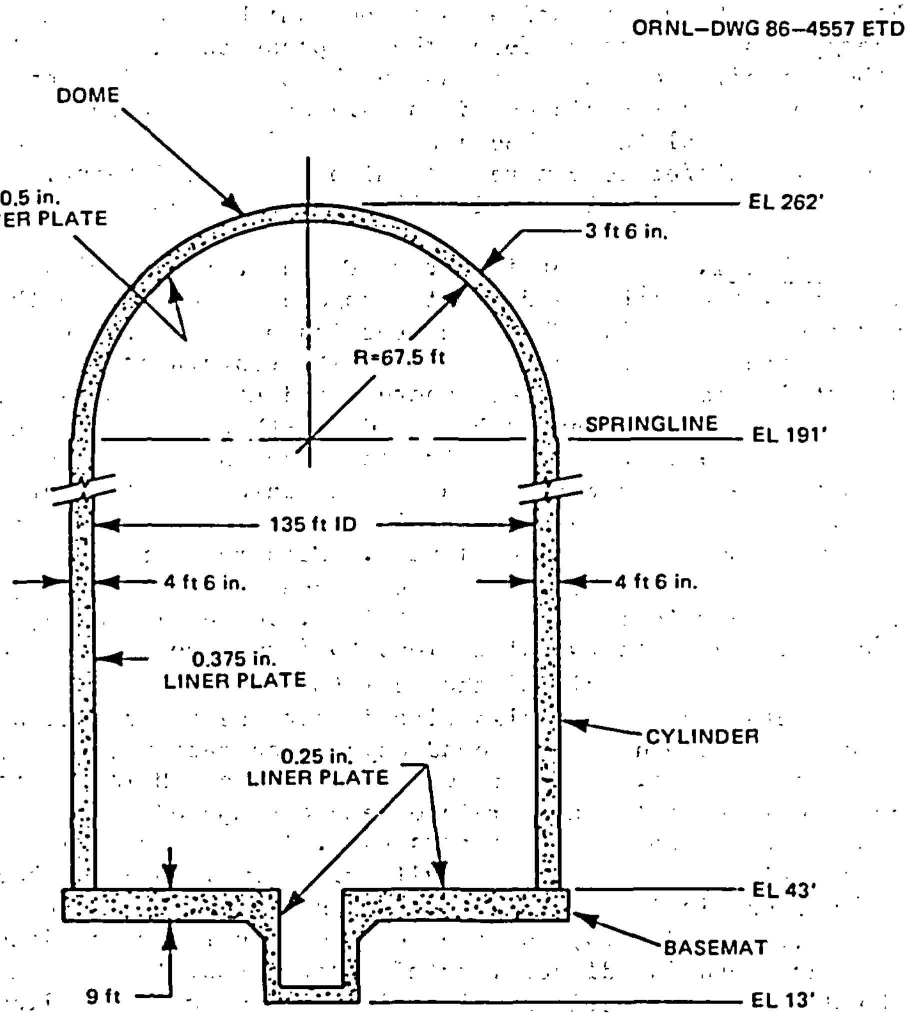


Fig. 6. Indian Point containment building. *Source*: S. Sharma, Y. K. Wang, and M. Reich, *Ultimate Pressure Capacity of Reinforced and Prestressed Concrete Containments*, NUREG/CR-4149, BNL-NUREG-57857, Brookhaven National Laboratory, Upton, New York, May 1985.

the base mat, 12.7 mm in the dome and bottom 9.1-m section of the cylinder, and 9.5 mm over the remaining height of the cylinder. Containment reinforcement consists primarily of Nos. 11, 14, and 18 Grade 60 reinforcing bars. Membrane reinforcement in the cylinder wall and dome is divided into two groups placed near the inside and outside faces of the containment wall. Each group consists of two layers of hoop bars and one layer of meridional bars. A layer of helical bars at +45° with the vertical axis is placed near the outside wall face to resist in-plane seismic forces. Secondary meridional reinforcement is also provided at the base mat—cylinder intersection to help resist the high bending moments and shear forces that could develop.

BWR Mark II. A pressure—suppression system consisting of a drywell and suppression chamber separated by a horizontal diaphragm slab is used as the containment at Limerick.[14] The containment is in the form of a truncated cone over a cylindrical section positioned on a flat foundation mat (Fig. 7). The upper conical section, which contains the reactor and high-energy piping systems, forms the suppression chamber. A floor separates the drywell from the suppression chamber, with downcomers providing communication. The containment is a reinforced concrete structure lined with welded steel plate and has a steel domed closure head at the top of the drywell. Reinforced concrete is also utilized internally in the containment as a sacrificial shield wall, reactor support, columns, and floors.

BWR Mark III. The containment at Grand Gulf[14,18,19] is a pressure—suppression system with the drywell completely enclosed by the containment structure (Fig. 8). The lower portion of the structure also serves to form the pressure—suppression pool. The containment is constructed of 34-MPa cast-in-place reinforced concrete and consists of an ~1.1-m-thick right circular cylinder capped by an ~0.8-m-thick hemispherical dome and founded on an ~2.9-m-thick flat circular base mat. The inside surface of the containment is covered by a 6.35-mm-thick steel liner, which forms a leak-tight barrier (type 304 stainless steel material is used in the area below the suppression pool level). Main reinforcement in the wall consists of inside and outside layers of hoop reinforcement (No. 18 bars), outside vertical reinforcement (Nos. 10 and 18 bars), and diagonal reinforcement placed in two directions to form a helix with an angle of ~45° from the vertical axis of the wall (Nos. 11, 14, and 18 bars). Additional reinforcement is also placed near the intersection of the wall and foundation mat and areas around major penetrations, pipe penetrations, floor brackets, polar crane brackets, etc. Main reinforcement in the dome consists of (1) hoop reinforcement composed of inner and outer layers of circumferential steel bars (Nos. 10, 14, and 18) extending from the intersection of the dome and cylindrical wall to ~46° above the spring line; (2) inside and outside groups of U-shaped reinforcement (Nos. 10, 14, and 18 bars) composed of two mutually perpendicular layers of steel bars; and (3) diagonal reinforcement (No. 11 bars) continuous with the diagonal cylindrical wall reinforcement and extending up to ~40° above the spring line. Additional dome reinforcement includes meridional bars (Nos. 14 and 18) as a continuation of inside face vertical reinforcement in the cylinder wall. Internal reinforced concrete structures include reactor support pedestal, shield walls, drywell walls, weir walls, etc.

ORNL–DWG 86–4558 ETD

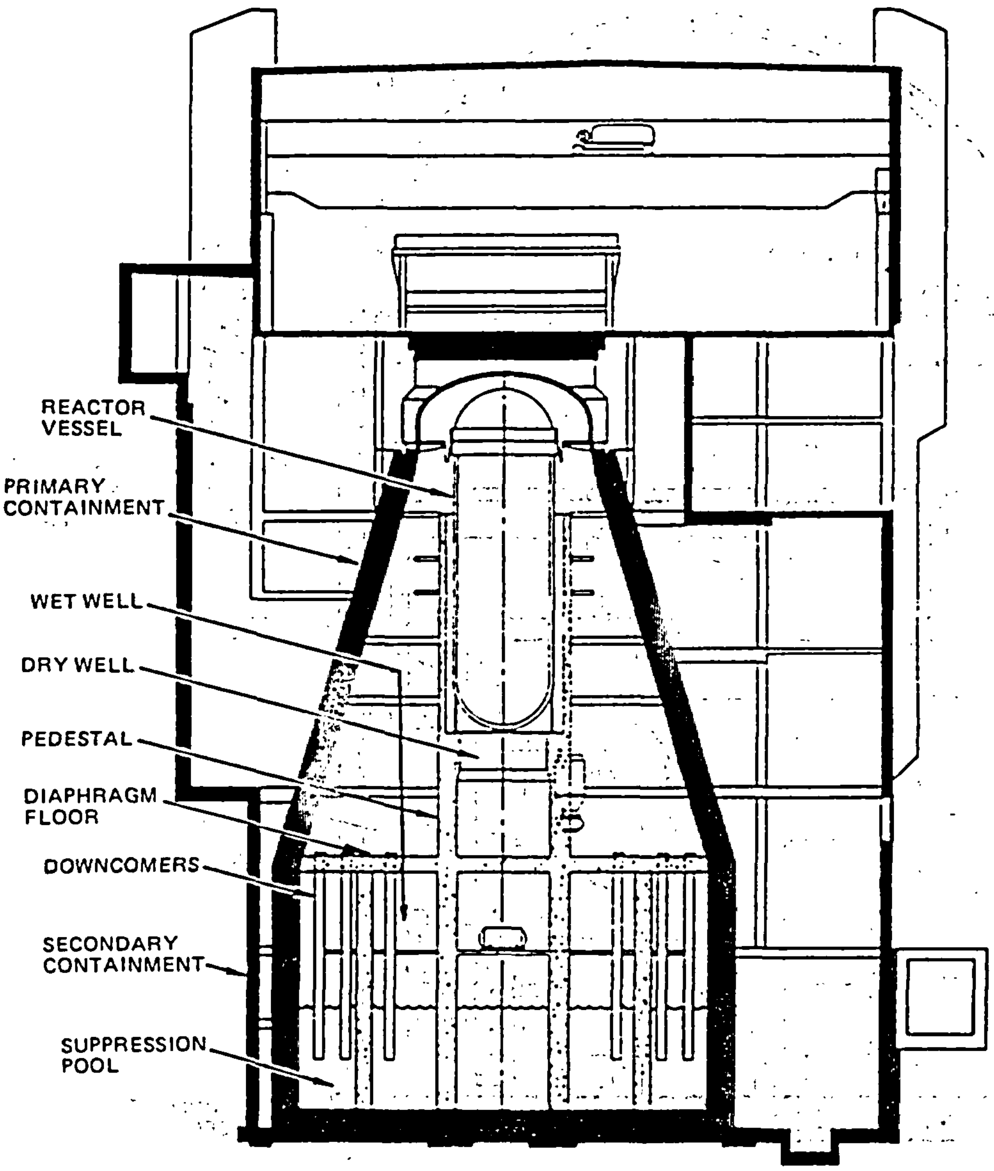


Fig. 7. BWR Mark II containment configuration. *Source: Containment Performance Working Group Report, Draft Report for Comment*, NUREG-1037, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.

PWR shallow dome — six buttresses. The containment at Zion[14,20] is a steel-lined prestressed concrete structure with an ~1.1-m-thick vertical cylinder wall and an ~0.8-m-thick shallow (ellipsoidal) domed roof supported on a reinforced concrete foundation slab (Fig. 9). The containment is fabricated from 34.5-MPa reinforced concrete, and a 6.35-mm-thick steel liner is utilized to provide leak tightness. A large ring girder is positioned at the intersection of the dome and wall for anchorage of the dome and vertical prestressing tendons, and six buttresses are

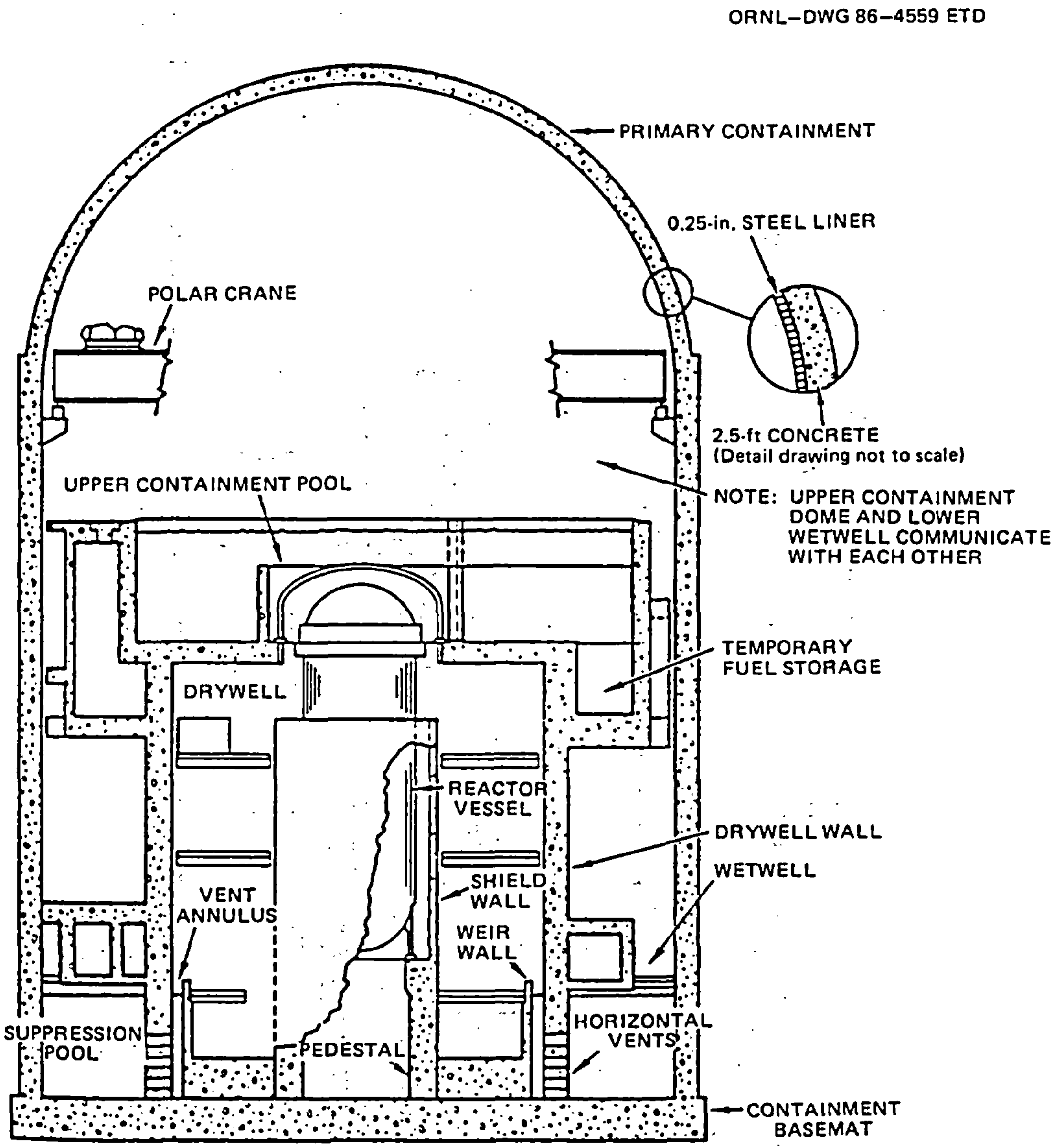


Fig. 8. BWR Mark III containment configuration. *Source: Containment Performance Working Group Report, Draft Report for Comment*, NUREG-1037, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.

provided for anchorage of the hoop tendons. Hoop prestressing is provided by 3 groups of 193 tendons, with each group anchored at 2 buttresses spanning 120°. Vertical prestressing is provided by 216 vertical (meridional) equidistant tendons placed near the inside and outside wall surfaces. Dome prestressing consists of 3 groups of 63 tendons, with each group at 120° with respect to the other groups. All tendons consist of ninety 6.35-mm-diam steel wires and are posttensioned to ~1.65 GPa. Light reinforcement consisting primarily of Nos. 10, 11, and 18 Grade 60

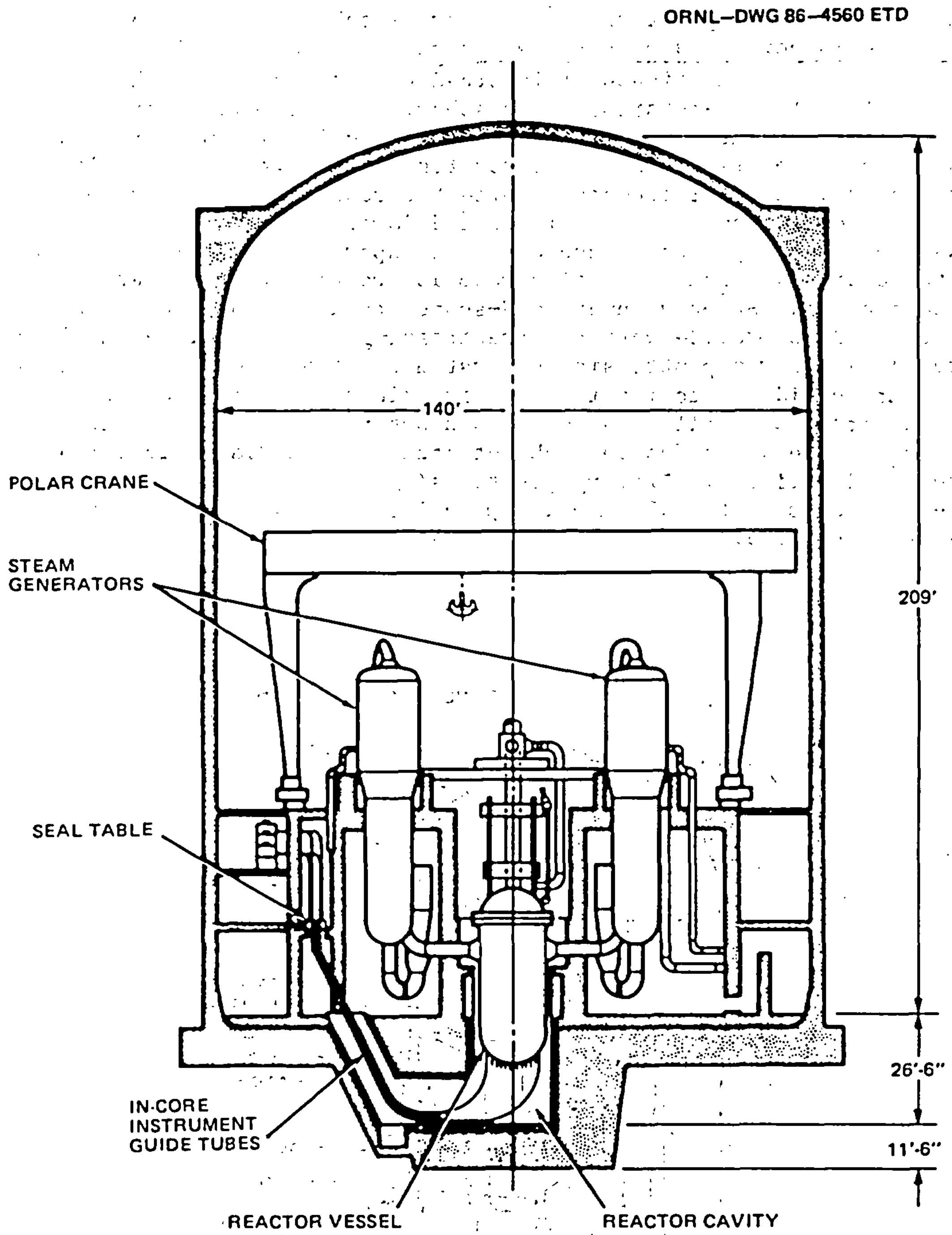


Fig. 9. PWR large dry containment configuration. *Source: Containment Performance Working Group Report, Draft Report for Comment*, NUREG-1037, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.

rebars is also present. Reinforced concrete is utilized also for biological shield and support of the reactor vessel and steam generators.

PWR hemispherical dome — three buttresses. The containment at Trojan[21] is a fully continuous reinforced concrete structure having a cylindrical wall ~1.1-m-thick, a hemispherical dome ~0.8 m thick, and an ~2.7-m-thick base mat (Fig. 10). The cylindrical and dome portions of the structure are prestressed by a posttensioning system consisting of horizontal and vertical tendons. Three buttresses are equally spaced around the containment. The cylinder and lower half of the dome are prestressed by 150 hoop tendons anchored 240° apart by bypassing the intermediate buttress. Each successive hoop is progressively offset 120° from the one beneath. Seventy inverted U-shaped tendons continuous over the dome are used to provide vertical prestressing in the cylinder wall and to provide a two-way posttensioning system for the dome. The U-shaped tendons are divided into two tendon sets oriented 90° to each other in the dome array, with all tendons in each set parallel to each other in the dome and the midtendon in each set located in a diametrical plane of the dome and cylinder. Primary prestressing for the containment consists of one-hundred seventy 6.35-mm-diam parallel wires with anchorage provided by buttonheading.

ORNL-DWG 86-4561 ETD

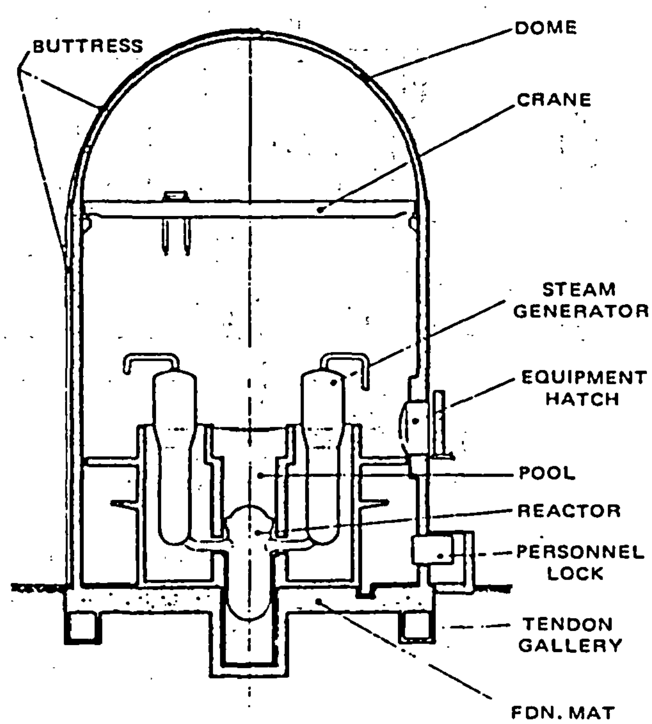


Fig. 10. PWR hemispherical dome — three buttress containment.

2.2.2 Containment base mats

Base mats for reactor containment vessels can be either reinforced, prestressed, or a combination of reinforced and prestressed; however, with very few exceptions the concrete foundation mats have been only conventionally reinforced. Design of the base mat is complicated because it must consider the system parameters (peak internal pressure, temperature, environmental loads) as well as the influence of the soil—structure interaction. The base mat is required to support other loads, including direct equipment loads and dead loads transmitted through the containment wall, primary loop compartment, and primary shield wall.

Depending on the siting conditions, the base mats may be founded on rock, soil, or piles (e.g., piles at Point Beach, La Crosse, Fort Calhoun, and Robinson 2). Thickness requirements of the base mats are controlled by the concrete shear capacity, maximum allowable compressive stress of concrete, maximum allowable steel area, and allowable soil-bearing pressure.[22] Where the containment concept requires a tendon gallery for providing access to the vertical prestressing tendon anchorages during construction and subsequent in-service inspections, the gallery can be considered as an integral part of the base mat and encircles it at the bottom.

Figures 3—10 present examples of base mat configurations that have been used in conjunction with the various containment concepts discussed in Sect. 2.2.1. As noted earlier, the base mats are fabricated of reinforced concrete. Thicknesses vary according to loading and soil conditions, but in general base mat thicknesses have ranged from ~2.6 m (Oconee) to 4.1 m (Palisades). The base mats are circular in design and may be >45 m in diameter. Concrete, normally fabricated from Type II cement with compressive strengths from 20.7 to 34.5 MPa, is used to fabricate the base mats. Either Grade 40 or Grade 60 steel bars, typically ranging in size from Nos. 9 to 18, are used to reinforce the base mat. Example rebar layouts for the Indian Point 3, Zion, and Grand Gulf containments are presented in Figs. 11—13, respectively.

2.2.3 Biological shield walls and buildings

Biological shield walls for commercial reactors are fabricated from standard weight reinforced concrete. Thicknesses of the shield walls typically range from ~1.5 to 4 m, and the walls can either support all or part of the reactor pressure vessel weight. Concrete compressive strengths ranging from 27.6 to 41.4 MPa are normally used for shield fabrication. Using Yankee Rowe as an example, the shield walls are reinforced with ~139 kg of rebars per cubic meter of concrete.[23] The reinforcing steel is provided to take flexural and seismic loads that would place portions of the wall in tension.

A shield building, or secondary containment, is a medium leakage reinforced concrete structure that surrounds the steel containment vessel (see Fig. 3). The building is designed to provide (1) biological shielding from accident conditions; (2) biological shielding from parts of the reactor coolant system during operation; and (3) protection of the containment vessel from low temperatures, adverse atmospheric conditions,

ORNL-DWG 86-4562 ETD

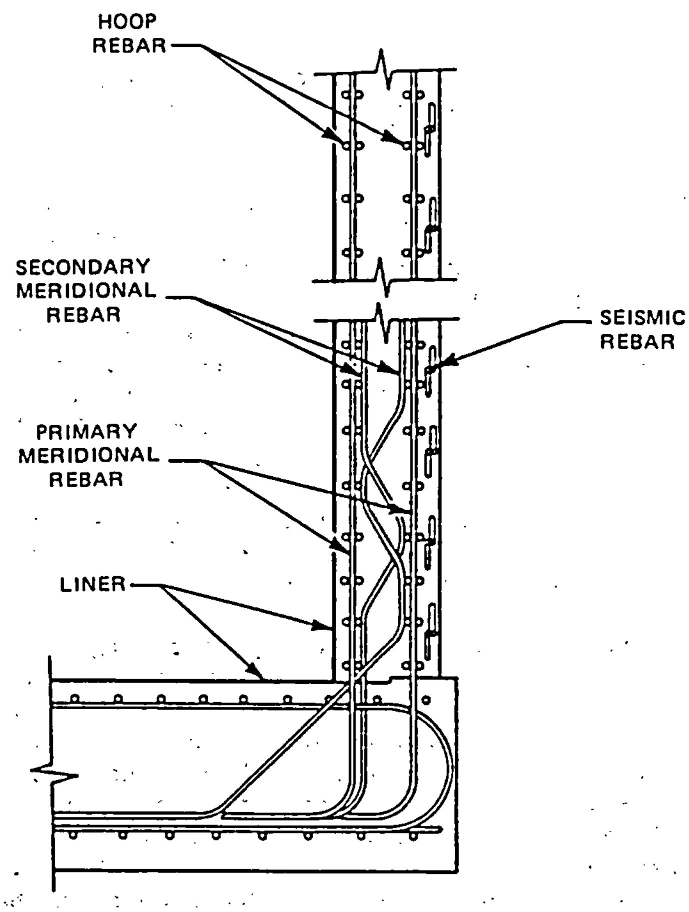


Fig. 11. Reinforcements in Indian Point containment cylinder. *Source*: S. Sharma, Y. K. Wang, and M. Reich, *Ultimate Pressure Capacity of Reinforced and Prestressed Concrete Containments*, NUREG/CR-4149, BNL-NUREG-57857, Brookhaven National Laboratory, Upton, New York, May 1985.

and external missiles.[13] Typically the building can be a reinforced concrete cylinder with a base slab and spherical dome. Cylinder wall thickness is ~0.9 m, and the dome is ~0.6 m thick. Concrete strengths used in construction of the building range from 20.7 to 27.6 MPa. Concrete reinforcement is provided by Grade 60 rebars provided in steel to concrete ratios ranging from 0.003 to 0.017.

2.2.4 Auxiliary buildings

Auxiliary buildings include functional units such as diesel generator building, control room/building, spent-fuel pit, fuel-handling building, safety valve room, radioactive waste building, and waste management building. Figure 14, obtained from Ref. 24, categorizes the

ORNL–DWG 86–4563 ETD

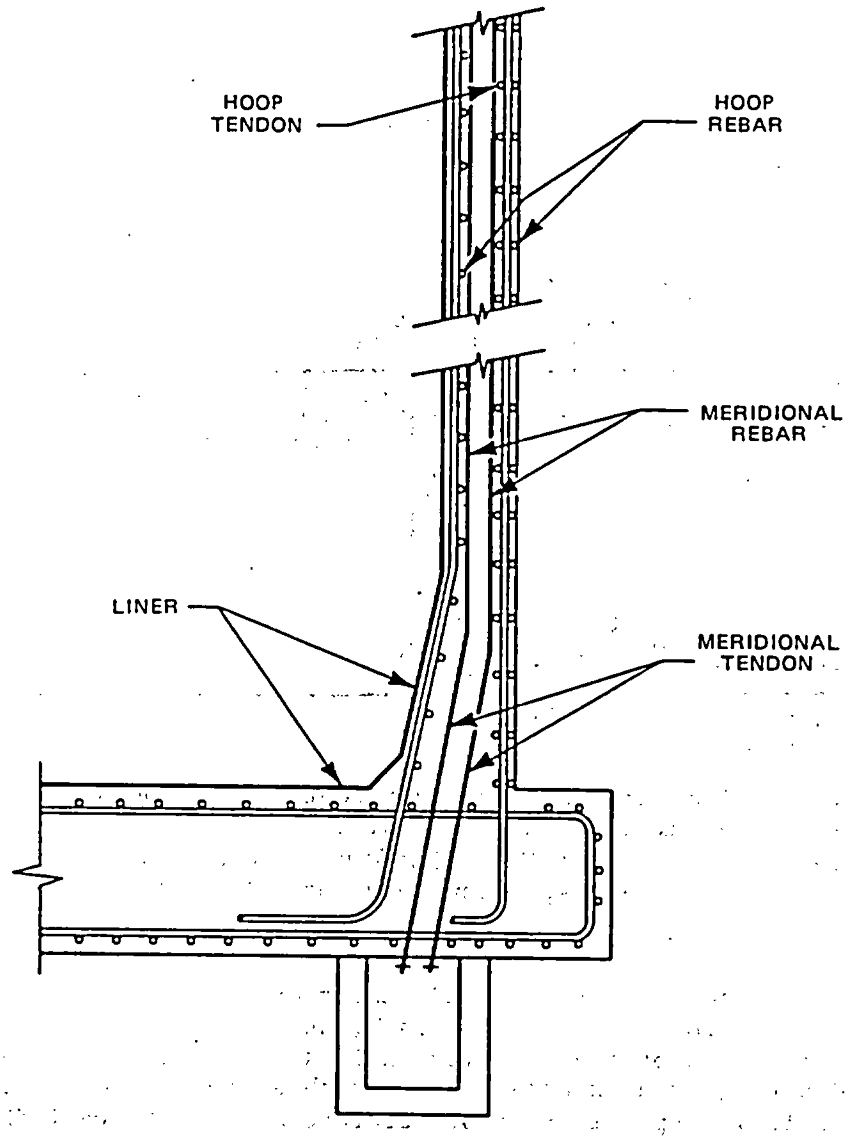


Fig. 12. Reinforcements and tendons in Zion containment cylinder. *Source*: S. Sharma, Y. K. Wang, and M. Reich, *Ultimate Pressure Capacity of Reinforced and Prestressed Concrete Containments*, NUREG/CR-4149, BNL-NUREG-57857, Brookhaven National Laboratory, Upton, New York, May 1985.

location of the auxiliary function units with respect to the reactor containment building into three basic types, with the auxiliary function units either being a single continuous structure or an aggregate of several disjoined buildings. In general, however, these structures are box-shaped, shear-wall buildings (see Fig. 15) constructed of reinforced concrete (concrete compressive strengths ranging from 27 to 41 MPa, 413-MPa steel rebar yield strength), but they may contain steel beams (A36 structural steel) that support the floor slabs. Basic structural components

ORNL–DWG 86–4564 ETD

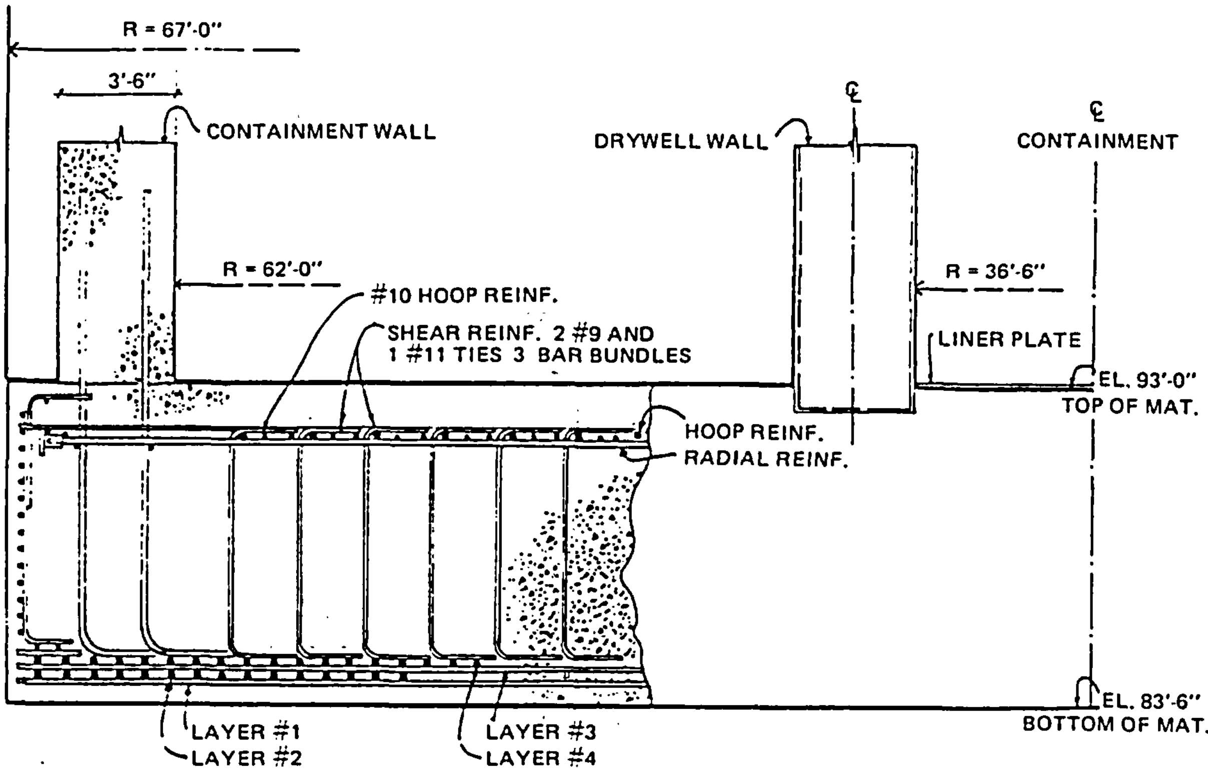


Fig. 13. Section through Grand Gulf containment foundation mat. *Source*: S. Sharma et al., *Failure Evaluation of a Reinforced Concrete Mark III Containment Structure Under Uniform Pressure*, NUREG/CR-1967, Brookhaven National Laboratory, Upton, New York, May 1982.

of the auxiliary function units include exterior and interior walls, base or foundation slab, roof slab, floor slabs, and columns.

The main function of the exterior walls is to protect safety class equipment and piping from external events such as tornadoes and tornado-generated missiles. Typical exterior reinforced concrete wall thicknesses range from 0.45 to 1.2 m, as determined by the most severe penetrating tornado-generated missile considered possible. Reinforcing steel requirements are generally based on the magnitude of pressure and energy loads acting normal to the walls.

Interior wall thicknesses range from 0.3 to 1.2 m. The walls may be constructed of reinforced concrete, concrete masonry, or heavyweight concrete. Wall thicknesses and the amount of reinforcing steel are selected on the basis of resisting loads resulting from internally generated missiles, equipment and pipe supports, pressure transients, jet impingement, thermal gradients, or radiation shielding requirements.

Base or foundation slab thickness requirements are dependent on site foundation conditions and plant seismic threat. Generally constructed of reinforced concrete, the base or foundation slabs range in thickness from 1.8 to 8.2 m, with larger thicknesses required where the plant is located on soft soils or piles.

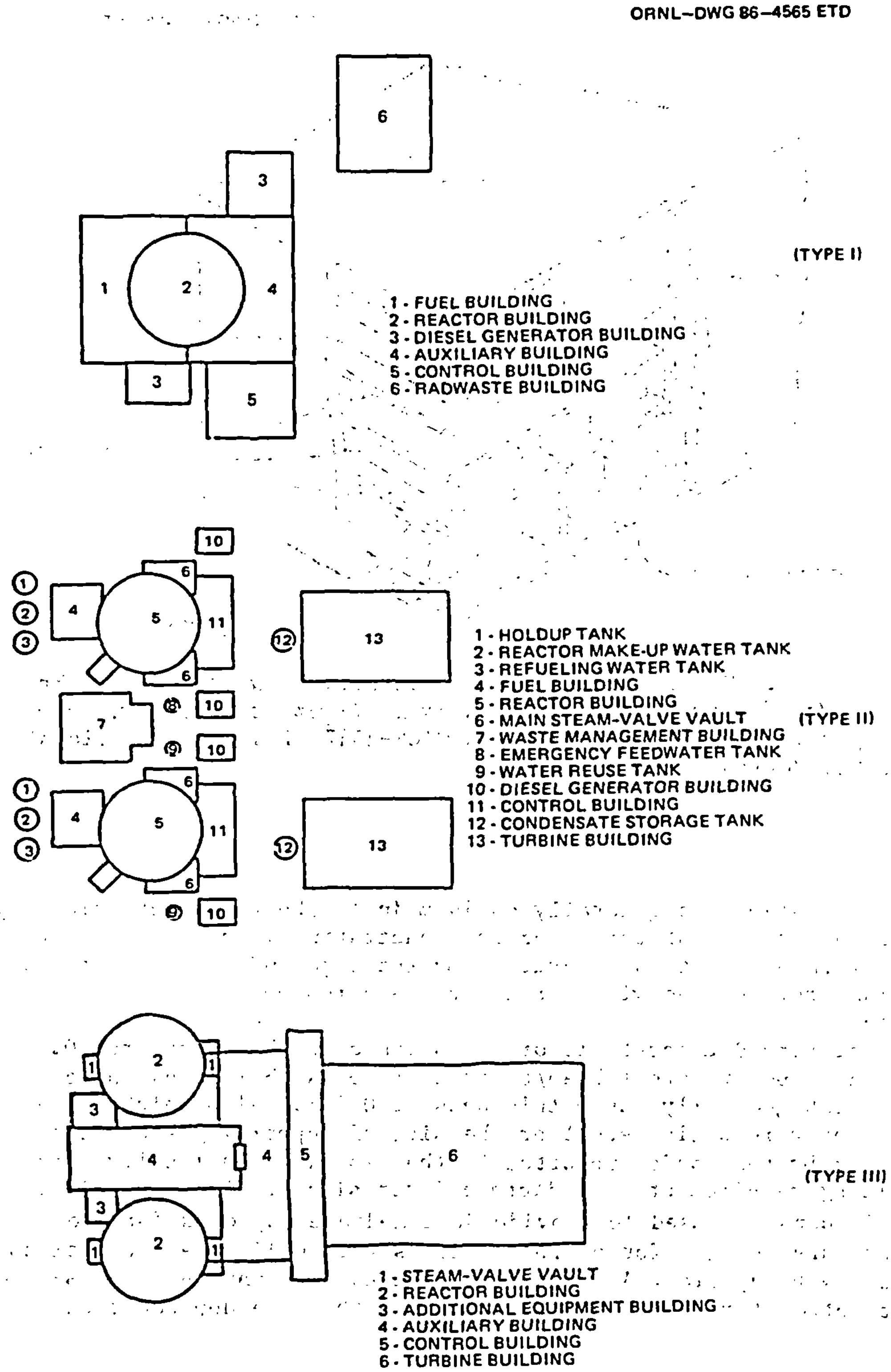


Fig. 14. Generic nuclear power plant building layouts. *Source:* E. Endebrock et al., *Margins to Failure — Category I Structures Program: Background and Experimental Program Plan*, NUREG/CR-2347, Los Alamos National Laboratory, New Mexico, September 1981.

ORNL–DWG 86–4566 ETD

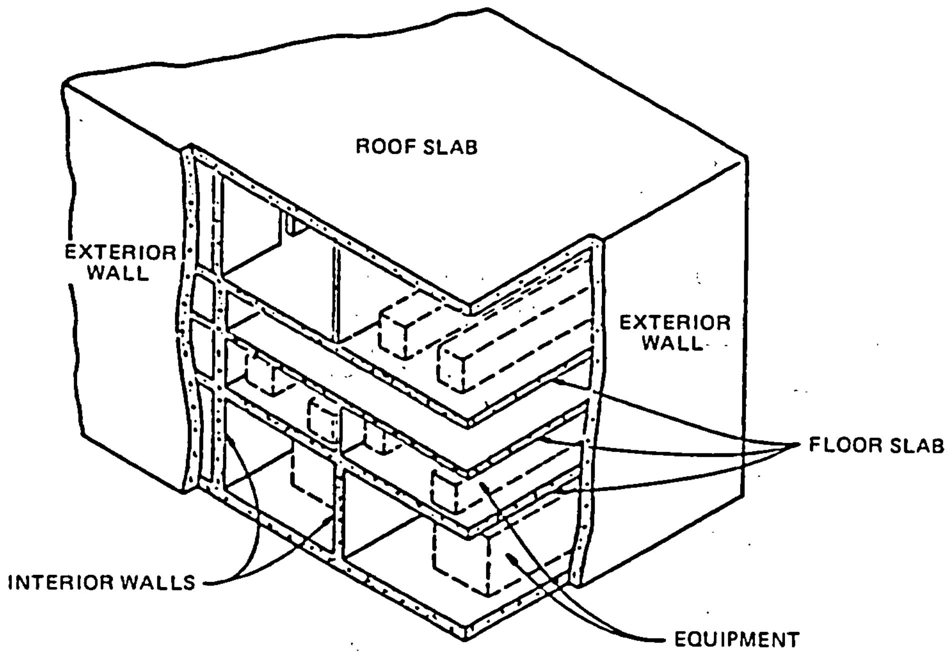


Fig. 15. Typical Category I structure. *Source*: E. Endebrock et al., *Margins to Failure — Category I Structures Program: Background and Experimental Program Plan*, NUREG/CR-2347, Los Alamos National Laboratory, New Mexico, September 1981.

Roof slabs are generally ~0.46 m in thickness as determined by requirements to resist tornado-borne penetrating missiles. The quantity of steel reinforcement is determined by the magnitude of pressure loads arising from tornado winds or natural environment phenomena such as snow or ice.

Reinforced concrete floor slabs range in thickness from 0.3 to 0.9 m. Where a composite steel beam/concrete floor slab is used the concrete floor slab generally has a thickness of 0.3 m. Slab thickness and the amount of reinforcing steel or the size of supporting steel beams are determined by the loads supported by the floor. Occasionally, radiation shielding requirements may dictate floor slab thickness requirements.

Columns are used to provide intermediate supports for floor slabs and primary support for overhead floors when walls are not available or unusually heavy floor loads occur. The columns may be steel sections or constructed of reinforced concrete, with the size dependent on the loading.

References

1. 10 CFR 50.

2. ACI Committee 318, "Building Code Requirements for Reinforced Concrete," ACI Standard 318-71, American Concrete Institute, Detroit, November 1971.

3. ACI Committee 349, "Code Requirements for Nuclear Safety Related Concrete Structures," ACI Standard 349-76, American Concrete Institute, Detroit, June 1976.

4. *Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments) (for comment issue)*, Regulatory Guide 1.142, Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., April 1978.

5. *Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants (LWR Edition)*, Regulatory Guide 1.70 (Rev. 3), Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., November 1978.

6. R. S. Orr and E. G. Hess, "ACI 318 or 349 for Radioactive Waste Facilities," Presented at *Concrete and Cementitious Materials for Radioactive Waste Management Seminar*, American Concrete Institute, New York, November 1, 1984.

7. Joint Technical Committee of the ACI and ASME, "Code for Concrete Reactor Vessels and Containments," *ASME Boiler and Pressure Vessel Code*, ACI Standard 359-83, American Society of Mechanical Engineers, New York, July 1, 1983.

8. U.S. Atomic Energy Commission, "Concrete Containment," Sect. 3.8.1 in *Regulatory Standard Review Plan*, NUREG-0800, Washington, D.C., July 1981.

9. *Material for Concrete Containments*, Regulatory Guide 1.136, Rev. 1, Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., October 1978.

10. "Rules for Construction of Unfired Pressure Vessels," *ASME Boiler and Pressure Vessel Code*, Sect. VIII, American Society of Mechanical Engineers, New York, 1965.

11. D. W. Halligan, "Prestressed Concrete Nuclear Plant Containment Structures," *J. Prestressed Concr. Inst.* 21(5) (September—October 1976).

12. E. G. Silver, "Operating U.S. Power Reactors," *Nucl. Saf.* 26(4), 520—32 (July—August 1985).

13. J. F. Allred et al., "Sequoyah Ice Condenser Containment Structure," *J. Power Division* 99(PO1), 19–40 (May 1973).

14. *Containment Performance Working Group Report (Draft Report for Comment)*, NUREG-1037, Office of Nuclear Regulation, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.

15. M. H. Shackelford et al., *Characterization of Nuclear Reactor Containment Penetrations Final Report*, NUREG/CR-3855, Argonne National Laboratory, Argonne, Ill., February 1985.

16. C. P. Tan, *A Study of Design and Construction Practices of Prestressed Concrete and Reinforced Concrete Containment Vessels*, F-C2121, The Franklin Institute Research Laboratories, Philadelphia, August 1969.

17. J. Kauakami et al., *Reliability Assessment of Indian Point Unit 3 Containment Structure*, NUREG/CR-3641, Brookhaven National Laboratory, Upton, N.Y., January 1984.

18. S. Sharma et al., *Failure Evaluation of a Reinforced Concrete Mark III Containment Structure Under Uniform Pressure*, NUREG/CR-1967, Brookhaven National Laboratory, Upton, N.Y., September 1982.

19. N. W. Edwards, "Design Loads, Loading Combinations and Structural Acceptance Criteria for BWR Containments in the United States," Paper J2/1, *5th International Conference on Structural Mechanics in Reactor Technology*, Berlin, Germany, August 13–17, 1979.

20. S. Sharma et al., *Ultimate Pressure Capacity of Reinforced and Prestressed Concrete Containments*, NUREG/CR-4149, Brookhaven National Laboratory, Upton, N.Y., May 1985.

21. J. K. Aldorsebaes and K. Wiendner, "Containment Structure for Trojan Nuclear Plant," *J. Power Division* 97(PO2), 351–66 (March 1971).

22. K-Y Chu and A. A. Broussard, "Nuclear Containment Mat Design Concepts," *J. Power Division* 100(PO1), 85–96 (July 1974).

23. A. Wachrameeff and R. D. Chellis, "Concrete Usage in Atomic Power Reactor Support," *J. Am. Concr. Inst.* 59(8), 1081–94 (August 1962).

24. E. Endebrock et al., *Margins to Failure — Category I Structures Program: Background and Experimental Program Plan*, NUREG/CR-2347, Los Alamos National Laboratory, Los Alamos, N.M., September 1981.

3. PERFORMANCE OF CONCRETE COMPONENTS IN BOTH NUCLEAR AND NON-NUCLEAR APPLICATIONS

Reference 1 identifies five broad classes of "deterioration influences" that can impact concrete structures: (1) deterioration caused by a change in environment (acid rain, sulfate-bearing groundwater, air pollution); (2) deterioration caused by a change in concrete properties (long-term behavior of concrete components); (3) short-term deterioration (nonaging-related problems detected early in the service life of the structure that would require either acceptance, repair, or removal of service); (4) failure through human error in design or construction (problems that should be discovered through quality assurance programs); and (5) deterioration that may result from sudden and/or unusual events (serious accident or environmental type situations that would require an investigation to establish structural integrity prior to a return to service). Although only classes (1) and (2) are in a strict sense related to aging phenomena,* the scope of this review of concrete component performance was not restricted to these two classes. The objective was also to obtain a broader spectrum on concrete performance so that in cases where problems developed, they could be categorized or trended. More specifically, these areas were addressed: (1) concrete longevity, (2) history of performance of concrete components in nuclear-safety-related applications, and (3) review of problems experienced with concrete material systems in both general civil engineering and nuclear components. Results of this review were then used to provide trending observations on concrete component performance.

3.1 Concrete Longevity

Concrete has been utilized as a construction material for several thousand years, probably starting with the use of gypsum mortars by the Egyptians to fabricate structures such as the Pyramid of Cheops in ~3000 B.C.[3] Although the longevity of concrete is attested to by the existence of the Colosseum in Rome and the Pont du Gard at Nîmes, which is still capable of supporting modern road traffic after 2000 years,[†4]

*Aging, as defined in NUREG-1144,[2] represents the cumulative changes with the passage of time that may occur within a component or structure because of one or more of the following factors: (1) natural internal chemical or physical processes during operation, (2) external stressors caused by storage or operating environment, (3) service wear including changes in dimensions and/or relative positions of individual parts or subassemblies by operational cycling, (4) excessive testing, and (5) improper installation, application, or maintenance.

†Aquaduct and Gard River Bridge were built about 2000 years ago. The first level of the bridge was transformed into a road bridge in the 13th century. A new bridge was constructed adjacent to the existing water bridge in 1747.

current hydraulic "portland" cement concretes have only been in existence since 1824 when Joseph Aspdin obtained a patent. Despite the existence of portland cement for over 160 years, relatively little documented information is available on the aging of concrete structures.* Three instances, however, have been identified in which concrete structures were examined after an extended period of service: (1) reinforced concrete bridge in Switzerland, (2) Portland Hall concrete wall in England, and (3) 30-year-old prestressed concrete beams in Belgium. Also, one reference was identified in which concrete properties were determined over an extended period of time (50 years).

In 1889–1890 a concrete bridge was built in Wildegg, Switzerland, using the Monier system (reinforced concrete arch with a 37.2-m span length).[5] After 84 years of service the bridge was scheduled for removal, thus permitting an opportunity to determine the behavior of the bridge structure and to evaluate properties of the concrete and reinforcing steel. Load capacity of the bridge was found to be higher than anticipated with measured and calculated deflections agreeing quite well. Fifty 5-cm-diam drill cores revealed the concrete to have a compressive strength of 60.8 MPa, to be well compacted, hardly damaged by frost, and having protected the reinforcing steel from corrosion with most of the reinforcement free of rust. Composition of the hydrated cement paste was investigated by means of a scanning electron microscope, chemical and thermal analyses, and X-ray diffraction analysis. Results of these analyses revealed that the 84-year-old hydrated cement paste generally appeared chemically and mineralogically quite similar to a cement paste hydrated only a few years and that the cement paste was almost completely hydrated. The investigation concluded that the properties of the portland cement concrete had not been adversely affected, even after more than 80 years of service.

A piece of portland cement concrete was obtained for examination from a precast concrete wall built in 1847 in front of Portland Hall, Gravesend, Kent, England.[6] Examination of one cut face of the concrete with phenolphthalein revealed that the concrete had been carbonated to a depth of only 5 mm, indicating that the concrete interior was still highly alkaline. Observations also showed that a coarsely ground cement in concrete having a low water-to-cement ratio and well compacted and cured may obtain a long-term, steady increase of denseness, strength, and durability due to slow continuous hydration of residual C_3S and C_2S. From these results, it was concluded that it was possible to make concrete of several hundred years' durability.

Two of the prestressed concrete beams forming part of the Desmet Bridge at Ghent, Belgium, were tested to failure under static loading after 30 years of service.[7] The beams were 28.8 m long, had a flange width of 5.15 m, a web 0.175 m thick, and a depth of 1.12 m. On loading, the safety factor of the beams was determined to be 2.2. Concrete

*Considerable research has been conducted on concrete durability, but current interest is more related to deteriorating influences that can impact the ability of a concrete component to provide additional service beyond the initial 40-year operating license of a nuclear plant.

strength was found to be 77% greater than its 28-d value, and the 7-mm-diam prestressing wire quality was essentially unchanged. Tests demonstrate that even after 30 years of service the beams were still in a satisfactory condition.

Despite the extensive amount of information available in the literature reporting results of research conducted on concrete materials and structures, only limited data are available on the long-term (40 to 80 years) properties of portland cement concrete that are of interest to this study. Where concrete properties are reported for conditions that have been well-documented, the results are generally for concretes having ages <5 years or for specimens that were subjected to extreme, nonrepresentative, environmental conditions such as seawater exposure. Reference 8, however, is an exception because test results have been obtained from concretes prepared under well-documented conditions for ages up to 50 years. In this study, several mortar and concrete mixes were prepared from a variety of aggregate and cement materials. After fabrication, the specimens were moist cured for either 14 or 28 d and then either stored indoors (16 to 27°C), outdoors (−32 to 35°C), or underwater. Results obtained from this study showed that (1) the compressive strength of comparable concrete cylinders stored outdoors made with high C_3S content and low surface area cements generally increased as the logarithm of age to 50 years, but concrete made with lower C_3S content and finer particle size cements appeared to reach peak strength at ages between 10 and 25 years with some strength retrogression thereafter; (2) concretes stored indoors exhibited little change in compressive strength for ages from 2 to 10 years, but thereafter showed strength increases from 30 to 70% at 50 years* (Fig. 16); (3) modulus of rupture at 50 years was approximately one-eighth the compressive strength for concrete stored outdoors and one-sixth the compressive strength for concrete stored indoors; (4) all concrete stored outdoors, despite undergoing ~25 cycles of freezing and thawing each winter, showed remarkably good weathering qualities during the 50-year exposure period; and (5) weight and volume changes during the 50-year storage period were small with the largest values obtained from specimens stored indoors (1.75% weight loss, 0.059% contraction).

3.2 History of the Performance of Concrete Components in Nuclear-Safety-Related Applications

As noted in Chap. 2, principal applications of concrete to nuclear-safety-related components has been in the form of containments, containment base mats, and biological shield walls. Other applications include balance-of-plant facilities. In the following sections an overview of the performance of these components will be presented. Specific items

*Companion cylinders stored outdoors exhibited strength increases from 10 to 40% during the 10- to 50-year period.

ORNL-DWG 86-4567 ETD

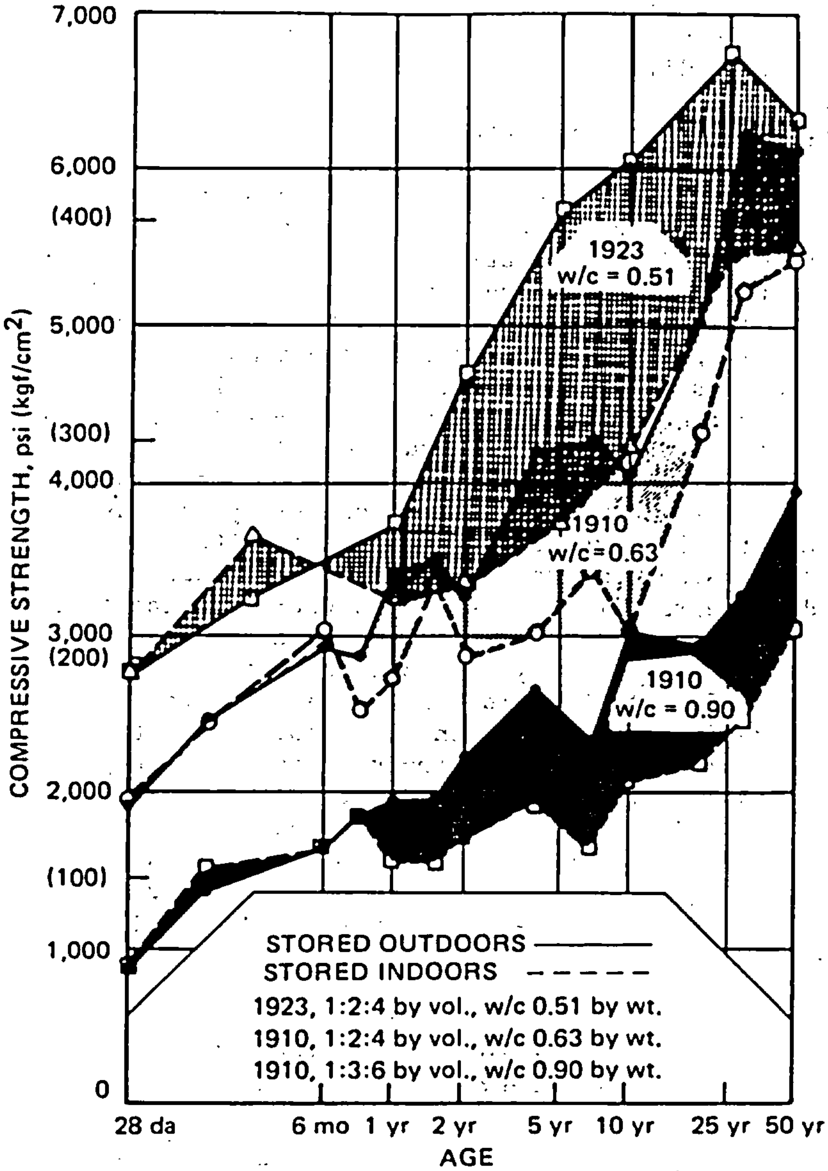


Fig. 16. Compressive strength—age relations for Series A and B concrete stored indoors and outdoors. *Source*: G. W. Washa and K. F. Wendt, "Fifty Year Properties of Concrete," *J. Am. Concr. Inst.* 72(1), 26 (January 1975).

addressed will include: prestressed concrete containments (PCCs),* prestressed concrete reactor vessels (PCRVs), and miscellaneous reactor structures.

3.2.1 Prestressed concrete containments

In-service surveillances of PCCs are conducted to ensure structural integrity and to identify and correct problem areas before they become critical. Results obtained from these investigations are invaluable not only for verifying that the containments will meet their intended function, but also from the standpoint of establishing performance histories. Utilizing the component history data obtained for a containment, aging trends should be relatively easy to establish and should significantly simplify the evaluation required for life extension. Information of this type is available from surveillances of PCCs conducted in the United States, France, and Sweden.[10]

3.2.1.1 United States. Performance of prestressing systems has generally been exemplary with the few documented problems† or abnormalities being minor in nature.‡ All the surveillance reports concluded that the respective containments were in good condition.[11,12] Except for one instance in which a significant amount of water was found in several tendon ducts,§ little water has been found during inspections. A few instances of wire corrosion have been reported, but these generally did not result in wire breaks and were so minor that component replacement was not required. The general conclusion was that corrosion had occurred prior to filling the ducts with corrosion inhibitor. Incidents of incomplete filling of tendon ducts with corrosion inhibitor and improper tendon stressing have been reported, but neither have caused any serious difficulties and have since been corrected. Missing buttonheads have been discovered on some wires of buttonheaded prestressing systems;

*Only prestressed concrete containments are addressed because they constitute a majority of the concrete containments in existing plants, and their performance is well-documented because of U.S. surveillance requirements[9] for the prestressing tendons.

†Problems documented are those that have generally been discovered during in-service inspections. Problems identified during construction or early in the containment life are detailed in Sect. 3.3.2.

‡Results of a review[11] of the durability performance of post-tensioning tendons used in conventional civil engineering structures (pavements, bridges, etc.) have produced a similar result. Of the over 30 million tendons used throughout the western world (to 1978), the number of corrosion incidents (200 in completed permanent structures) represents an extremely small percentage (0.0007). All of the corrosion-related incidents identified were related to either ill-conceived detailing, poor construction, or contaminants causing corrosive environments.

§Despite the presence of water, corrosion was found to be very minor, and steps were taken to eliminate recurrence. This demonstrates the effectiveness of corrosion inhibitors even under severe conditions.

however, the number of noneffective wires allowed in the design for a tendon or group of tendons was not exceeded.

3.2.1.2 France. Prestressed concrete containments in France utilize grouted tendons except for four vertical tendons* in the first unit to be built at a site. As of 1982 ten leakage and structural pressure tests had been conducted. All leakage rates were within satisfactory limits and the response of the containment structure was elastic and consistent with the design analysis. The few cracks that occurred during construction were due to shrinkage and did not grow during the pressure test.

3.2.1.3 Sweden. As of 1982, six prestressed concrete containments were in operation. Five of the containments utilize ungrouted tendons. Periodic in-service inspections of the prestressing system are carried out mainly in accordance with Ref. 9. For inspections reported in Ref. 10, no serious corrosion, broken wires, or missing buttonheads have been observed. Small amounts of water have been found in a few grease caps and tendon ducts, but physical tests of the grease showed that it was in good condition, and tensile and bending tests of the wires yielded good results. Steel properties have not been affected by time, and prestress losses were generally less than expected.

3.2.2 Prestressed concrete reactor vessels

As of 1982 25† PCRVs are operating, under construction, or planned.[10] Experience from surveillance of PCRVs is available from the United Kingdom, France, and the United States.

3.2.2.1 United Kingdom. Checks on residual anchorage force are made on at least 1% of the tendons in each vessel during an inspection. In general, no problems have been encountered with loss of tendon load, although individual tendons at Oldbury and Hinkley have been found to be at a lower load than expected.‡ Anchorage condition has been good except some slippage was observed where tendons were removed for corrosion examination and replaced. A small number (147 out of 320,000) of missing buttonheads were found at the Dungeness "B" vessels. Major corrosion instances occurred during construction with the main causes being a combination of moisture, chloride contamination, and impressed electrical currents (probably due to improper grounding of dc welding machines). No instances of corrosion were revealed that could be considered serious enough to warrant tendon replacement. Extensive concrete surface exams in which all cracks were mapped and their lengths and widths noted have revealed all cracks to be narrow (<0.1 to 0.2 mm), of no structural significance, and associated with either drying shrinkage, construction joints, or steel embedments. Foundation settlement has been small compared with allowable limits for settlement and tilt.

---

*Tendon ducts filled with grease.

†Marcoule G2 and G3 in France have been closed down.

‡Some tendons at Hinkley were retensioned, and subsequent inspections have revealed no further significant loss of load.

3.2.2.2 France. Performance of the French PCRVs,[13] of which one has been in service over 15 years, has been satisfactory.* Each structure is equipped with instrumentation to monitor: forces in selected prestressing tendons (most are grouted); horizontal outside diameter at selected elevations; and deflections, overall tilt, concrete temperature, and unit deformations. Visual inspections are also made. Except for a few isolated locations, temperatures have been found to be within permissible tolerances. Concrete dimensions have tended to decrease slightly because of shrinkage and creep. Relatively few extremely fine surface cracks are visible.

3.2.2.3 United States. Performance of the concrete vessel at Fort St. Vrain has been good up to 1984 with surveillances performed continuously since the initial proof test in 1971. Structural response of the vessel to pressure changes was found to be essentially linear, with strains and deflections being in general agreement with those predicted by elastic analysis. A number of exceptions to the expected temperature levels were found in small areas at discontinuities at penetrations or internal attachments to the liner caused by shortcomings of the insulation or cooling system, but additional design assessments determined that these temperature levels were acceptable. However, during a scheduled 1984 tendon surveillance, certain PCRV tendons had broken, and corroded wires were discovered.[14,15] Failure was caused by general corrosion and stress corrosion cracking resulting from the presence of acetic and formic acids formed by microbiological attack on the anticorrosion grease. Most of the corrosion failures were observed near the top anchor assembly of longitudinal tendons and near the anchor assembly on bottom crosshead tendons. An analysis to evaluate the integrity of the PCRV with degraded tendons found that the reactor vessel was capable of withstanding the operating pressures with the degraded tendons as determined at that time.[15] The licensee has proposed halting degradation by filling the tendon sheaths with an inert nitrogen blanket and revising the surveillance program to increase the frequency of the visual inspection and lift-off tests. The surveillance program will compare an uncorroded tendon control group with a corroded tendon group to establish the effectiveness of the corrosion-arresting method and the trend in tendon wire degradation. Based on these provisions, an updated Fort St. Vrain Tendon Corrosion Safety Evaluation has recommended that plant restart be permitted.[16]

3.2.3 Miscellaneous reactor structures

Probably one of the most documented surveys of the condition of concrete components (other than containments) in nuclear power plants that

---

*Extensive corrosion of longitudinal tendons in the Marcoule G2 and G3 containments was detected during periodic surveillances conducted in 1962–63. Cause was attributed to excess humidity in conduits. Subsequent corrosion was arrested by changing the conduit air sweeping system from periodic to continuous, which maintained the relative humidity at 10%. Circumferential cables protected by several layers of bituminous material exhibited only minor rust. Marcoule reactors are presently decommissioned.

have been in service for several years is contained in Ref. 17, which is related to extension of the service life of the Savannah River Plant (SRP) reactors. Reactors at SRP have operated for ~25 years with three plants still operating (P, K, and C), one shutdown in 1964 (R), and one that operated to 1968 that is being considered for restart (L). Accumulated fast fluence in the reactor vessel walls was $\sim 2 \times 10^{21}$ neutrons/cm^2 ($E > 0.1$ MeV). The tank wall temperature at full power was 100 to 120°C, and the maximum thermal fluence was 2×10^{22} neutrons/cm^2.

Reactor buildings in the P, K, and C areas were inspected for signs of structural distress and to determine if they would support operation for the next 20 to 30 years. The reactor support and biological shield, actuator towers, crane haunches in the process rooms, and crane maintenance areas were inspected, and all were found suitable for continued support of reactor operation for the next 20 to 30 years. Minor random cracking was found in the P reactor building on all surfaces of the biological shield wall. Some hairline cracks were also found running from the wall to the edge of the crane haunches, and some cracks were also noted in the actuator tower. In the K building minor cracking was found in the wall, grouting had failed under some of the remotely controlled charge and discharge crane rail support plates, some hairline cracks were noted in the vertical face of the crane haunches, and a vertical crack was found in the actuator tower. The most extensive cracking was found in the biological shield of the C reactor building (Fig. 17). Additional

ORNL PHOTO 4009–86

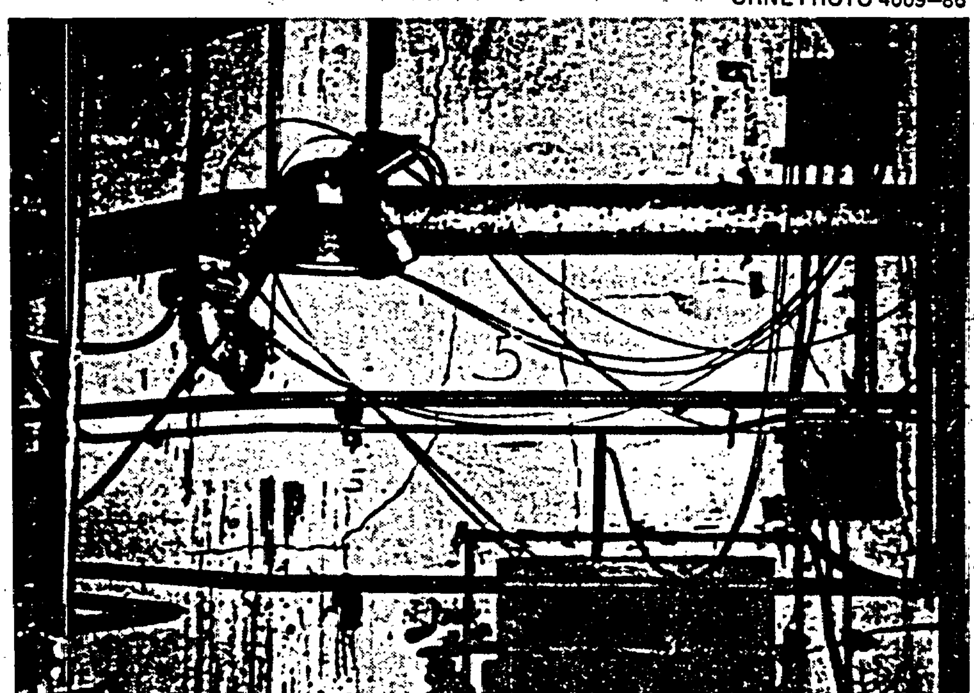


Fig. 17. C reactor cracks in biological shield at SRP. *Source*: D. A. Ward, *Extended Service Life of Savannah River Plant Reactors*, DPST-80-539, Savannah River Plant and Laboratory, Aiken, South Carolina, October 1980, p. 29.

cracking in the C building was found beneath the anchor point of one beam from which a piping support was suspended in a pump room, spalled and cracked grout was found under one side of the charge and discharge crane rail supports, and one hairline diagonal crack was found in the actuator tower.

Consultants reviewed cracking in these buildings to assess the feasibility of repair. Although it was determined that the cracks were not associated with a structural load condition, they should be monitored with time to verify that they are inactive. Testing of core samples was also recommended so that the cracks could be diagnosed more closely (depth) and petrographic studies conducted. After repair of the cracks by epoxy injection, the structure should be able to continue to meet its functional requirements for an additional 25 years.

3.3 Problems Experienced with Concrete Material Systems in Both General Civil Engineering and Nuclear Components

Results presented in the previous two sections demonstrate that concrete fabricated from good quality materials and exposed to a normal atmospheric environment has indefinite longevity and that the general performance of concrete in nuclear-related applications has been exemplary. Problems do occur, however, that can result in concrete distress. To trend the type of problems that have been experienced with concrete materials and structures, the literature was reviewed with respect to both general civil engineering structures and light-water reactor (LWR) applications.

3.3.1 General civil engineering components

Reference 18 presents results of 277 cases of errors in concrete structures obtained from a survey of consulting engineers and government agencies in North America conducted by American Concrete Institute Committee 348. Approximately three-quarters of the 277 cases of error reported were actually discovered by the structure with 39 cases of collapse and 172 cases of distress, cracking, spalling, leakage, settlement, deflection, or rotation. About one-half of the errors originated in the design and the other one-half were due to faulty construction with each phase responsible for approximately the same number of collapses. Design errors were far more prevalent than construction errors in elements requiring close attention to detail (connections, joints, and prestressed members). Three-quarters of the errors caused by faulty construction were detected during construction and over one-half of the errors resulted in failure or distress. Installation of reinforcement and concreting procedures accounted for a majority of the construction errors. Design errors, however, were generally not detected until occupancy, with most resulting in serviceability problems. Design errors resulted largely from improper consideration of details or shrinkage and temperature effects. A limitation of the study[18] was that the information presented is strongly biased toward errors that escaped detection until revealed by the structure and thus does not present a true picture of the error detection process of the review-check system. Also, the survey favored those structures and serviceability characteristics that reveal themselves in a short period of time and thus does not represent the actual incidents of concrete deterioration.

A similar study[19] reviewed ~800 European failures with the focus being on the most efficient way to maintain a given level of structural safety. Results of the survey indicated that few structures actually fail in use. Where failures did occur, the type of structures involved included general buildings (52%), industrial buildings (22%), highway construction (11%), hydraulic construction (7%), fallout shelters (2%), and unknown (6%). Primary components initiating the failure were the structure (44%), interior works (19%), technical installations (11%), secondary construction (9%), construction equipment (7%), excavation pit (5%), and unknown (5%). With respect to time of discovery of failure, 52% were discovered during construction, 45% during occupation, and 3% during demolition. Of the 384 cases of structural failure, 63% resulted in sudden failure (loss of equilibrium, rupture with collapse, and rupture without collapse) and 37% in unsatisfactory conditions (excessive cracking and excessive displacements). In some cases, the structure itself initiated the failure because of unfavorable influences of the natural environment and incorrectly introduced factors either in the planning or construction phase. Errors in the planning phase occurred primarily in conceptual work or during structural analysis. Both the engineer and contractor were involved, each committing errors because of insufficient knowledge or ignorance. Only very few errors were unavoidable, and in a majority of cases additional checking would have helped considerably. From these results it was concluded that a primary deficiency in structural safety was attributed to insufficient data checking.

3.3.2 LWR concrete components

Results presented in Sect. 3.2 indicate that in general the in-service performance of concrete materials and components in nuclear-safety-related applications has been very good. This to a large degree can be attributed to the effectiveness of the quality control/quality assurance programs[20] in detecting potential problems (and the subsequent remedial measures) prior to plant operation.* To obtain information on the type of problems that have been experienced (detected) with LWR components, computer searches of Licensee Event Reports (LERs), the Nuclear Plant Reliability Data System (NPRDS), the Nuclear Power Experience (NPE) data base, and Construction Deficiency Reports (CDRs) have been conducted. Information has also been obtained from the DOE/RECON computerized information retrieval system and appropriate journals (Prestressed Concrete Institute, American Society of Civil Engineers, Engineering

*Additional information on the effectiveness of structural concrete quality assurance practices in nuclear power plant construction is presented in Ref. 20, which reviewed nine nuclear and three fossil fuel plant construction projects.

News-Record, etc). Obviously, because of the sheer magnitude of documentation available for even one plant, all problem areas were not determined. However, the anomalies identified are characteristic of those problems that occur and thus provide trending information.

Figure 18 summarizes results of the survey according to problem type (concrete cracking, tendon failure, etc.), and an annotated problem listing is presented in Appendix B. The majority of problems were related to concrete cracking, concrete voids, or honeycombing, and concrete compressive strength values that were low relative to design values at a specific concrete age. In almost all cases, the concrete cracks were considered to be structurally insignificant or easily repaired using techniques such as epoxy injection. Voids and honeycombed areas were restored by removing faulty materials and making repairs using accepted procedures such as grout injection, drypacking, or shotcreting. In a few instances low-strength concrete materials had to be removed and replaced, but in the majority of situations either the in-situ strength was determined to be in excess of design requirements or subsequent tests conducted at later concrete ages achieved acceptable strength levels.

Although the vast majority of the problems detected did not present a threat to public safety or jeopardize the structural integrity of the particular component, five incidences were identified that if not discovered and repaired could potentially have had serious consequences. These incidences were all related to the concrete containment and involved two dome delaminations, voids under tendon bearing plates, anchor head failures, and a breakdown in quality control and construction management. Note that these incidences were attributed either to design, construction, or human errors, but not to aging.

After 110 of 165 tendons in the containment dome of Turkey Point 3 had been tensioned, it was noted that sheathing filler was leaking from a

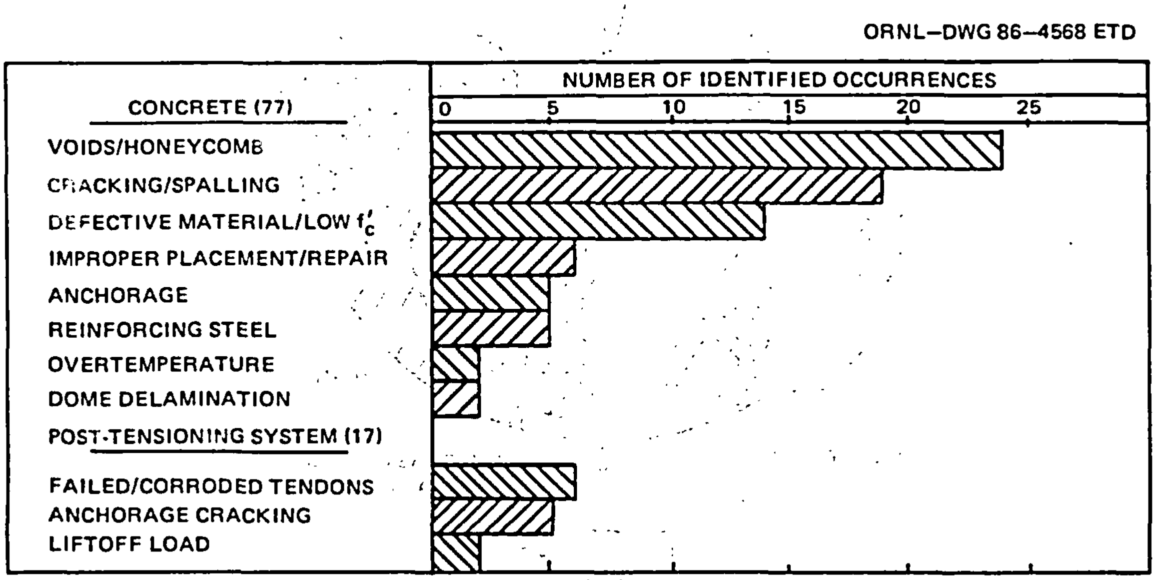


Fig. 18. Distribution of LWR concrete component problem areas.

crack in the dome surface.[21] A small amount of concrete was chipped away adjacent to the crack to reveal a crack plane parallel to the surface (delamination) with evidences of sheathing filler flow on the delaminated surfaces. Five days later, a small bulge was noted in the dome surface, which when broken through revealed a delamination at a depth of about 12.7 mm. Exploratory chipping revealed that the delamination became thicker as the dome center was approached, reaching ~102 mm when chipping was terminated at a 4.6-m radius. Soundings were taken with a Swiss hammer and steel sledge hammer to indicate the area affected. Sixty-five 102-mm-diam cores were drilled to estimate the depth and extent of delaminations. The core samples revealed that the depth and extent of delaminations was considerable and symmetrical, the delaminations appeared to have originated at a meridional construction joint, and many of the cores had sheathing filler in them as well as showing signs of multiple delaminations. Dome tendons, of which all but two had been tensioned at this time, were detensioned. The delaminated concrete was removed by chipping guns and jack hammers. As shown in Fig. 19, the delaminations covered >50% of the dome and reached depths to ~0.4 m. Exposed concrete

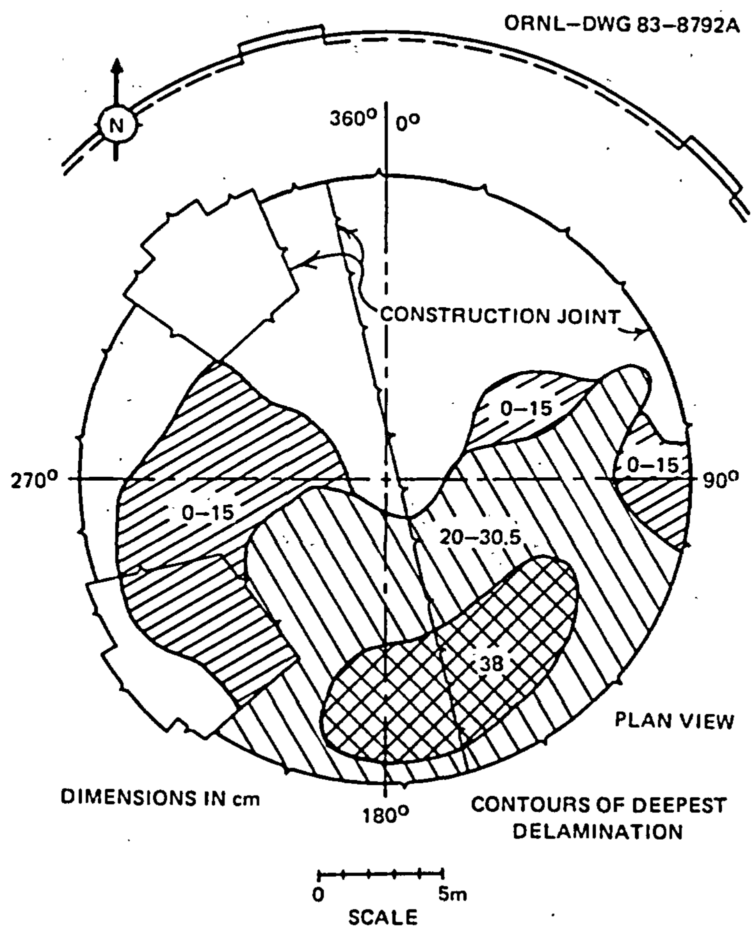


Fig. 19. Extent of dome delamination for Turkey Point Unit 3.

surfaces were cleaned using a high-pressure air-water blast technique. A system of radial rock anchors was installed to assist in providing radial forces on the replaced concrete. The concrete was then replaced using extreme care to ensure good bond with the existing concrete. Tendons were retensioned using a new sequence to reduce unbalanced loadings during the prestressing operation. An ensuing structural integrity test revealed no recurrence of delaminations. The cause of the delamination was determined to be attributed to insufficient contact area in the southern portion of the meridional construction joint and around the ventilation blockouts, together with unbalanced posttensioning loads.

Delamination of the containment dome also occurred at Crystal River Unit 3.[22] Discovery of the delamination occurred 2 years after completion of concrete placement and 1 year after tendon tensioning when electricians could not secure some drilled-in anchors to the top surface of the dome. Further investigation revealed an area of dome concrete that sounded hollow when struck with a hammer. Exploratory holes were then cored and concrete samples removed. Results of this investigation revealed that the dome had delaminated over an area having a diameter of ~32 m (Fig. 20) and had a maximum thickness of delamination of 0.38 m

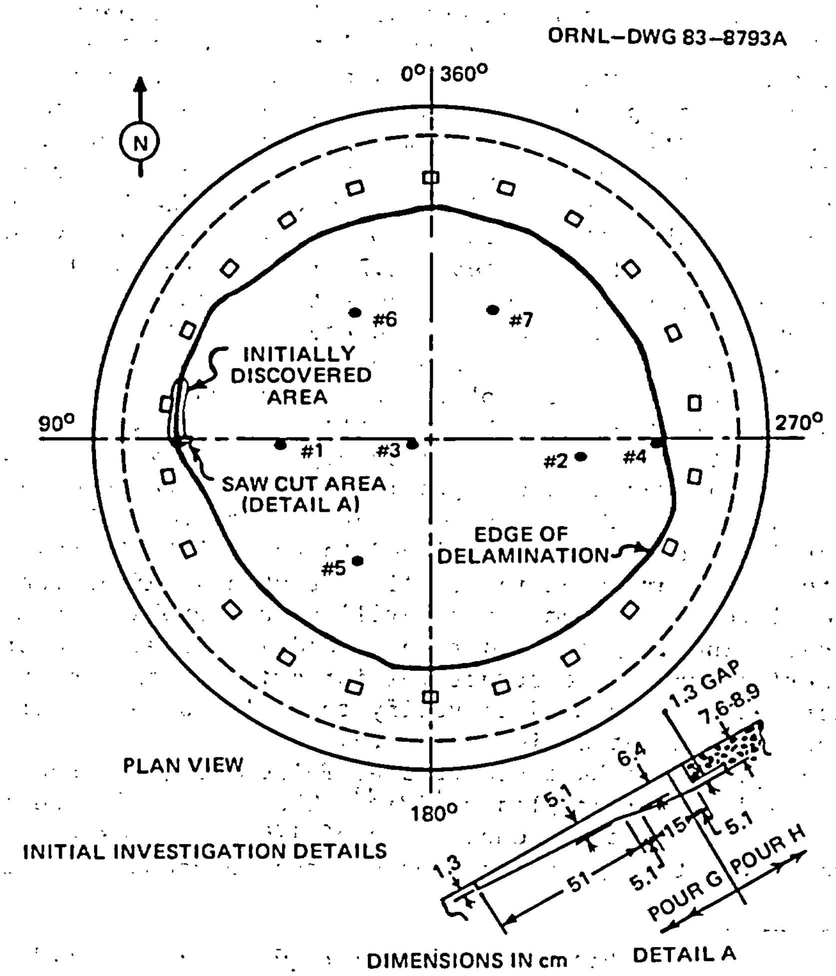


Fig. 20. Extent of dome delamination for Crystal River Unit 3.

near the apex with a gap of ~51 mm between layers. Analysis of the structure indicated that it was acting as a 0.62-m prestressed concrete dome having a 0.31-m unstressed concrete cap and that although it was safe for any normally anticipated loads, it would require repair to withstand accident conditions. While the delaminated cap was still in place, ~1850 radial holes 25.4 mm in diameter were drilled into the dome to provide a means for further inspection, to serve as grouting and venting holes, and to provide access for placing radial reinforcement. The delaminated cap was then removed, and cracks were repaired by pressure injecting a low-viscosity epoxy. Nonprestressed meridional and hoop reinforcement was provided to enhance the membrane and tensile capacity of the structure and to control cracking. Concrete materials for the new cap were the same as those in the original concrete. After concrete placement and curing, 18 tendons that had been detensioned to obtain strain and deformation data were retensioned, and a structural integrity test was successfully conducted. Based on analytical and experimental evaluations, it was concluded that radial tension stresses combined with biaxial compression stresses initiated laminar cracking in the concrete that had lower than normal tensile strength and limited crack-arresting capability.

At Calvert Cliffs nuclear plant during posttensioning, 11 top bearing plates of the 204 vertical tendons of Unit 1 containment and 1 bearing plate of Unit 2 containment depressed into the concrete.[23] Ten of the Unit 1 plates had depressed from ~0.8 to 4.8 mm with the depressions generally on the inside plate edge. However, when the last tendon was stressed the upper bearing plate deformed and sank ~25.4 mm along its inside edge. Eight months later this upper bearing plate was removed to reveal that the plate was supported on the outside edge by concrete occupying only ~20% of the total area and on the inside edge by the upper reinforcing bar, which had deflected ~12.7 mm. The revealed void was deepest (~305 mm) next to the trumpet and extended outside the bearing plate area. Tension was then released on one of the tendons whose plate had depressed 4.8 mm and the plate removed to reveal concrete only in contact over about one-third the plate area. The void was deepest (178 to 203 mm) adjacent to the trumpet. At this point, it was decided that a comprehensive investigation of all tendon upper-bearing plates was required. Using sound transmission, probing, and air pressure techniques, ~190 bearing plates on each unit were identified as possibly having voids. Affected tendons were detensioned to about 3.4 MPa and voids repaired by either pressure grouting or flow grouting. After repairs a number of the plates were examined by drilling and probing or using air pressure to determine if grout had been placed in the area under the tendon shims. During subsequent structural integrity testing of Unit 1, dial gage micrometers were used to verify that plates were rigid up to 1.15 times the containment design pressure.

Anchor head failures have occurred at Bellefonte,[24] Byron,[25] and Farley Units 1 and 2 nuclear plants.*[26,27] The failures at Bellefonte occurred in eight of the top anchor heads of 170-wire rock anchor tendons just prior to a two-stage grouting process used to anchor the tendons to

*Tendons and anchor heads for all three plants were supplied by the same vendor.

the rock. In one of the anchor head failures 23 of 170 wires in the tendon also failed. Environmental, metallographical, and fractographical studies indicated that the failures were the result of stress corrosion cracking of highly stressed AISI 4140 anchor heads in an aqueous environment of varying pH levels. Also between first- and second-stage grouting, the top anchor heads were covered by grease cans filled with lime water having a pH of 11 to 13. Anchor heads have been replaced with cleaner steel and other improvements have been made. At Byron four anchor heads of 179-wire tendons failed between 1 and 64 d after posttensioning the Unit 1 containment. A thorough study of the chemistry, metallurgy, and fracture phenomena indicated that the failure was caused by tempered martensite embrittlement (vanadium grain refinement process used with temperatures not high enough) and occurred in a decreasing stress field. Anchor head failures at Farley Units 1 and 2 are of recent vintage and unique from the standpoint that the failures occurred about 8 years after posttensioning rather than during construction when failures are most likely. Using magnetic particle testing, cracks in 6 anchors in Farley 1 and 18 anchors in Farley 2 were discovered in addition to the 3 anchors that had already failed in Farley 2.* Laboratory tests have concluded that the cause of tendon anchor head failures was stress corrosion cracking caused by a combination of high-strength low-alloy steel under high stress in the presence of moisture and impurities. (Inspections revealed that although only slight amounts of water were found in three hoop and one dome anchor, 47 of 103 vertical tendons were found with water ranging from trace amounts to 5.7 L.) All cracked and failed anchors have been replaced and grease has been applied using an improved procedure to prevent the water-caused problem from recurring. In addition all tendons from the same lot as the failed tendon have been inspected, and 20 of the vertical tendons have been replaced with a superior heat material.

Safety-related concrete work at Marble Hill Units 1 and 2 was halted by the NRC because of quality control and construction management inadequacies.[28] Reasons for the halt in construction were (1) an excessive amount of honeycomb and air voids with ~4000 patches existing ranging in size up to several square meters in area, (2) imperfections in many instances had been improperly repaired and/or unacceptable materials utilized, (3) quality control records traceable to repairs were either nonexistent or otherwise inadequate, (4) personnel responsible for repairs were inadequately trained and supervised, and (5) the licensee was neither in control nor sufficiently aware of the above circumstances. All patches were required by NRC to be removed and repaired, and independent consultants were retained to provide an assessment of the type and extent of deficiencies in concrete construction, to provide an assessment of any needed repairs or remedial actions, and to provide conclusions regarding the capability of the affected structure to perform its intended function. Based on the independent consultant's investigation, it was concluded that the Marble Hill concrete structures were constructed of

*Design margin is not an issue because the containment has a margin permitting failure of several tendons. What is important is establishing the cause and preventing subsequent failures.

high-quality (strength) concrete materials, but with concrete placement that failed to meet requirements near the surface due to difficulty in properly vibrating the concrete between the form and reinforcement layer. Nondestructive testing results and coring indicated that the internal concrete consisted of homogeneous concrete and was acceptable. Methods used to detect defective concrete and concrete patch areas were consistent with good construction practice. Investigation of a selected number of visible surface defects verified that procedures currently being used to prepare surfaces and repair areas are consistent with good construction practice. Furthermore, it was concluded that if the repair procedures reviewed were followed and high standards of workmanship maintained, the structural integrity and biological shielding requirements of the concrete structures should be met.

3.4 Trending Observations on the Performance of Concrete Components

Although the data base evaluated in the review was somewhat limited, results obtained are considered to be sufficiently representative that some general observations can be made on concrete aging and component performance. When concrete is fabricated with close attention to the factors shown in Fig. 21 (Ref. 29) related to the production of good concrete, the concrete will have infinite durability unless subjected to extreme external influences* (overload, elevated temperatures, industrial liquids and gases, etc.). Under normal environmental conditions aging of concrete does not have a detrimental effect on its strength for concrete ages to at least 50 years.† Review of the performance of concrete components in general civil engineering structures indicates that few structures actually fail in use and that the errors‡ that do occur are predominantly detected during construction. The source of these errors is generally the result of either construction or design detail errors. The overall performance of concrete components in nuclear applications has been very good. With the exception of the anchor head failures at Farley 2, errors detected during the construction phase or early in the structure's life were of no structural significance or "easily" repaired and were nonaging related. The rigorous in-service inspection programs required of nuclear components are achieving their desired objective of uncovering and correcting potential problem areas and provide a valuable

---

*Chapter 4 presents a discussion of environmental stressors and aging factors to which concrete components may be subjected.

†A limit on age for which well-documented data has been identified. The number of concrete structures in existence having ages of 40 to 70 years, with a few in service for thousands of years, indicates that this value is conservative. Also, many structures continue to meet their functional and performance requirements even when conditions are far from ideal.

‡Errors could be significantly reduced by additional quality assurance/quality control procedures.

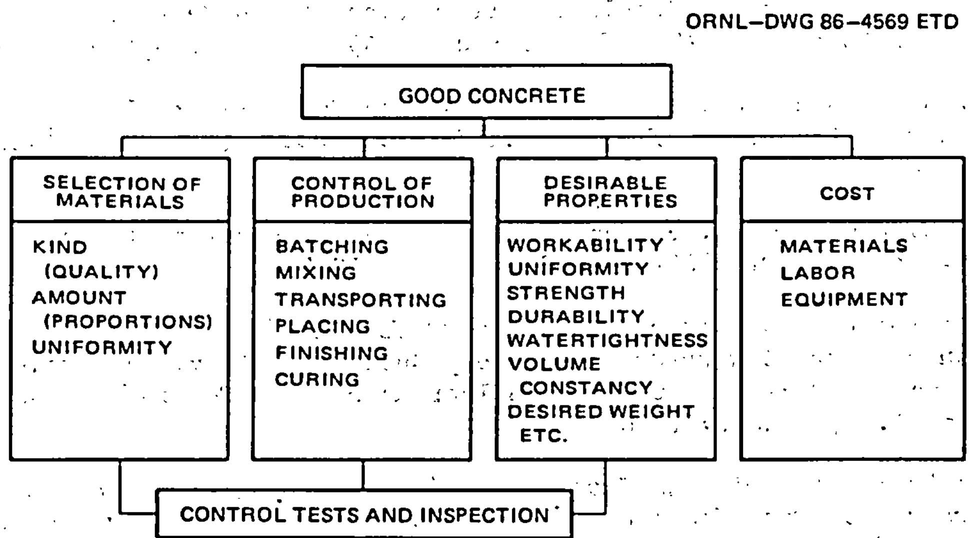


Fig. 21. Factors in production of good concrete. *Source*: G. E. Troxell et al., *Composition and Properties of Concrete*, 2nd ed., McGraw-Hill Book Co., New York, 1968.

source of data for trending component performance. In the one example of a nuclear plant that was identified where component life extension was being considered, the main distress of concrete components identified was cracking. Once the cracks were repaired with a procedure such as epoxy injection, it was felt that the structures should be able to meet their functional requirements for at least an additional 25 years.

References

1. C. A. Negin et al., *Extended Life Operation of Light Water Reactors: Economic and Technological Review*, EPRI NP-2418, vols. 1 and 2, Electric Power Research Institute, Palo Alto, Calif., June 1982.

2. B. M. Morris and J. P. Vora, *Nuclear Plant Aging Research (NPAR) Program Plan*, NUREG-1144, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, Washington, D.C., July 1985.

3. S. Mindess and J. F. Young, *Concrete*, Prentice-Hall, Englewood Cliffs, N.J., 1981.

4. P. E. Halstead, "Causes of Concrete Decay," *The Consulting Eng.* (Supplement) (April–May 1971).

5. J. Gebauer and A. B. Harnik, "Microstructure and Composition of Hydrated Cement Paste of an 84 Year Old Concrete Bridge Construction," *Cem. Concr. Res.* 5(2), 163–9 (March 1975).

6. G. M. Idorn and N. Thaulow, "Examination of 136 Year Old Portland Cement Concrete," *Cem. Concr. Res.* 13(5), 739–43 (September 1983).

7. F. G. Riessauw and L. Taerwe, "Tests on Two 30-Year-Old Prestressed Concrete Beams," *J. Prestressed Concr. Inst.* 25(6), 70–72 (November–December 1980).

8. G. W. Washa and K. F. Wendt, "Fifty Year Properties of Concrete," *J. Am. Concr. Inst.* 72(1), 20–28 (January 1975).

9. *Inservice Surveillance of Ungrouted Tendons in Prestressed Concrete Containment Structures*, Regulatory Guide 1.35 (Rev. 2), Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., January 1976.

10. *Technical Report — An International Survey of In-Service Inspection Experience with Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors*, FIP/3/5, FIP Commission on Concrete Pressure and Storage Vessels, Federation Internationale de la Precontrainte, Wexham Springs, Slough, England, 1982.

11. M. Schupack, "A Survey of the Durability Performance of Post-Tensioning Tendons," *J. Am. Concr. Inst.* 75(10), 501–10 (October 1978).

12. J. R. Dougan, *Evaluation of Inservice Inspections of Greased Prestressing Tendons*, ORNL/TM-8278, Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., September 1982.

13. G. Douillet, "Inspection of Prestressed Concrete Pressure Vessels of Natural Uranium Gas-Cooled Reactors Owned by Electricite de France," pp. 400–17 in *Specialists' Meeting on Design, Criteria and Experience with Prestressed Concrete Reactor Pressure Vessels for Gas-Cooled Reactors*, International Atomic Energy Agency, Lausanne, Switzerland, Dec. 4–7, 1984.

14. D. J. Naus, "Review and Current Status of Fort St. Vrain with Particular Reference to Its PCRV," *Specialists' Meeting on Design, Criteria and Experience with Prestressed Concrete Reactor Pressure Vessels for Gas-Cooled Reactors*, International Atomic Energy Agency, Lausanne, Switzerland, Dec. 4–7, 1984.

15. J. Eggebroten, *Corrosion of PCRV Tendon Wires*, Licensee Event Report 267-005 (Rev. 3), Public Service Company of Colorado, Denver, July 5, 1985.

16. "Fort St. Vrain Tendon Corrosion-Safety Evaluation (Updated)," memorandum from H. L. Thomson, Jr., Division of Licensing, Office of Nuclear Regulatory Regulation, to R. P. Denise, Division of Reactor Safety and Projects (Region IV), U.S. Nuclear Regulatory Commission, July 2, 1985.

17. D. A. Ward, *Extended Service Life of Savannah River Plant Reactors*, DPST-80-539, Savannah River Plant and Laboratory, Aiken, S.C., October 1980.

18. J. Fraczek, "ACI Survey of Concrete Structure Errors," *J. Am. Concr. Inst.* 1(12), 14–20 (December 1979).

19. R. Hauser, "Lessons Learned from European Failures," *J. Am. Concr. Inst.* 1(12), 21–25 (December 1979).

20. J. H. Willenbrock et al., *Final Summary Report: A Comparative Analysis of Structural Concrete Quality Assurance Practices of Nine Nuclear and Three Fossil Fuel Power Plant Construction Projects*, Report No. COO/4120-3, Department of Civil Engineering, The Pennsylvania State University, University Park, Pa., December 1978.

21. *Containment Dome Report Turkey Point Unit 3*, NRC-PDR Docket No. 50-250, Florida Power and Light Co., Miami, Dec. 23, 1970.

22. F. L. Moreadith and R. E. Pages, "Delaminated Prestressed Concrete Dome: Investigation and Repair," *J. Struct. Eng.* 109(105), 1235–49 (May 1983).

23. *Study Report on Vertical Tendon Bearing Plates*, Appendix 50 of Calvert Cliffs Nuclear Power Plant Preliminary Safety Analysis Report, Baltimore Gas and Electric Co., NRC-PDR Docket Nos. 50-317 and 50-319, July 1974.

24. W. E. Berry et al., *Examination of the Cracked Rock Anchor in the TVA Bellefonte Nuclear Power Plant*, Battelle Columbus Laboratories, Columbus, Ohio, 1976.

25. S. E. Presswalla, *Report on the Failure Investigation of Post-tensioning Anchorheads Used in the Byron Nuclear Containment Structure*, NRC-PDR Docket Nos. 50-454 and 50-455, Inryco, Melrose Park, Ill., 1980.

26. C. Hudgins, "The Cause of Tendon Anchor Failures at Alabama Power Co.'s Farley-2," *Nucleonics Week* 26(9), 4 (Feb. 28, 1985).

27. C. Hudgins, "Farley Tendon Problem Blamed on Water as NRC Mulls Generic Impact," *Nucleonics Week* 26(10), 1 (Mar. 7, 1985).

28. A. L. Parme and R. C. Hamm, *Review of the Evaluation of Concrete at Marble Hill Nuclear Generation Station Units 1 and 2*, Report No. IE-124, NRC-PDR Docket Nos. 50-546 and 50-547, Public Service Co. of Indiana, New Washington, Ind., April 28, 1981.

29. G. E. Troxell et al., *Composition and Properties of Concrete*, 2d ed., McGraw-Hill Book Co., New York, 1968.

4. POTENTIAL ENVIRONMENTAL STRESSORS* AND AGING FACTORS TO WHICH LWR SAFETY-RELATED CONCRETE COMPONENTS MAY BE SUBJECTED

Reactors are generally designed for a plant life of about 40 years, which, with an anticipated availability factor of 80 to 90%, yields 32 to 36 full-power years. Over this period of time, changes in concrete's material or reinforcing steel properties in all likelihood will occur as a result of environmental effects such as elevated temperature or irradiation. As noted in the Chap. 3, the changes in properties do not have to be detrimental to the point that the structure has deteriorated and is unable to meet its functional and performance requirements. This is also pointed out by Mather,[1] who notes that when the specifications covering concrete's production are correct and are followed, concrete will not deteriorate. Concrete in many structures can suffer undesirable degrees of change with time because of improper specifications or a violation of specifications. Mechanisms (factors) that, under unfavorable conditions, can produce premature concrete deterioration include (1) freezing and thawing, (2) aggressive chemical exposure, (3) abrasion, (4) corrosion of steel and other embedded material, (5) chemical reactions of aggregates, and (6) other factors (unsound cement and shrinkage cracking).[1] Table 3 (Ref. 1) presents concrete characteristics, environmental characteristics, and the manifestation of deterioration for each of these factors. For concrete components utilized in nuclear-safety-related structures, an additional factor can be added, extreme environmental exposure (e.g., elevated temperature and irradiation).

In nuclear-safety-related concrete components, the relevant degradation factors that can influence component performance vary by application. Potential degradation factors for reinforced concrete containments (RCCs) are related to those that cause deterioration of the concrete or reinforcing steel. For prestressed concrete containments (PCCs), the factors would be the same as for RCCs except that factors that would cause deterioration of the prestressing system would have to be added. Factors affecting containment base mats would also be the same as those for RCCs, plus those contributing to foundation settlement and aggressive chemical attack by the groundwater. Biological shield walls would be susceptible to factors that would produce a loss of concrete strength or shielding efficiency. Table 4 presents a summary of the predominant environmental stressors to which safety-related components in a light-water reactor (LWR) plant could be subjected that may cause an effect leading to deterioration (nonaccident conditions). In the following sections potential deterioration of these components is discussed in terms of factors that

*An extreme load condition is not considered because it is not an aging-related occurrence. If an overload condition occurred, it would be a readily discernible event and require detailed structural inspection and evaluation.

Table 3. Interacting factors for mechanisms producing premature concrete deterioration

Factor that may induce premature deterioration	Characteristic of the concrete	Characteristic of the environment	Manifestation of deterioration
Freezing and thawing	Lack of entrained air in the cement paste or excessively porous aggregate, or both, in saturated concrete	Moisture and freezing and thawing	Internal expansion and cracking
Aggressive chemical attack			
Sulfate attack	Excessive amounts of hydrated calcium aluminates in the cement paste	Moisture containing dissolved sulfates in excessive concentration	Internal expansion and cracking
Leaching	Excessive porosity	Moisture of low pH and low dissolved lime content	Dissolution and removal of soluble constituents
Abrasion	Lack of resistance to abrasion	Abrasive, often in or under water	Removal of material
Corrosion of embedded metal	Corrodible metal and (usually) corrosion-inducing agents in the concrete	Moisture (or moisture and corrosion-inducing agents)	Internal expansion and cracking
Alkali—silica reaction	Excessive amounts of soluble silica in the aggregate and (usually) alkalies in the cement	Moisture (or moisture and alkalies)	Internal expansion and cracking
Other			
Unsound cement	Excessive amounts of unhydrated CaO of MgO in the cement	Moisture	Internal expansion and cracking
Plastic shrinkage cracking	Lack of maintained moisture content during specified curing period	High evaporation rate for moisture	Cracking at very early ages

Source: B. Mather, "Concrete Need Not Deteriorate," *J. Am. Conc. Inst.* 1(9), 33 (September 1979).

can affect the durability of the materials used to fabricate the components, that is, concrete, reinforcing steel, prestressing steel, and anchorage embedments.*

---

*Although anchorage embedments are not a constituent of concrete components per se, they must function with the concrete within which they are embedded.

Table 4. Predominant environmental stressors to which safety-related components in a LWR may be subjected

Structural subsystem	Material components	Important material parameters[a]	Predominant environmental stressors[b]
Prestressed concrete containment	Concrete	f_c', E_c, ν, CR	D, L
	Rebars	f_y, E_s, σ_u, e	C, L
	Prestressing	f_y, E_s, σ_u, R	C, L
Reinforced concrete containment	Concrete	f_c', E_c, ν	D, L
	Rebars	f_y, E_s, σ_u, e	C, L
Containment base mat	Concrete	f_c', E_c, ν	D, L, S
	Rebars	f_y, E_s, σ_u, e	C, L
Biological shield wall or building	Concrete	f_c', E_c, ν	T, I, L
	Rebars	f_y, E_s, σ_u, e	T, I, L
	Prestressing	f_y, E_s, σ_u	T, I, L
Auxiliary buildings	Concrete	f_c', E_c, ν	D, L
	Rebars	f_y, E_s, σ_u, e	C, L

[a] f_c' = concrete compressive strength
E = modulus of elasticity
ν = Poisson's ratio
CR = concrete creep
σ_u = ultimate strength
e = elongation or ductility
R = prestressing relaxation
f_y = steel yield strength

[b] T = temperature
D = durability
I = irradiation
C = corrosion
L = external, internal, or dead loads
S = subgrade settlement

4.1 Concrete Degradation

Concrete is a general term for a class of ceramic materials that vary widely in their properties and applications. The American Concrete Institute (ACI) defines concrete as "a composite material that consists essentially of a binding medium within which are embedded particles or fragments of aggregate; in portland cement concrete the binder is a mixture of portland cement and water."[2] By varying the constituents and their relative proportions in the mixture, concretes of widely differing properties can be obtained, for example, strengths from 0.7 to 100 MPa and unit weights from 800 to 4000 kg/m^3. Concrete materials utilized in LWR applications generally have compressive strengths ranging from 20.7 to 41.4 MPa and unit weights from 2240 to 2400 kg/m^3. Potential causes of deterioration of concrete would be cracking, aggressive environments, embedment corrosion, or extreme environmental exposure.

4.1.1 Concrete cracking

Cracking occurs in virtually all concrete structures and, because of concrete's inherently low tensile strength and lack of ductility, can never be totally eliminated. Cracks are significant from the standpoint that they can indicate major structural problems (active cracks); provide an important avenue for the ingress of hostile environments (active or dormant cracks); and inhibit a component from meeting its performance requirements, such as providing biological shielding (active or dormant crack). As noted in Table 5 (Ref. 3), cracking results from each material component and can occur while the concrete is in either a plastic or hardened state.

Table 5. Causes of concrete cracking

Component	Type	Cause of distress	Environmental factor(s)	Variables to control
Cement	Unsoundness	Volume expansion	Moisture	Free lime and magnesia
	Temperature cracking	Thermal stress	Temperature	Heat of hydration, rate of cooling
Aggregate	Alkali—silica reaction	Volume expansion	Supply of moisture	Alkali in cement, composition of aggregate
	Frost attack	Hydraulic pressure	Freezing and thawing	Absorption of aggregate, air content of concrete, maximum size of aggregate
Cement paste	Plastic shrinkage	Moisture loss	Wind and temperature	Temperature of concrete, protection of surfaces
	Drying shrinkage	Moisture loss	Relative humidity	Mix design, rate of drying
	Sulfate attack	Volume expansion	Sulfate ions	Mix design, cement type, admixtures
	Thermal expansion	Volume expansion	Temperature change	Temperature rise, rate of change
Reinforcement	Electrochemical corrosion	Volume expansion	Oxygen moisture	Adequate concrete cover

Source: S. Mindess and J. F. Young, *Concrete*, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1981, p. 572.

4.1.1.1 Cracking of concrete during initial setting (plastic concrete cracking). Cracking of concrete during initial setting can result from a number of causes: (1) settlement due to unstable subgrade; (2) poor form construction; (3) lack of, insufficient, or improper rebar placement; (4) rebar corrosion; (5) high-slump concrete; (6) improper consolidation; (7) lack of curing; (8) volume change due to solids settlement; (9) insufficient expansion or control joints; and (10) early stripping of forms.[4] Most cracking of plastic concrete, however, occurs in three primary forms: plastic shrinkage cracking, settlement cracking, and crazing.

Plastic shrinkage cracking occurs most frequently on the exposed surfaces of freshly placed floors and slabs subjected to a rapid loss of surface moisture caused by low humidity, wind, or high temperature. The cracks form as a result of differential volume change in which concrete near the surface tries to shrink but is restrained by the concrete below. Shrinkage cracks can range from a few millimeters to several meters in length, with spacings from a few millimeters to >3 m; and although generally shallow, they can penetrate the full depth of an elevated slab. Cracks of this type are not a direct threat to the structural integrity of a member but indirectly can have an effect if they are of sufficient width to permit entry of a hostile environment.

Settlement cracks result from constraint provided by obstacles such as reinforcing bars or other embedments, form work, or a prior concrete placement. These cracks do not occur where the foundation was properly prepared, forms were properly designed, the mix was properly proportioned to have the lowest possible slump consistent with placement, and adequate concrete compaction was provided.[5] Settlement cracks have the same effect on a structure as plastic shrinkage cracks.

Crazing sometimes occurs in a hexagonal pattern on the concrete surface at an early age because of improper curing (excessive water loss) and finishing procedures (excessive flotation or troweling). Crazing is surface related and thus not a threat to the structure.

4.1.1.2 Cracking of hardened concrete.* Cracking of hardened concrete results from shrinkage with restraint, thermal effects, and chemical reactions.†

Drying of hardened concrete. Concrete contracts (shrinks) as it loses water; if the concrete is constrained cracking can occur when the tensile strength of the concrete is exceeded. Factors that affect the volume change of mortars or concretes caused by variations in moisture conditions include: (1) cement and water contents, (2) composition and fineness of cement, (3) type and gradation of aggregate, (4) admixtures, (5) age, (6) test duration, (7) moisture and temperature conditions, (8) specimen size and shape, (9) form absorption, and (10) amount and distribution of reinforcement.[6] Cracking due to volume change not only may impair the ability of a structure to carry its designed loads but also may affect its durability and damage its appearance.

Carbon dioxide, present in the atmosphere, under some conditions may react with the $Ca(OH)_2$ or other lime-bearing compounds in hardened concrete to produce a reduction in volume and an increase in weight. Cracking resulting from carbonation is generally confined to a thin layer near

*Although not considered in this section, primary concrete containments can develop cracks during structural acceptance testing when the containment's internal pressure is increased to 1.15 times the design pressure.

†Effects of embedment corrosion including concrete cracking are covered in Sect. 4.1.3.

the surface. Another effect of carbonation is that in the areas where calcium carbonate forms, the pH of the concrete is lowered sufficiently (8.5 to 9.0) to destroy the passivating effect of the concrete on rebars, which potentially enables the rebars to corrode. Fortunately the carbonation process is slow, and its rate is inversely proportional to concrete quality, so it should have minimal effect on mass concrete structures.

Thermal effects resulting from cement hydration. The setting and hardening process of concrete is a chemical reaction that liberates heat on the order of 60 to 120 cal/g of cement.[6] If the heat cannot be dissipated to its surroundings fast enough, a temperature rise of 40°C or greater can occur,[3] and the mass will also expand. During cooling, the outer concrete surface cools first and shrinks, which can result in the formation of cracks. Because of the constraint provided by the inner concrete mass, which is still at a higher temperature, these cracks can be significant. Problems of this type are primarily associated with massive concrete structures such as dams rather than with LWR concrete components. In addition, precautions to reduce hydration effects include using low or moderate heat of hydration cements and "cool" materials as well as partially replacing cement with fly ash.

Chemical reactions. Concrete cracking can also result from a number of deleterious chemical reactions related generally to the aggregate materials: alkali-aggregate reactions, cement-aggregate reactions, and carbonate aggregate reactions.

Expansive reactions between aggregates containing active silica and alkalies derived from either cement hydration, admixtures, or external sources have caused many concrete structural failures in the past (late 1920s to early 1940s). The problem, which is generally confined to certain areas of the country, however, has been significantly reduced in recent years through proper aggregate material selection, use of low alkali cements, and addition of pozzolanic materials. The alkali-aggregate reaction therefore should not be a problem for LWR concrete components, because these structures generally were all fabricated after 1960, and petrographic examination techniques were available to identify potentially reactive aggregates.

Highly siliceous aggregate materials in Kansas, Nebraska, and Wyoming areas have produced concrete deterioration (map cracking) due to reaction with alkalies in cement. This type of distress should not be a problem for LWR concrete components, because the problem is regional, and it can be controlled by replacing 30% of the materials with crushed limestone aggregates.

Certain dolomitic limestone aggregates containing some clay and found in only a few geographical locations in the United States and Canada react with alkalies to produce expansive reactions. This problem can be identified and controlled by diluting the reactive aggregate with a less susceptible material and using low-alkali-content cement.

4.1.2 Aggregressive environments

Aggressive environments that could potentially lead to deterioration of concrete include weathering (i.e., freeze-thaw and wetting-drying), leaching and efflorescence, and aggressive chemicals.

4.1.2.1 Weathering. Porous materials containing moisture are susceptible to damage under repeated cycles of freezing and thawing. Several different processes can contribute to the paste behavior during freezing, including generation of hydraulic pressure due to ice formation, desorption of water from calcium silicate hydrate (C-S-H), and segregation of ice. Although a 9% volume increase occurs as water turns to ice, which in turn will produce dilation in the microcracks, this is insufficient to produce all the dilation that occurs in concrete. The primary cause of dilation is internal hydraulic pressure generated by capillary water as it is being compressed during ice formation.[7] When the hydraulic pressure exceeds the tensile strength of the cement paste, cracking occurs. If the concrete is partially dry or air-entrained, damage will not occur, because sufficient capillary space is available to prevent pressure buildup. Other factors leading to production of frost-resistant structures include: (1) selection of aggregates with adequate durability* (certain rocks having fine pores and relatively high absorption combined with low permeability, such as cherts and shales, are susceptible to failure under freezing-thawing conditions); (2) use of low water-cement ratio concretes properly handled, placed, and cured; and (3) design of structures to minimize exposure to moisture and facilitate drainage.

Alternate wetting and drying causes cycles of swelling and shrinkage. During periods of increasing humidity, absorption of water on the C-S-H surfaces creates a disjoining pressure. This pressure increases with increasing thickness of absorped water (increased humidity) until it reaches the point that it can exceed the van der Waal's attractions between adjacent particles, forcing them apart to create a dilation. Under decreasing disjoining pressure (lower humidity), the particles are drawn together by the van der Waal's forces — resulting in contraction. Disjoining pressure is significant only where the relative humidity is >50%.

4.1.2.2 Leaching and efflorescence. In structures containing areas of poorly consolidated materials, cracks, or improperly treated construction joints, water may enter and pass through. As the water passes, some of the readily soluble calcium hydroxide and other solids are leached out. With time this leaching can increase the concrete's porosity, which in turn lowers its strength and increases its vulnerability to aggressive chemicals. The rate of leaching is dependent on the amount of dissolved salts contained in the percolating water and on the water temperature (calcium hydroxide is more soluble in cold water). Water flowing over concrete's surface does not provide significant leaching.

Efflorescence is more of a surface phenomenon and consists of deposited salts that have been leached from the concrete and are crystallized on subsequent evaporation of the water or on intersection with carbon dioxide in the atmosphere. Efflorescence is an aesthetic problem

*Certain aggregates [shales, clayey rocks, friable sandstones, various cherts, and some micaeous material(s) that are readily cleavable and structurally weak or are very absorptive and swell when moistened] are subject to disintegration upon exposure to ordinary weathering conditions. These materials can be identified through ANSI/ASTM C88 "Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate."[8]

rather than a structural problem, but it is important in that it indicates that leaching is taking place in the structure.

4.1.2.3 Aggressive chemicals. Concrete that is properly proportioned, placed, and cured is relatively impervious to most waters, soils, and atmospheres. Some chemical environments (in solution form above a minimum concentration), however, can cause deterioration of even good-quality concrete. Because of the alkalinity of hydrated cement paste, alkaline materials usually do not attack it. Acidic materials, on the other hand, readily attack basic materials such as concrete through accelerated leaching of calcium hydroxide by the hydrogen ion. Reference 9 lists various chemical agents and their effect on concrete as well as commonly used protective treatments.

Sulfates of sodium, potassium, and magnesium present in alkali soils and waters have caused deterioration of concrete structures. The sulfates react chemically with the hydrated lime and hydrated calcium aluminate in cement paste to form calcium sulfate and calcium sulfoaluminate, with considerable associated expansion and disruption of the concrete. Sulfate resistance can be improved by the use of special sulfate-resisting cements or admixtures such as pozzolans and blast-furnace slag.

Sugar in solution is also very aggressive to concrete, because it dissolves more than just calcium hydroxide; it attacks both C-S-H and calcium aluminate hydrates.

4.1.3 Embedment corrosion

Spalling and cracking of concrete can result from the corrosion of embedded metals. The primary embedded material is reinforcing steel,* and the basic mechanism is tensile forces created in the concrete through formation of rust, which is an expansive reaction. Aluminum materials embedded in concrete may cause the same destructive effects of corrosion caused by (1) galvanic action between the aluminum and reinforcing steel, (2) stray electric currents, and (3) alkalies in the concrete.[6] Galvanic corrosion of aluminum is accelerated if chlorides and moisture are present. Lead and zinc behave somewhat like aluminum but to a lesser degree. Copper and copper alloys have good resistance to corrosion unless chlorides are present.

4.1.4 Extreme environmental exposure

Extreme environmental conditions that could cause deterioration of safety-related concrete components include prolonged exposure to elevated temperatures and/or irradiation.

4.1.4.1 Elevated temperature effects. Elevated temperature and thermal gradients are important to concrete structures in that they affect the concrete's strength (ability to carry loads) and stiffness (structural deformations and loads that develop at constraints). These property variations result largely because of changes in the moisture content of the concrete constituents and progressive deterioration of

---

*Corrosion of reinforcing steel is discussed in detail in Sect. 4.2.

the paste and aggregate (especially significant where thermal expansion values for cement paste and aggregate are markedly different). Other factors of interest when a structure operates under elevated temperature conditions are whether the component is under load (creep) or experiences load cycling, the long-term effects on strength and modulus of elasticity, the concrete-rebar bond strength, and the effectiveness of concrete radiation shielding.

General behavior. Concrete made with portland cement undergoes a number of transformations when subjected to elevated temperatures.[10,11] In addition to crystal transformations of the aggregate materials, a number of reactions occur to disintegrate the structure of the matrix. At low temperatures (<105°C) these reactions take the form of water expulsion. Dehydration of calcium hydroxide occurs when the temperature exceeds 400°C. Dissociation of calcium carbonate aggregates (if present) is complete by ~900°C.* Above 1200°C and up to 1300°C, some components of concrete begin to melt; and some of the aggregates, such as igneous rocks (basalt), show degassing and expansion. Above 1300 to 1400°C, concrete exists in the form of a melt, with melting initiating in the cement paste matrix. Refractory concretes utilizing special cements and aggregates are available for use in environments experiencing temperatures to 1800 to 2000°C, but they have not been used for fabrication of LWR components.

References 12–26 present results obtained from elevated temperature testing of concrete. Figures 22 and 23 summarize some of the published results on the residual compressive strength of concrete exposed to elevated temperatures for hot and cold testing, respectively.† Figure 24 summarizes the effect of elevated temperature on concrete's residual modulus of elasticity for both open-hot and closed-cold conditions. Examples of the effect of moderate elevated temperature exposure ($T < 180°C$) on the stress-strain behavior of sealed and unsealed limestone concrete specimens are presented in Figs. 25 and 26, respectively.[26]

Time-dependent response and thermal cycling. Time-dependent deformations (creep) at elevated temperature and thermal cycling can have an

*Aggregates containing quartz undergo a crystalline transformation from α-quartz (trigonal) to β-quartz (hexagonal) between 500 and 650°C. A substantial (~5.7%) increase in volume accompanies this transformation.[11]

†In cold testing, the specimens are gradually heated to a specified temperature, permitted to thermally stabilize at that temperature for a prescribed period of time, permitted to cool slowly to ambient, and then tested to determine mechanical properties. In hot testing, the specimens are gradually heated to a specified temperature, permitted to thermally stabilize at that temperature for a prescribed period, and then tested at temperature to determine mechanical properties. During testing, specimens are maintained either in an open environment where water vapor can escape or in a closed environment where the moisture is contained. The closed environment condition represents conditions for mass concrete where moisture does not have ready access to the atmosphere, and the open environment represents conditions where the element is either vented or has free atmospheric communication.

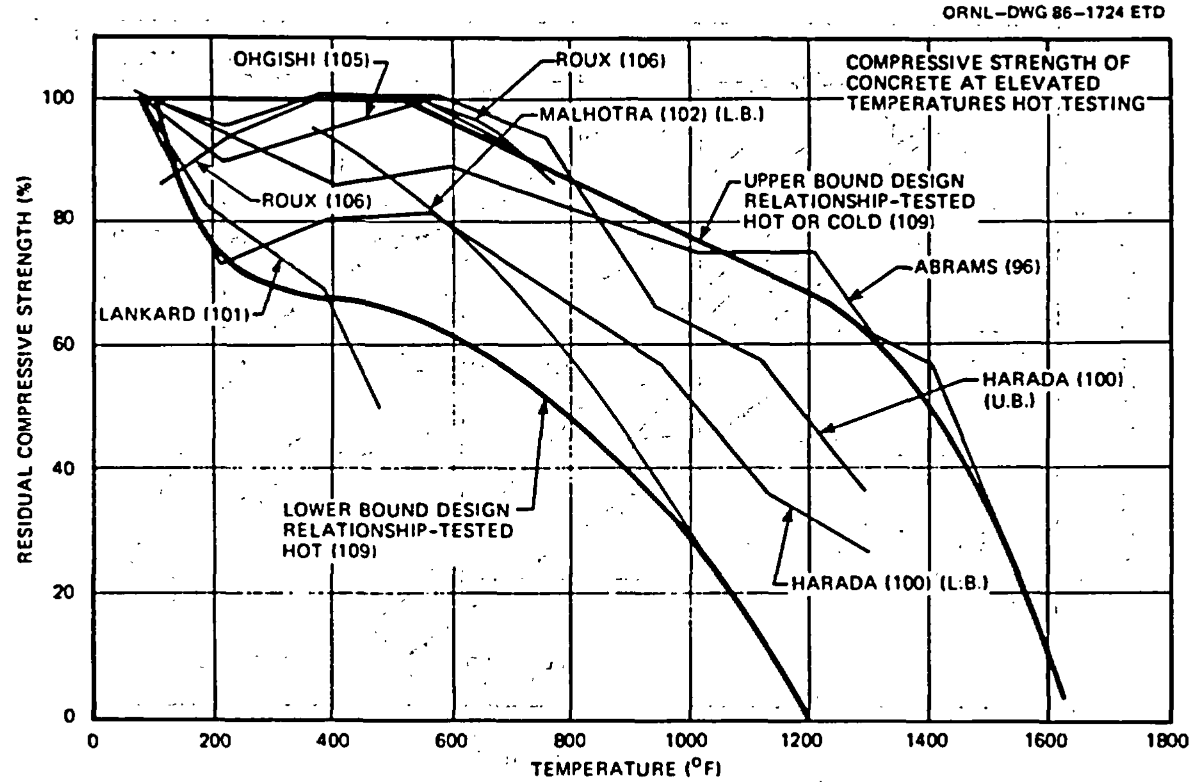


Fig. 22. Effect of temperature exposure on compressive strength of concrete hot testing. *Source*: G. N. Freskakis et al., "Strength Properties of Concrete at Elevated Temperatures," *Civ. Eng. Nucl. Power*, Vol. 1, ASCE National Convention, Boston, Mass., April 1979. (References noted in parentheses correspond with those cited in Ref. 25 in Chap. 4.)

effect on the performance of concrete components with respect to increased deformations (alignment) and potential strength loss, respectively.

Creep,* defined as "an increase in strain in a structural member with time due to a sustained stress," is important because it affects strains, deflections and stress distributions.† Figure 27 (Ref. 30) indicates the development of strain (creep) in a member with age (time since loading) and the effect that type of aggregate can have on creep magnitude for concretes maintained at room temperature. Like other

*Creep of concrete in tension also occurs and is of the same magnitude as creep in compression.[27] The following discussion of creep will pertain to creep under compressive loadings.

†Because actual structures are generally under a multiaxial stress state, creep of concrete under multiaxial loading is important. Information on creep of concrete under multiaxial loadings, while at elevated temperature, can be obtained from Ref. 28, which presents data obtained using a specially designed large triaxial-torsion testing machine with hygrothermal control (described in Ref. 29).

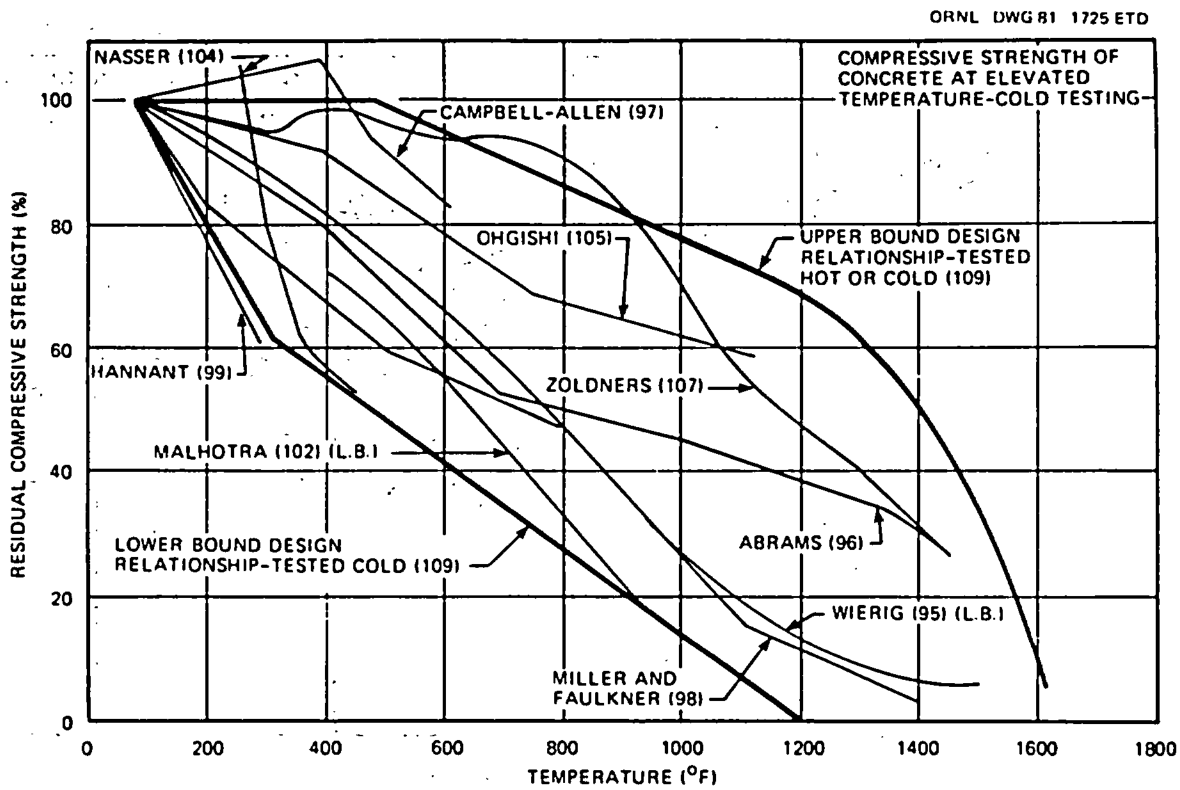


Fig. 23. Effect of temperature exposure on compressive strength of concrete cold testing. *Source*: G. N. Freskakis et al., "Strength Properties of Concrete at Elevated Temperatures," *Civ. Eng. Nucl. Power*, Vol. 1, ASCE National Convention, Boston, Mass., April 1979. (References noted in parentheses correspond with those cited in Ref. 25 in Chap. 4.)

solids, creep of concrete increases with temperature. Below 100°C, concrete creep at moderate stress levels originates in the cement paste, probably because of the mutual approach of adjacent laminar particles of cement gel, which is facilitated by the presence of water in gaps between the particles.[11] Another effect of temperature is the acceleration of hydration (aging) at moderately elevated temperatures. At temperatures above 105°C, dehydration occurs in a loaded concrete specimen, which probably accelerates creep as shown in Fig. 28 (Ref. 19).

Thermal cycling, even at relatively low temperatures (65°C), can have some deleterious effects on concrete's mechanical properties. Results presented in Refs. 31 and 32 indicate that the compressive, tensile, and bond strengths and the modulus of elasticity are reduced and that Poisson's ratio is increased. At higher temperatures (200 to 300°C), the first thermal cycle causes the largest percentage of damage, with the extent of damage markedly dependent on aggregate type and associated with loss of bond between the aggregate and matrix. The effect of temperature cycling on a limestone aggregate concrete is presented in Fig. 29 (Ref. 31).

ORNL-DWG 81-1726 ETD

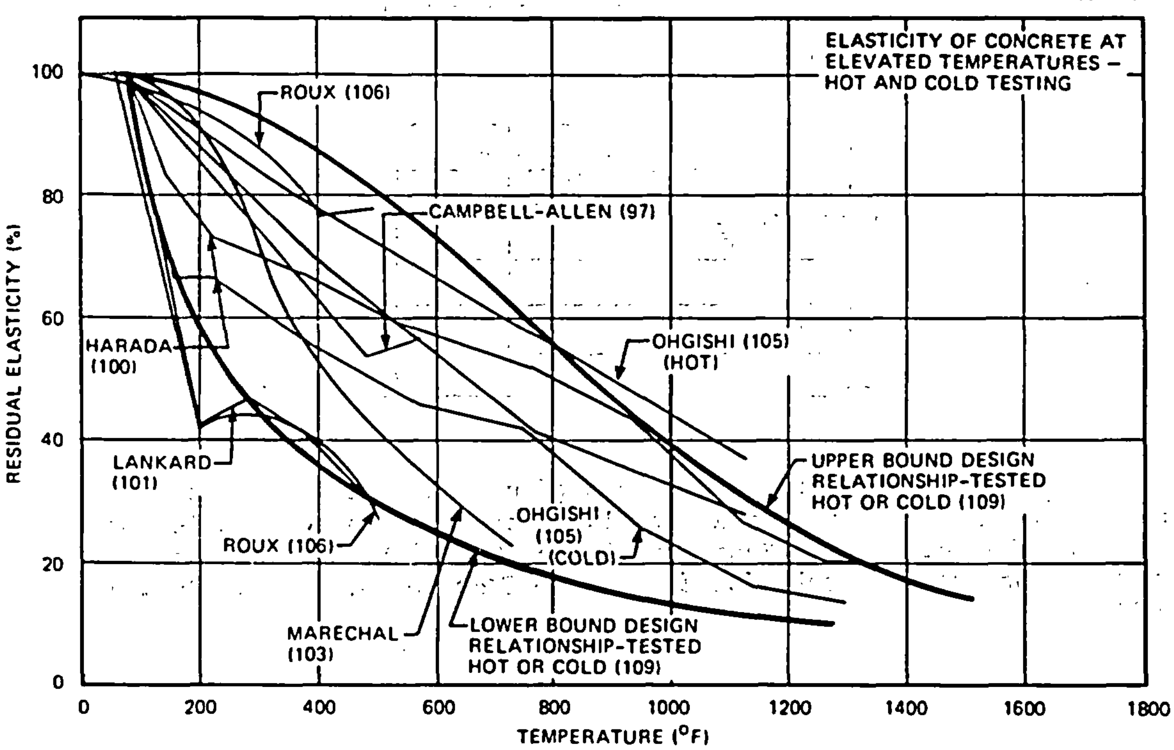


Fig. 24. Effect of temperature exposure on modulus of elasticity of concrete hot and cold testing. *Source*: G. N. Freskakis et al., "Strength Properties of Concrete at Elevated Temperatures," *Civ. Eng. Nucl. Power*, Vol. 1, ASCE National Convention, Boston, Mass., April 1979. (References noted in parentheses correspond with those cited in Ref. 25 in Chap. 4.)

ORNL-DWG 86-4570 ETD

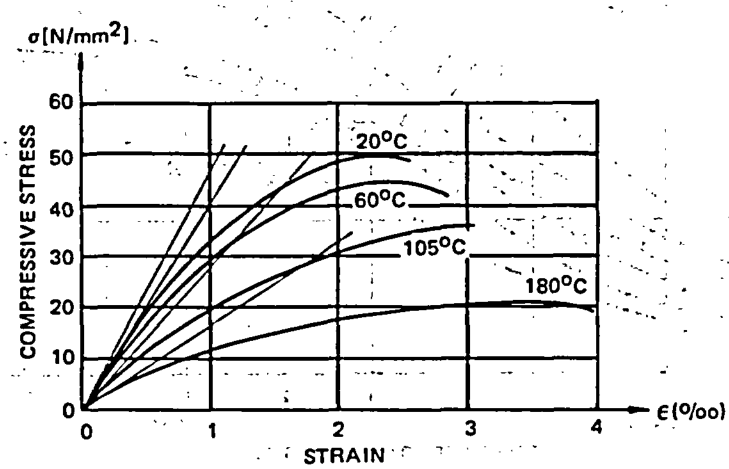


Fig. 25. Stress-strain diagrams of sealed limestone concrete specimens. *Source*: R. Kottas et al., "Strength Characteristics of Concrete in the Temperature Range of 20° to 200°C," Paper H1/2, *5th Int'l. Conf. on Structural Mechanics in Reactor Technology*, Berlin, Aug. 13–17, 1979.

ORNL-DWG 86-4571 ETD

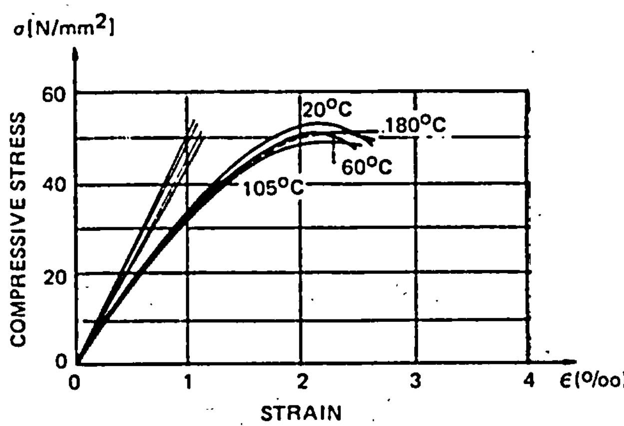


Fig. 26. Stress-strain diagrams of unsealed limestone concrete specimens. *Source*: R. Kottas et al., "Strength Characteristics of Concrete in the Temperature Range of 20° to 200°C," Paper H1/2, *5th Int'l. Conf. on Structural Mechanics in Reactor Technology*, Berlin, Aug. 13–17, 1979.

ORNL-DWG 81-1720 ETD

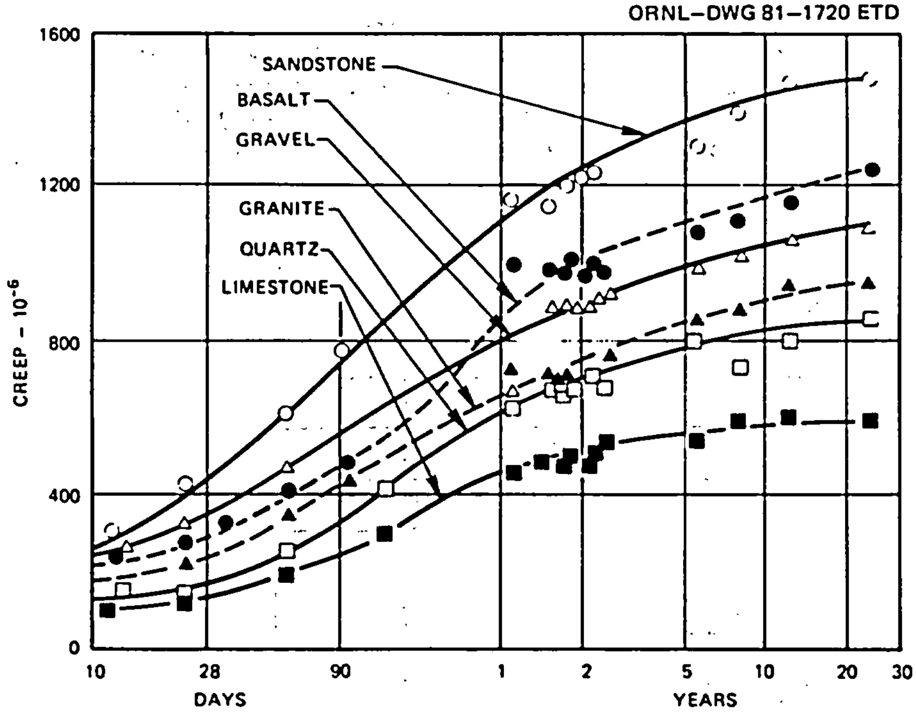


Fig. 27. Creep of concretes with different aggregates. *Source*: G. E. Troxell et al., *Long-Time Creep and Shrinkage Tests of Plain and Reinforced Concrete*, ASTM Proc. 48, pp. 1101–20, 1958.

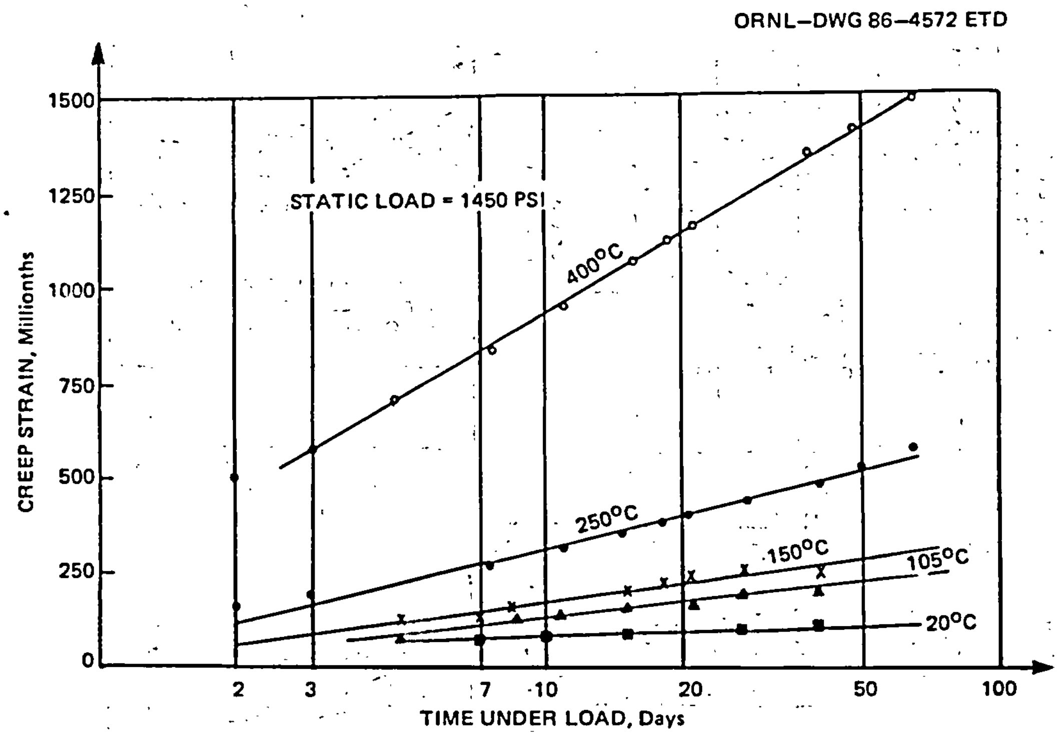


Fig. 28. Creep of portland cement/porphyry concrete at various temperatures. *Source*: J. C. Marechal, "Variations in the Modulus of Elasticity and Poisson's Ratio with Temperature," SP-34, vols. 1–3, Paper SP 34-27, *Concrete for Nuclear Pressure Vessels*, American Concrete Institute, Detroit, 1972.

Long-term exposure (aging). The design lifetime of nuclear plants, and thus of concrete components, is nominally 40 years. Over a plant's operating lifetime certain concrete components (i.e., the biological shield-pedestal) may be subjected to moderately elevated temperatures, which could affect the concrete's mechanical properties. Unfortunately despite the potential significance of this effect, only a limited number of data have been identified relating the effects of long-term elevated temperature exposure (aging) to concrete properties.[33–36]

Carette et al.[33] conducted an investigation to determine the changes in mechanical properties of a limestone aggregate concrete after exposures to temperatures up to 600°C for periods up to 8 months. For thermal exposure to 75°C, compressive and splitting-tensile strengths after 8 months' exposure were 98 and 94%, respectively, of their reference values. However, on exposure to 600°C for just 1 month, compressive and splitting-tensile strengths were only 23 and 38%, respectively, of their reference values. In companion mixes, where either fly ash or blast furnace slag was used, no improvement in mechanical properties occurred after exposure to sustained high temperatures as a result of partial replacement of the cement.

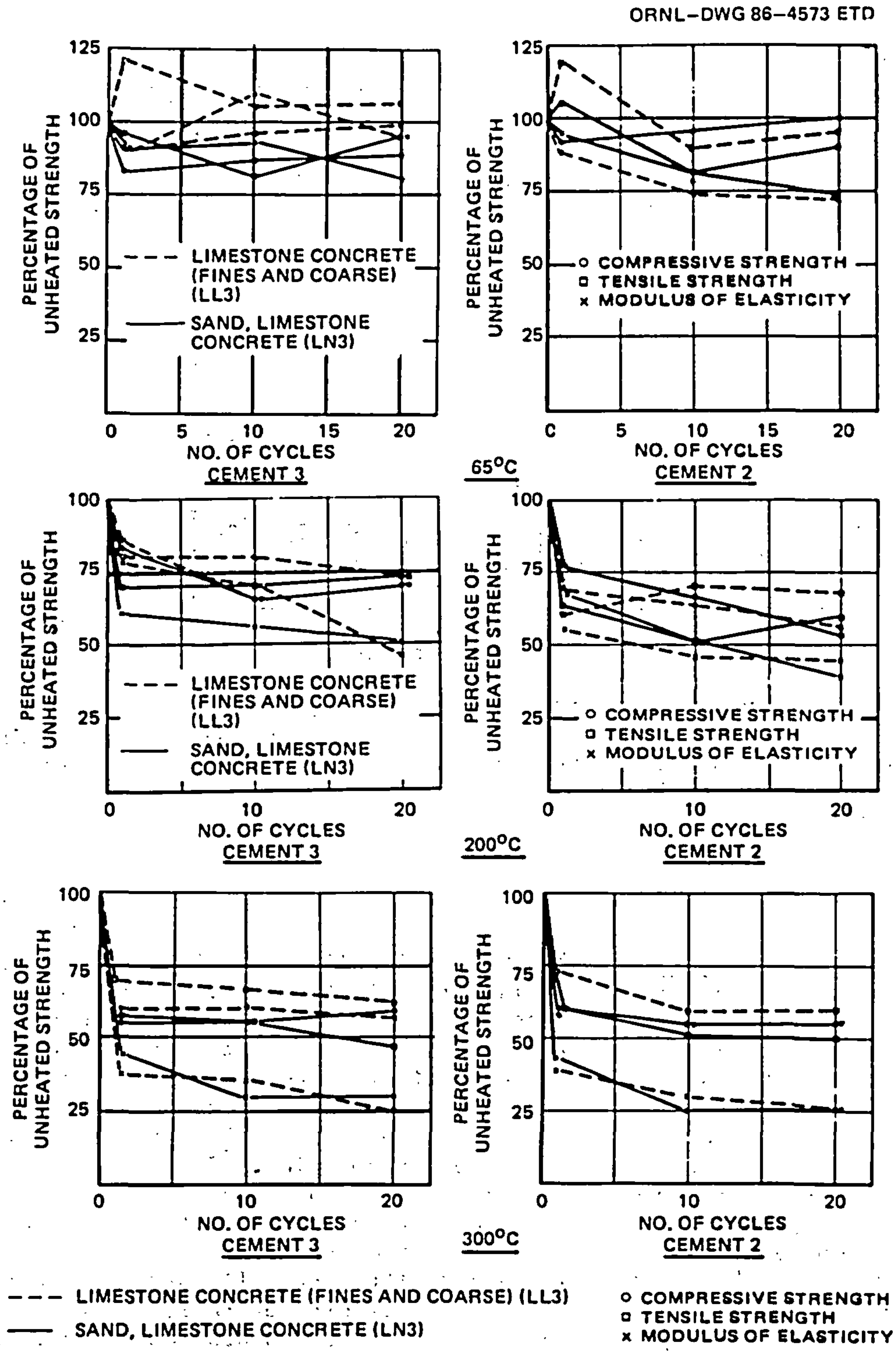


Fig. 29. Effects of temperature cycles on limestone concretes. *Source*: D. Campbell-Allen and P. M. Desai, "The Influence of Aggregate on the Behavior of Concrete at Elevated Temperature," *Nucl. Eng. Des.* 6(1) (1967).

Mears[34] investigated the effect of long-term exposure (up to 13 years) at moderately elevated temperature (65°C) on the mechanical properties of a limestone aggregate concrete. These tests were somewhat unusual in that the specimens were first subjected to a simulated temperature-vs-time cement hydration cycle. Also, because the concrete mix was being evaluated for an application that experienced exposure to sulfate-bearing groundwater at elevated temperatures (~65°C), both ordinary and sulfate-resistant portland cements were investigated. Specimens, after being subjected to the simulated cement hydration cycle, were stored either in water at 19°C (control specimens) or in a sodium sulfate solution of 2000 ppm at 65°C. Frequently during the test program, the sodium sulfate solution was changed, which required cooling to room temperature; the specimens were therefore also subjected to thermal cycling. Results of the study indicated that there was no evidence of long-term degradation in compressive strength for any of the concrete mixes and heat treatments utilized and that for a given compressive strength, the dynamic modulus of elasticity was lower for the concrete that had been heated.

A five-year testing program was conducted to determine the effects of long-term exposure to elevated temperature on the mechanical properties of concrete used in constructing the radioactive underground storage tanks at Hanford Engineering Development Laboratory (HEDL).[35] Tests were conducted using specimens fabricated from the same mix proportions and materials specified for the concrete used to fabricate the tanks (20.7- and 31.0-MPa design compressive strengths). Concrete strength, modulus of elasticity, and Poisson's ratio values were determined from specimens subjected to either 121, 177, or 232°C for periods of up to 33 months. The effect of thermal cycling was also investigated. Results showed that the compressive strengths in general tended to decrease with increasing temperature and also with length of exposure; however, with the exception of the cylinders exposed to 232°C, all compressive strength results obtained after a 900-d exposure exceeded design values. Splitting-tensile strength results also decreased somewhat with increasing temperature and length of exposure. Modulus of elasticity was affected most significantly by the elevated temperature exposure; after 920 d of heating at 232°C, it had a value of only 30% — the value obtained from an unheated, control specimen. Poisson's ratio, although exhibiting somewhat erratic values, was relatively unaffected by either the magnitude or the length of elevated temperature exposure. Thermal cycling (<18 cycles) to 177°C produced moderate reductions in compressive strength (5 to 20%), significant reductions in modulus (30 to 50%), and slight reductions in Poisson's ratio (0 to 20%). Time-dependent (creep) and thermal property data were also obtained from the concrete mixes.

Associated with the laboratory investigation described in the previous paragraph was a study to confirm the laboratory results by testing samples removed from the underground storage tanks and process buildings at HEDL.[36] Cores 76 mm in diameter were obtained over the length of the haunch, wall, and footing of a single-shell tank that was built in 1953; contained waste for about 8 years; reached temperatures in the range of 127 to 138°C; and experienced a radiation field of 0.10 to 0.13 C/kg/h (400 to 500 R/h). Although considerable scatter was obtained from the data because of different concrete pours and different environmental exposures, after about 29 years of exposure, only one data point fell below the 20.7-MPa design compressive strength. Figure 30 presents compressive strength results obtained from these tests as well as those obtained from tests on concretes from other structures and compares them to predicted values obtained from laboratory work.

Concrete-reinforcing steel bond strength. Only limited data are available on the effect of elevated temperatures on the bond strength between concrete and steel reinforcement. Kagami[37] — in testing specimens fabricated from river gravel concretes containing embedded plain, round, steel bars — found that the residual bond stress after subjecting the specimens to 300°C for 90 d and then cooling to room temperature was only ~50% the value before heating. Milovanov and Salmanov[38] demonstrated the importance of reinforcement type when they showed that ribbed bars experienced a loss of bond strength only above 400°C but that smooth bars lost strength after only a small temperature increase. Results presented in Refs. 39 and 40 indicate that for exposure temperatures <150°C the loss in bond strength between concrete and steel reinforcement is small (<15%).

Radiation shielding effectiveness. Portland cement concrete possesses many of the physical qualities of an ideal radiation shield. It

ORNL–DWG 86–4574 ETD

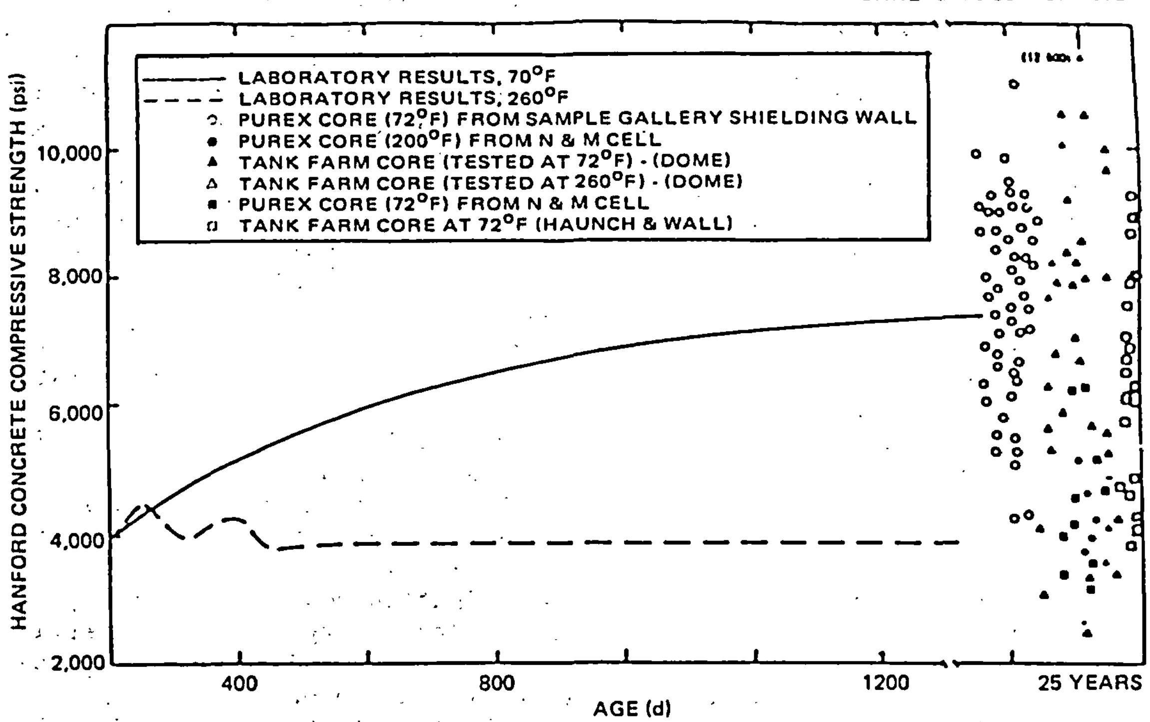


Fig. 30. Laboratory vs actual sample compressive strength data. *Source*: M. P. Gillen et al., *Strength and Elastic Properties of Concrete Exposed to Long-Moderate Temperatures and High Radiation Fields*, RHO-RE-SA-55 P, Rockwell Hanford Operations, Richland, Wash., 1984.

is a polyphase material consisting of particles of aggregate contained in a matrix of portland cement paste. Gamma rays are absorbed by the high-density aggregate materials, and neutrons are attenuated by hydrogen atoms in the cement paste. The effectiveness of concrete as a shield, however, may be reduced under service conditions (elevated temperature) as drying reduces the hydrogen content or cracking occurs.

Results of elevated temperature exposure on shielding of heavyweight aggregate (iron limonite and magnetite limonite) concretes are presented in Fig. 31 (Ref. 41). Significant changes in attenuation effectiveness were found as the concrete was heated to 100 and 175°C, with little additional change effected in heating to 320°C. Despite the loss of neutron and gamma attenuation efficiency with increasing temperature, it was concluded that the concrete would serve as a satisfactory shield material. If increasing efficiency were required at higher temperatures, it could be accounted for in the design.

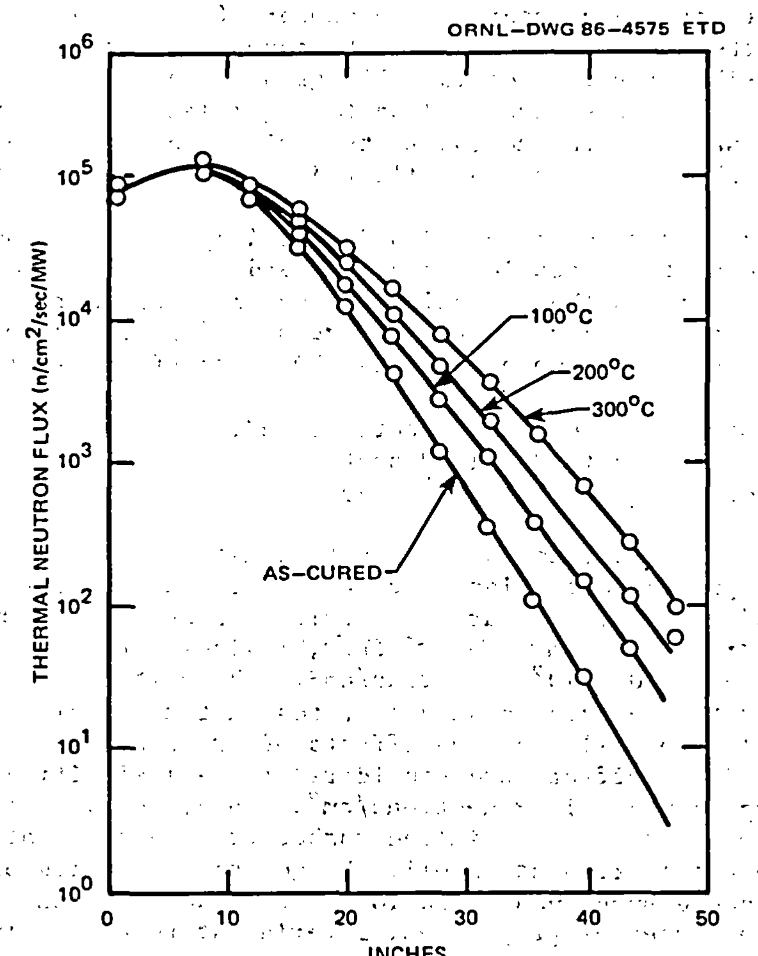


Fig. 31. Thermal neutron distribution in ordinary concrete as a function of temperature. *Source*: E. G. Peterson, *Shielding Properties of Ordinary Concrete as a Function of Temperature*, HW-65572, Hanford Atomic Products Operation, Richland, Wash., Aug. 2, 1960.

Shielding effectiveness of concrete is also reduced if through cracks develop. Reference 42 investigated the effect of gamma rays through a concrete shield containing straight and crooked cracks. In the immediate vicinity of the concrete surface, leakage of gamma rays through a slit contributed significantly to γ-dose rate but diminished rapidly with distance from the surface as a result of shield thickness and scattering effects. Reference 43 investigated the shielding effectiveness of cracked concrete and developed formulas to define the resulting effects. Guidelines developed for compensation for cracking concluded that it might be economically advantageous to allow a concrete shield to crack and then shield the resulting irradiation by other means.

4.1.4.2 Irradiation effects. Concrete has traditionally been used as a shielding material because it attenuates radiation with reasonable thickness requirements, has sufficient mechanical strength, can be constructed in virtually any size and shape at reasonable cost, and requires minimal maintenance. Irradiation, however, in the form of either fast and thermal neutrons emitted by the reactor core or gamma rays, produced as a result of capture of neutrons by members (particularly steel) in contact with the concrete can affect the concrete. The fast neutrons are mainly responsible for the considerable growth, caused by atomic displacements, that has been measured in the aggregate. Gamma rays produce radiolysis of water in the cement paste, which can affect concrete's creep and shrinkage behavior to a limited extent and also result in evolution of gas.

Operation of a reactor over its 30- to 40-year life expectancy may subject the concrete to considerable fast and thermal neutron fluxes. Reference 44 estimates the following values for maximum radiation to which the prestressed concrete reactor vessel of a high-temperature gas-cooled reactor could be subjected after 30 years of service:

thermal neutrons: 6×10^{19} neutrons/cm^2,
fast neutrons: 2 to 3×10^{18} neutrons/cm^2,
gamma radiation: 10^9 Gy (10^{11} rad).

For a 1250-MW(e) pressurized-water reactor, Ref. 45 estimates the integrated flux exposure to the inner surface of the biological shield as 10^{19} fast neutrons/cm^2 after 40 years of service. Section III, Division 2, of the ASME *Boiler and Pressure Vessel Code*,[46] gives a radiation exposure level allowable to 10×10^{20} neutrons/cm^2. The British Code for prestressed concrete pressure vessels[47] states that the maximum permissible neutron dose is controlled by the effects of irradiation on the concrete properties, and the effects are considered to be insignificant for exposure levels up to 0.5×10^{18} neutrons/cm^2. Note, however, that these criteria are based on a very limited number of data and that quantifying the extent to which irradiation will change the properties of concrete is impossible because such quantification is dependent on many factors, such as variation of material properties, material state of testing, neutron energy spectrum, and neutron dose rate.

Several reports have been written on the effects of irradiation on concrete properties.[44,48–75] The apparent availability of data on irradiation effects on concrete properties is, however, misleading because of technical and experimental difficulties in conducting meaningful tests. In addition, available data are generally not comparable because (1) different materials were used, (2) mix proportions varied, (3) specimen size was inconsistent, (4) temperatures varied, and (5) both cooling and drying conditions were used. Reference 44 presents an excellent summary of experimental data that are available on irradiation effects on concrete properties. Twelve conclusions can be drawn from these data. (1) For some concretes, neutron radiation of >1 × 10^{19} neutrons/cm^2 may cause some reduction in compressive strength (Fig. 32) and tensile strength (Fig. 33). (2) The decrease of tensile strength due to neutron radiation is more pronounced than the decrease of compressive strength. (3) Resistance of concrete to neutron radiation apparently depends on the type of neutrons (slow or fast) involved, but the effect is not clarified. (4) Resistance of concrete to neutron radiation depends on mix proportions, type of cement, and type of aggregate (Fig. 34). (5) The effect of gamma radiation on concrete's mechanical properties requires clarification. (6) The deterioration of concrete properties associated with a temperature rise resulting from irradiation is relatively minor. (7) Coefficients of thermal expansion and conductivity of irradiated concrete differ little from those that would result from temperature-exposed concrete. (8) The modulus of elasticity of concrete when exposed to neutron irradiation decreases with increasing neutron fluence (Fig. 35). (9) Creep of concrete is not affected by low-level radiation exposure, but for high levels of exposure creep is likely to increase with exposure because of the effects of irradiation on the concrete's tensile and compressive strengths. (10) For some concretes, neutron radiation with a fluence of >1 × 10^{19} neutrons/cm^2 can cause a marked increase in volume. (11) In general, concrete's irradiation resistance increases as the irradiation resistance of aggregate increases. (12) Irradiation has little effect on shielding properties of concrete beyond the effect of moisture loss due to a temperature increase.

Although detailed information is very limited, Ref. 66 indicates the effect of extended periods of irradiation on concrete properties. In the study, concrete was removed from the 2.13-m-thick graphite reactor shield (Fig. 36) at Oak Ridge National Laboratory after being in place for 12 years. To obtain a complete picture of the conditions to which the shield had been subjected, temperature gradients (19 to 40°C), gamma ray [8.1 × 10^{-8} (8.1 × 10^{-1}) to 1.9 × 10^{-1} J/g·h (1.9 × 10^{6} erg/g·h)] and fast-neutron [undetectable to 1.78 × 10^{-3} J/g·h (1.78 × 10^{4} erg/g·h)] dose rates, and thermal-neutron fluxes (1.88 × 10^{2} to 4.47 × 10^{10} neutrons·cm^{2}·s) were determined.* Analysis of a 117-mm-diam core sample through the shield showed that the chemical properties and density of the shield had not changed appreciably since a similar investigation done 8 years earlier; however, the compressive strength at the reflector-shield interface had dropped as much as 40% (16.9 to 10.1 MPa), while near the back of the shield (thickness = 2.0 m) the change was negligible (11.4 to 11.1 MPa). Damage to the concrete by irradiation was felt to be less than that caused by related temperature effects.

---

*Highest values were at the reflector-shield interface (thickness = 0 m) and lowest values at the back of shield (thickness = 2.13 m).

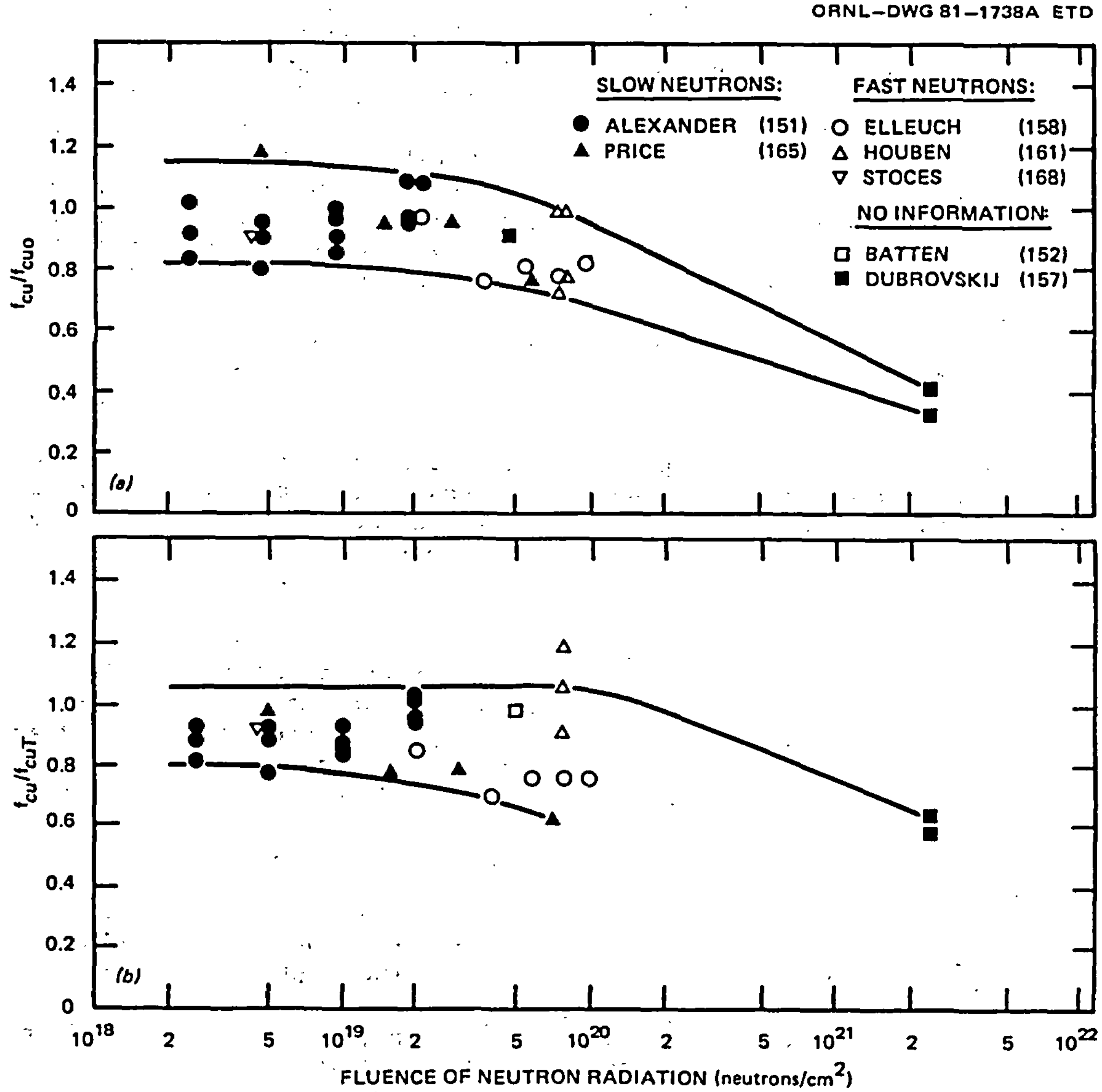


Fig. 32. Compressive strength of concrete exposed to neutron radiation relative to untreated concrete: thermal effects on strength (*a*) not included, (*b*) included. *Source*: H. K. Hilsdorf et al., "The Effects of Nuclear Radiation on the Mechanical Properties of Concrete," Douglas McHenry International Symposium on Concrete and Concrete Structures, Publication SP-55, American Concrete Institute, Detroit, 1978. (References noted in parentheses correspond to those cited in Ref. 25 in Chap. 4.)

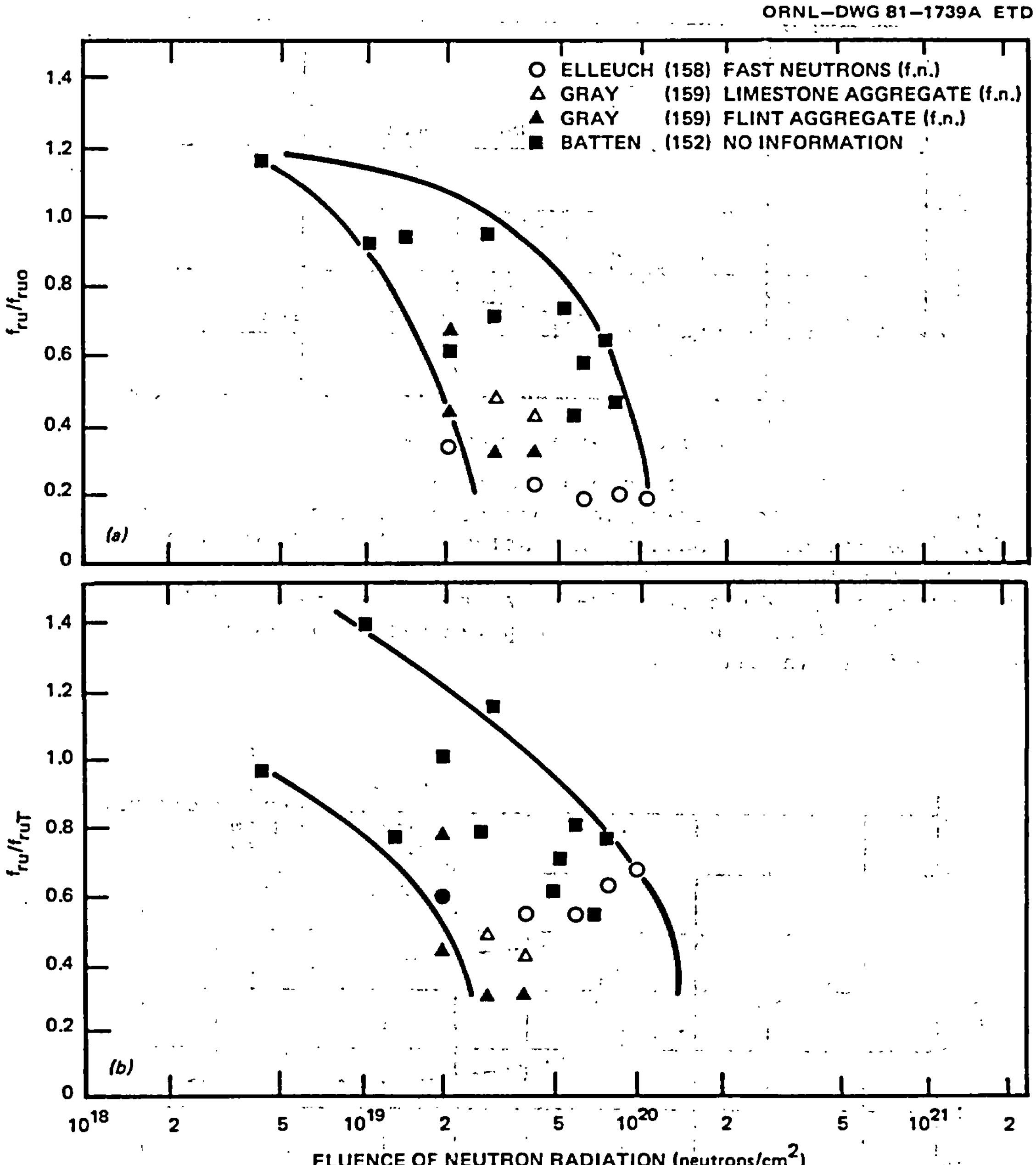


Fig. 33. Tensile strength of concrete exposed to neutron radiation relative to untreated concrete: thermal effects on strength (*a*) not included, (*b*) included. *Source*: H. K. Hilsdorf et al., "The Effects of Nuclear Radiation on the Mechanical Properties of Concrete," Douglas McHenry International Symposium on Concrete and Concrete Structures, Publication SP-55, American Concrete Institute, Detroit, 1978. (References noted in parentheses correspond to those cited in Ref. 25 in Chap. 4.)

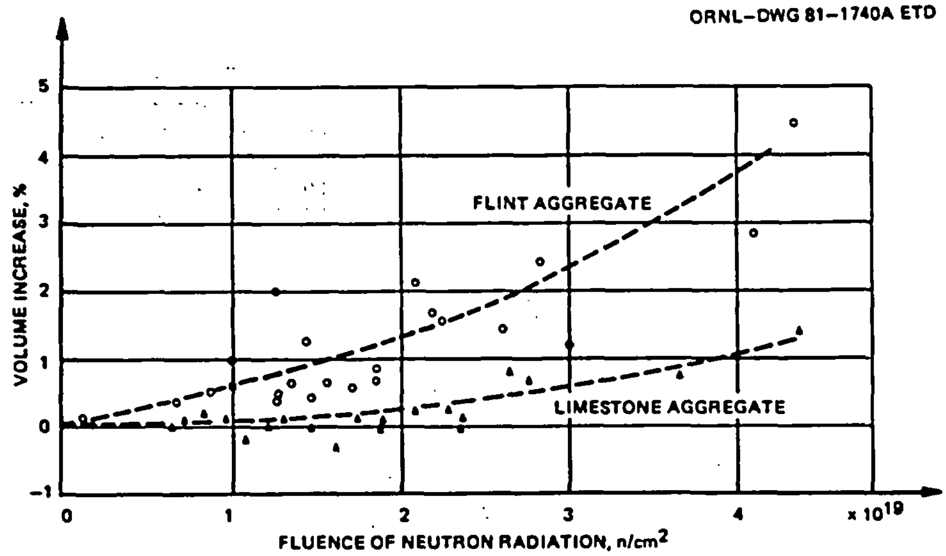


Fig. 34. Effects of fast neutron exposure on volume change of flint aggregate and limestone aggregate concretes. *Source*: H. K. Hilsdorf et al., "The Effects of Nuclear Radiation on the Mechanical Properties of Concrete," Douglas McHenry International Symposium on Concrete and Concrete Structures, Publication SP-55, American Concrete Institute, Detroit, 1978. (References noted in parentheses correspond to those cited in Ref. 25 in Chap. 4.)

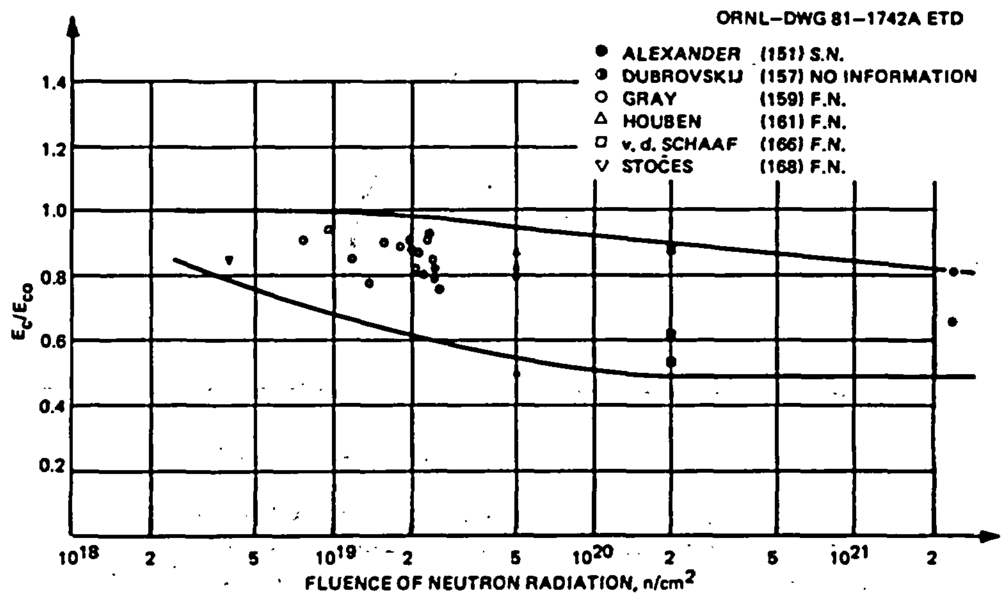


Fig. 35. Modulus of elasticity of concrete exposed to neutron radiation relative to untreated concrete: thermal effects on modulus not included. *Source*: H. K. Hilsdorf et al., "The Effects of Nuclear Radiation on the Mechanical Properties of Concrete," Douglas McHenry International Symposium on Concrete and Concrete Structures, Publication SP-55, American Concrete Institute, Detroit, 1978. (References noted in parentheses correspond to those cited in Ref. 25 in Chap. 4.)

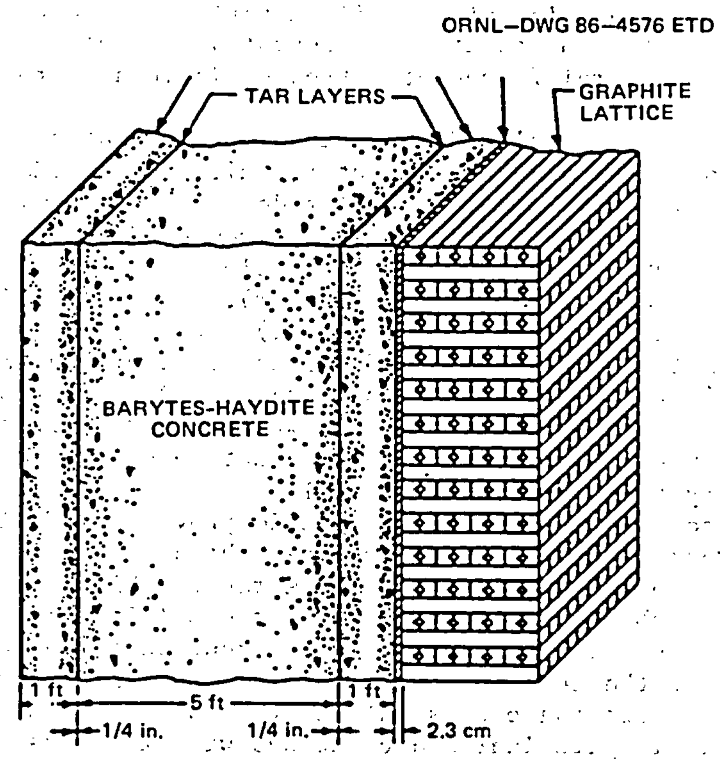


Fig. 36. Cross section of ORNL graphite reactor shield. *Source:* T. V. Blosser et al., *A Study of the Nuclear and Physical Properties of the ORNL Graphite Reactor Shield*, ORNL-2195, Aug. 25, 1958.

4.2 Concrete Reinforcing Steel Degradation

Mild steel reinforcing bars are provided to control the extent of cracking and the width of cracks at operating temperatures, resist tensile stresses and compressive stresses for elastic design, and provide structural reinforcement where required by limit condition design procedures.[46,76] Potential causes of degradation of the reinforcing steel are corrosion, elevated temperature exposure, and irradiation.

4.2.1 Corrosion

When portland cement hydrates, the silicates react with water to produce calcium silicate hydrates and calcium hydroxide. The high alkalinity of this chemical environment normally protects embedded steel because of the formation of a protective oxide film (gamma Fe_2O_3) on the steel. Passivity of this protective film, however, can be destroyed by penetration of aggressive ions or a reduction in the pH to <11, which can be caused by leaching of alkaline substances by water or by reaction with carbon dioxide or acidic materials.[77] Carbonation, which is discussed earlier, is primarily a surface effect of insignificance unless the concrete is of poor quality or the rebar has very shallow cover. Leaching by flowing water or reaction with acidic materials is felt to be only a remote possibility for LWR concrete components. Therefore, the most likely cause of steel reinforcement corrosion is related to chloride ions.

Possible sources of chloride ions include aggregates containing chlorides, saline water used as mix water, calcium chloride accelerators, cements containing small amounts of chlorides, and the environment.* For steel corrosion to occur, four essential elements of an electrochemical cell must be present: (1) anode (point of electron release and where ions go into solution), (2) conductor (rebar), (3) cathode (electrons consumed in presence of oxygen and moisture), and (4) electrolyte (moist concrete). When the metallic iron of the steel reinforcement is converted to rust [$Fe(OH)_3$], a volume increase of 600 to 700% occurs, which causes cracking and spalling of concrete where its tensile capacity is exceeded. The extent of corrosion is somewhat dependent on the orientation and geometry of the crack† and on time.

Most research reports and the ASME Code provisions deal with the width of the crack at the concrete surface, which is not in any way uniquely related to the crack width at the rebar. Width of a crack at the level of the rebar is related to the crack origin, amount of concrete cover, steel stress, bar diameter, reinforcement ratio, arrangement of bars, and depth of the tensile zone. When a crack is transverse to the rebar, localized corrosion occurs only over about three bar diameters. When the crack is longitudinal and coincides with the rebar, passivity is lost at many locations, and corrosion can proceed unchecked. Beeby[79] relates corrosion to time by noting that

$$t_0 + t_1 > \text{design life of structure} ,$$

where t_0 is the initiation phase (time from construction to passivity of rebar is destroyed) and t_1 is the active phase during which corrosion occurs. If t_0 and t_1 can be established, the life of the structure can be determined; however, in reality this is difficult because t_0 depends on several unknowns (environment, concrete permeability, whether cracking has occurred, and cover) as does t_1, which also must account for rate effects and defining an acceptable level of corrosion.

---

*Cracks in concrete accelerate the onset of corrosion that results from chloride ion penetration; however, the corrosion is confined to the point of intersection with the reinforcement. Some analysts feel that because chloride ions eventually can penetrate uncracked concrete to initiate more widespread corrosion, little difference exists between the amount of corrosion in cracked and uncracked concrete.[78]

†Reference 78 presents information on tolerable crack widths in reinforced concrete structures for different exposure conditions. Building codes protect reinforcement from corrosion by (1) specifying minimum cover, (2) establishing minimum concrete quality, and (3) limiting crack widths.

4.2.2 Elevated temperature effects

The properties of reinforcing steel used in design are generally a function of the yield stress, which is affected by exposure to elevated temperature. The yield strength of ANSI/ASTM A36 structural steel is relatively unaffected for temperature exposures <93°C (Ref. 80). Data for German reinforcing steels (Fig. 37) indicate that for temperatures up to ~200°C the yield strength is reduced <10% and that at 500°C it falls to ~50% of its reference value, with hot rolled steels performing better than cold twisted or cold drawn steels.[10] The modulus of elasticity exhibits a similar reduction pattern with increasing temperature.

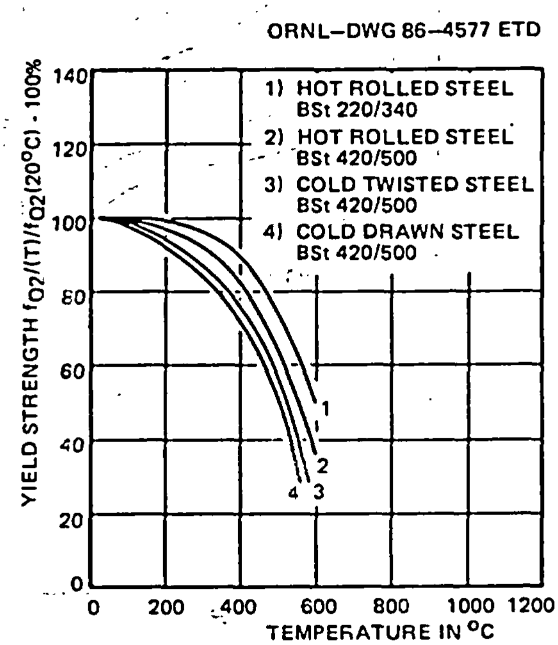


Fig. 37. Effect of temperature on yield strength (0.2%) of four types of German reinforcing steel. *Source*: U. Schneider et al., "Effect of Temperature on Steel and Concrete for PCRV's," *Nucl. Eng. Des*. 67, 245–58 (1981).

4.2.3 Irradiation effects

Neutron irradiation produces changes in the mechanical properties of structural steel, for example, an increase in the materials yield strength and a rise in the ductile/brittle transition temperature.[47] These changes are shown in Figs. 38 and 39, which present the effects of irradiation on the stress-strain curve for a mild steel and Charpy V-notch energy and temperature curves for unirradiated and irradiated mild steel, respectively.[81]

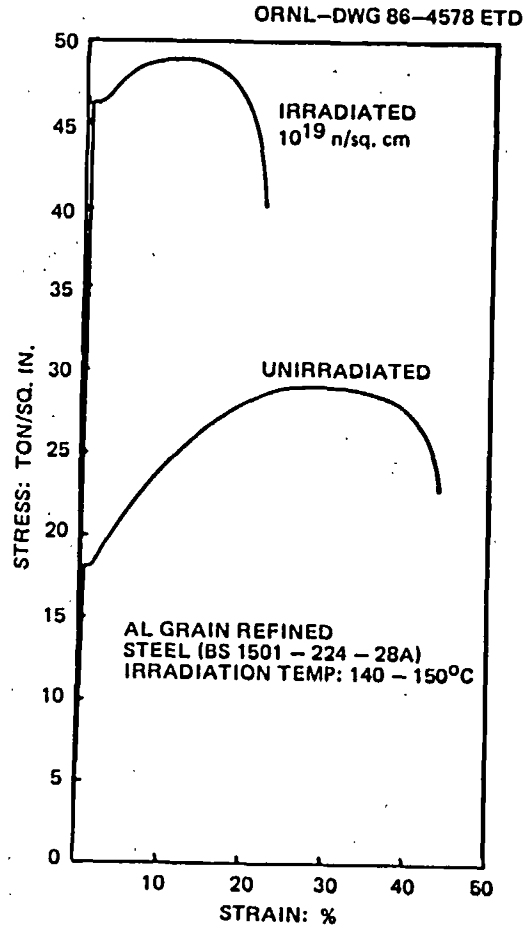


Fig. 38. Stress-strain curves for unirradiated and irradiated mild steel. *Source*: A. Cowan and R. W. Nichols, "Effect of Irradiation on Steels Used in Pressure Vessels," Group D Paper 20, Prestressed Concrete Pressure Vessels, The Institute of Civil Engineering, London, 1968.

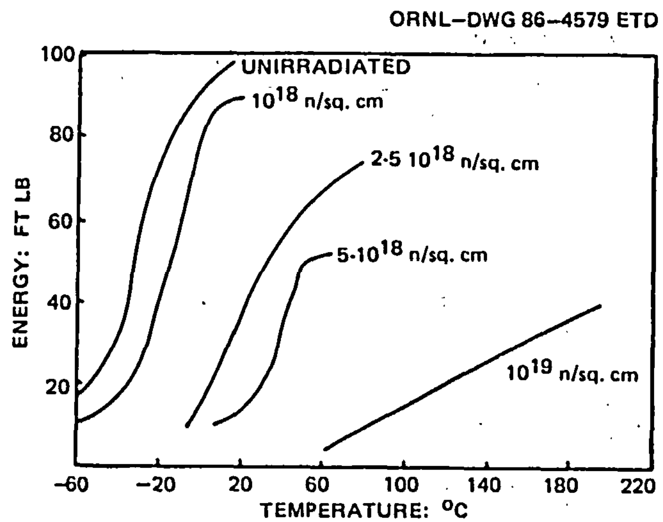


Fig. 39. Charpy V-notch energy/temperature curves for unirradiated and irradiated mild steel. *Source*: A. Cowan and R. W. Nichols, "Effect of Irradiation on Steels Used in Pressure Vessels," Group D Paper 20, Prestressed Concrete Pressure Vessels, The Institute of Civil Engineering, London, 1968.

4.3 Concrete Prestressing Steel Degradation

A posttensioned prestressing system consists of a prestressing tendon in combination with methods of stressing and anchoring the tendon to harden concrete. To attain satisfactory performance, prestressing systems are designed to have (1) consistently high strength and strain at failure, (2) serviceability throughout their lifetime, (3) reliable and safe prestressing procedures, and (4) the ability to be retensioned and replaced (nongrouted systems).[82] Prestressing systems may be grouped into three major categories, depending on the type of tendon used: wire, strand, or bar. In the United States the 8.9-MN systems, which are approved for use in containments, include (1) BBRV (wire), (2) VSL (strand), and (3) Stressteel S/H (strand). Potential degradation modes for these prestressing systems include corrosion, elevated temperature exposure, and irradiation.

4.3.1 Corrosion

Corrosion may be highly localized or uniform. Most prestressing corrosion-related failures have been the result of localized attack produced by pitting, stress corrosion, hydrogen embrittlement, or combinations of these. Pitting is an electrochemical process that results in local penetrations into the tendon to reduce the cross section to the point where it is incapable of supporting its load. Stress corrosion cracking results in brittle fracture of a normally ductile metal or alloy under stress (tensile or residual) in specific corrosive environments. Hydrogen embrittlement, frequently associated with hydrogen sulfide, occurs when hydrogen atoms enter the metal lattice and significantly reduce its ductility. Protection of the prestressing systems is provided by filling the ducts containing the posttensioned tendons either with microcrystalline waxes (petrolatums) compounded using organic corrosion inhibitors (nongrouted tendons) or with portland cement grout (grouted tendons). Regulatory requirements for inspection and replacement have made nongrouted posttensioned steel tendons the dominant prestressing system used in containments.

Reviews[83–85] of the performance of prestressing tendons contained in both nuclear power plant and conventional civil engineering structures indicate that corrosion-related incidents are extremely limited (see Chap. 3). The evolution of corrosion inhibitors and the use of organic petrolatum-based compounds designed especially for corrosion protection of prestressing materials have significantly reduced corrosion of prestressing materials. The few incidences of corrosion that were identified generally occurred early in the use of prestressed concrete for containment structures and either resulted from the use of off-the-shelf corrosion inhibitors that had not been specially formulated for prestressing materials or were the result of poor construction practices. The problems were subsequently identified and corrected during the construction phase, the initial structural integrity test, or subsequent inservice inspections.

4.3.2 Elevated temperature effects

The effect of elevated temperature on all heat-treated and drawn wires can be significant, and on cooling they do not regain their initial strength because the heating destroys the crystal transformations achieved by the heat process. Short-term heating, on the order of 3 to 5 min, even to temperatures as high as 400°C, however, may not do any harm.[86] Results of a Belgian study[10] involving 30 types of prestressing steels indicate that thermal exposures up to ~200°C do not significantly reduce (<10%) the tensile strength of prestressing wires or strands [see Fig. 40 (Ref. 10)]. Stress-strain curves for ANSI/ASTM A 421 steel (stress-relieved wire for prestressed concrete) as a function of temperature are presented in Fig. 41 (Ref. 80).

Elevated temperature exposures also affect the relaxation and creep properties of prestressing tendons. An indication of the effect of moderately elevated temperatures (20°C < T < 100°C) on the relaxation of a low-relaxation strand with data extrapolated to 50 years is presented in Fig. 42 (Ref. 87). Reference 88 indicates that losses in a 15.2-mm-diam strand initially stressed to 75% guaranteed ultimate tensile strength at 40°C will be 5 to 6.4% after 30 years. Relaxation losses of tendons composed of stress-relieved wires have relaxation losses of about the same

ORNL-DWG 86-4580 ETD

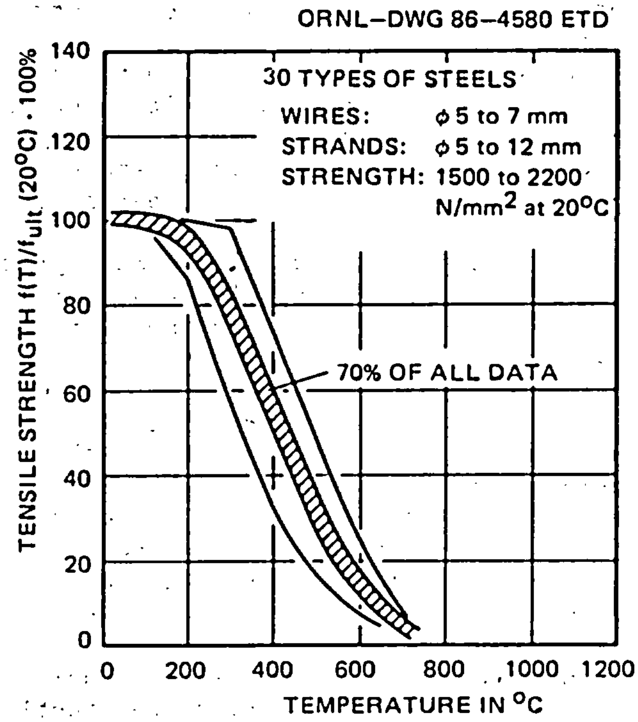


Fig. 40. Effect of temperature on ultimate strength of 30 different types of prestressing steel. *Source*: U. Schneider et al., "Effect of Temperature on Steel and Concrete for PCRV's," *Nucl. Eng. Des.* 67, 245–58 (1981).

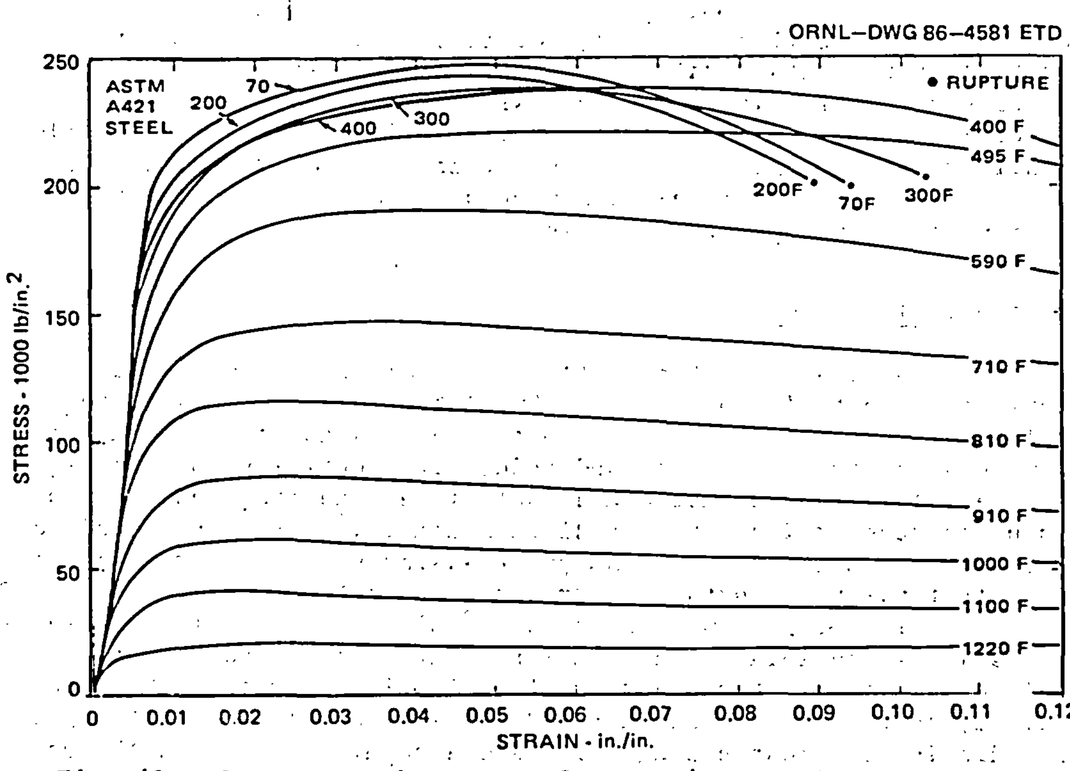


Fig. 41. Stress-strain curves for ANSI/ASTM A 421 steel at various temperatures. *Source*: T. Z. Harmathy and W. W. Stanzack, *Elevated Temperature Tensile and Creep Properties of Some Structural and Prestressing Steels*, ASTM STP 464, Fire Test Performance, 1970.

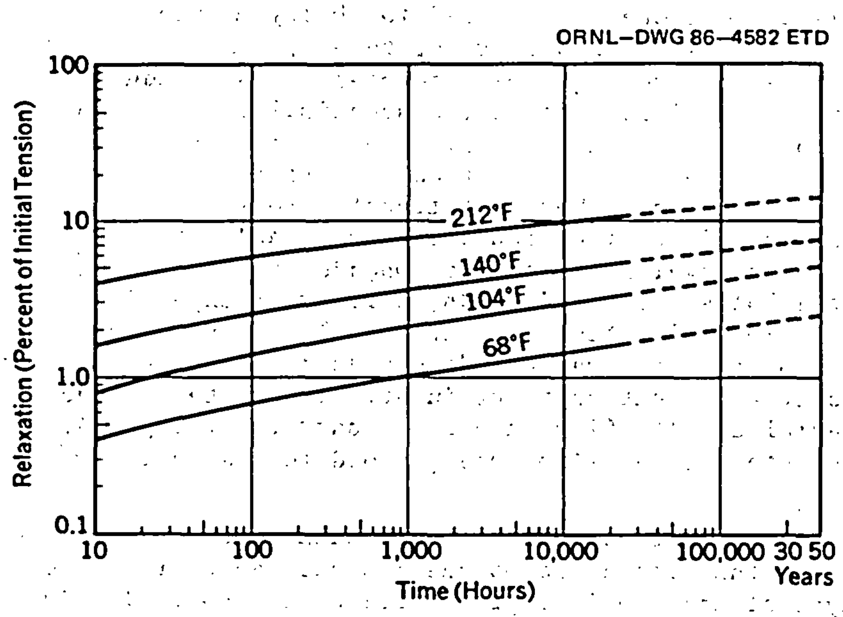


Fig. 42. Relaxation vs time curve for low-relaxation strand stressed to 70% GUTS and held at various temperatures. *Source*: J. R. Libby, *Modern Prestressed Concrete, Design Principles and Construction Methods*, Von Nostrand Reinhold Co., New York, 1971.

magnitude as stress-relieved strand, but relaxation of a strand is greater than that of its straight constituent wire because of the combined stress relaxation in the helical wires.[89] Creep (length change under constant stress) of stress-relieved wire is negligible up to 50% its tensile strength. Also, the creep effect in steel varies with its chemical composition as well as with mechanical and thermal treatment applied during the manufacturing process.

4.3.3 Irradiation effects

Irradiation of steel affects its mechanical properties because atoms are displaced from their normal sites by high-energy neutrons to form interstitials and vacancies. These defects can grow together and effectively both strengthen the steel and reduce its ductility; or, at higher temperatures, they can recombine and annihilate each other and, for a given neutron dose, reduce the irradiation damage.[81] Results obtained from studies[81] in which 2.5-mm-diam prestressing wires were stressed to 70% of their tensile strength and irradiated to a total dose of 4×10^{16} neutrons/cm^2 (flux of 2×10^{10} neutrons$\cdot$cm$^2\cdot$s) showed that for exposures up to this level the relaxation behavior of irradiated and unirradiated materials was similar. Because these flux levels are higher than the level likely to be experienced in a LWR containment, it does not appear that irradiation of prestressing will have a harmful effect on the containment.*

4.4 Anchorage Embedment Degradation

Anchorage to concrete is required for heavy machinery, structural members, piping, ductwork, cable trays, towers, and many other types of structures. An anchorage might have to meet certain requirements for ease of installation, load capacity, susceptibility to vibration, preload retention, temperature range, corrosion resistance, postinstallation or preinstallation, and ease of inspection and stiffness.[91] In meeting its function, loads that the anchor must transfer to the concrete vary over a wide combination of tension, bending, shear, and compression. Examples of types of anchors available include embedded bolts (A-307, A-325, or A-490), grouted bolts, embedded studs, self-drill expansion anchors, and wedge anchors. Several potential factors related to failure or degradation of the anchorage systems include design detail errors, installation errors (improper embedment depth or insufficient lateral cover, improper torque), material defects (low anchor or concrete strengths), shear or shear-tension interaction, slip, and preload relaxation.[91,92] Aging

---

*Irradiation of corrosion inhibitors such as used in PCCs of LWR plants indicates that there are no changes outside of the specification ranges in physical and chemical properties of the corrosion inhibitors when irradiated to 10^5 Gy (1×10^7 rad). This exceeds the gamma radiation level expected during the 40-year life of a nuclear power plant.[90]

effects that could impair the ability of an anchorage to meet its performance requirements would be primarily those that result in a deterioration of concrete properties, because if a failure did occur, it would most likely initiate in the concrete.

References

1. B. Mather, "Concrete Need Not Deteriorate," *J. Am. Concr. Inst.* 1(9), 32–37 (September 1979).

2. ACI Committee 116, *Cement and Concrete Terminology*, ACI SP-19, Detroit, 1967.

3. S. Mindess and J. F. Young, *Concrete*, Prentice-Hall, Englewood Cliffs, N.J., 1981.

4. J. Ropke, *Concrete Problems, Causes and Cures*, McGraw-Hill Book Co., New York, 1982.

5. W. H. Price, "Control of Cracking During Construction," *Concr. Int.* 4(1), 40–43 (January 1982).

6. G. E. Troxell, H. E. Davis, and J. W. Kelly, *Composition and Properties of Concrete*, 2nd ed., McGraw-Hill Book Co., New York, 1968.

7. T. C. Powers, "Resistance of Concrete to Frost at Early Ages," pp. 1–47 in *Proceedings RILEM Symposium on Winter Concreting*, Session C, Copenhagen, 1956.

8. "Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate," pp. 48–53 in *Annual Book of ASTM Standards, Part 14, Concrete and Mineral Aggregates*, ANSI/ASTM C88, American Society for Testing and Materials, Philadelphia, 1979.

9. ACI Committee 515, *Guide for the Protection of Concrete Against Chemical Attack by Means of Chemical Coatings and Other Corrosion-Resistant Materials, Part 3, Products and Processes, ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, 1977.

10. U. Schneider et al., "Effect of Temperature on Steel and Concrete for PCRVs," *Nucl. Eng. Des.* 67, 245–58 (1981).

11. Z. P. Bazant et al., *Normal and Refractory Concretes for LMFBR Applications — Vol. 1: Review of Literature on High-Temperature Behavior of Portland Cement and Refractory Concretes*, EPRI Report NP-2437, Northwestern University and Portland Cement Association, Chicago, June 1982.

12. M. S. Abrams, *Compressive Strength of Concrete at Temperatures up to 1600°F*, SP-25 Temperature and Concrete, American Concrete Institute, Detroit, pp. 33–58, 1971.

13. D. A. Campbell-Allen et al., "An Investigation of the Effect of Elevated Temperatures on Concrete for Reactor Vessels," *Nucl. Struct. Eng.* 2, 382–88 (1965).

14. *Effect of Long Exposure of Concrete to High Temperatures*, ST 32-3-53, Concrete Information, Portland Cement Association, Chicago.

15. D. J. Hannant, "The Effects of Heat on Concrete Strength," *Engineering* 197(5105), 302 (February 1963).

16. T. Harada et al., "Strength, Elasticity and Thermal Properties of Concrete Subjected to Elevated Temperatures," SP-34, vols. 1–3, Paper SP 34-21, pp. 337–406 in *Concrete for Nuclear Pressure Vessels*, American Concrete Institute, Detroit, 1972.

17. D. T. Lankard et al., "Effects of Moisture Content on the Structural Properties of Portland Cement Concrete Exposed to Temperatures Up to 500°F," SP-25 Temperature and Concrete, pp. 59–109 in *Concrete for Nuclear Pressure Vessels*, American Concrete Institute, Detroit, 1971.

18. H. L. Malhotra, "The Effect of Temperature on the Compressive Strength of Concrete," *Mag. Concr. Res.* 8(2), 382–88 (1965).

19. J. C. Marechal, "Variations in the Modulus of Elasticity and Poisson's Ratio with Temperature," SP-34, vols. 1–3, Paper SP 34-30, pp. 547–64 in *Concrete for Nuclear Reactors*, American Concrete Institute, Detroit, 1972.

20. K. W. Nasser and R. P. Lohtia, "Mass Concrete Properties at High Temperatures," *J. Am. Concr. Inst.* 68(3), 180–86 (March 1971).

21. S. Ohgishi et al., "On Properties of Magnetite and Serpentine Concrete at Elevated Temperatures for Nuclear Reactor Shielding," SP-34, vols. 1–3, Paper SP 34-57, pp. 1243–53 in *Concrete for Nuclear Reactors*, American Concrete Institute, Detroit, 1972.

22. F. J. Roux, *Concrete at Elevated Temperatures*, Ph.D. thesis, University of Capetown, South Africa, August 1974.

23. N. G. Zoldners, "Effect of High Temperatures on Concrete Incorporating Different Aggregates," *Am. Soc. Test. Mater. Proc.* 60, 1087–1108 (1960).

24. *Behavior of Concrete Under Temperature Extremes*, Publication SP-39, American Concrete Institute, Detroit, 1973.

25. D. J. Naus, *Task 2: Concrete Properties in Nuclear Environment — A Review of Concrete Material Systems for Application to Prestressed Concrete Pressure Vessels*, ORNL/TM-7632, Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., May 1981.

26. R. Kottas et al., "Strength Characteristics of Concrete in the Temperature Range of 20° to 200°C," Paper H1/2, vol. H, Session 1, Paper 2, *5th Int'l. Conf. on Structural Mechanics in Reactor Technology*, Berlin, Germany, Aug. 13–17, 1979.

27. U.S. Bureau of Reclamation, *A 10-Year Study of Creep Properties of Concrete*, Concrete Laboratory Report No. SP-38, Denver, July 28, 1953.

28. Z. P. Bazant and S. Prasannan, *High-Temperature Triaxial Torsional Creep Tests of Concrete at Various Hygral Conditions*, Report No. 85-9/285 h, Center for Concrete and Geomaterials, The Technical Institute, Northwestern University, Evanston, Ill., September 1985.

29. Z. P. Bazant et al., *Large Triaxial-Torsional Testing Machine with Hygrothermal Control*, Report No. 85-7/285 L, Center for Concrete and Geomaterials, The Technical Institute, Northwestern University, Evanston, Ill., July 1985.

30. G. E. Troxell et al., *Long-Time Creep and Shrinkage Tests of Plain and Reinforced Concrete*, ASTM Proc. 48, American Society for Testing and Materials, Philadelphia, pp. 1101–20, 1958.

31. D. Campbell-Allen and P. M. Desai, "The Influence of Aggregate on the Behavior of Concrete at Elevated Temperature," *Nucl. Eng. Des.* 6(1), 65–77 (August 1967).

32. V. V. Bertero and M. Polivka, "Influence of Thermal Exposure on Mechanical Characteristics of Concrete," SP-34, Paper 28, pp. 505–31 in *Concrete for Nuclear Reactors*, American Concrete Institute, Detroit, 1972.

33. G. G. Carette et al., "Sustained High Temperature Effects on Concretes Made with Normal Portland Cement, Normal Portland Cement and Slag, or Normal Portland Cement and Fly Ash," *Concr. Int.* 4(7), 41–51 (July 1982).

34. A. P. Mears, "Long Term Tests on the Effect of Moderate Heating on the Compressive Strength and Dynamic Modulus of Elasticity of Concrete," SP-34, Paper 20, pp. 355–75 in *Concrete for Nuclear Reactors*, American Concrete Institute, Detroit, 1972.

35. C. DeFigh-Price et al., *Effects of Long-Term Exposure to Elevated Temperature on Mechanical Properties of Concrete*, ACI Symposium, Concrete and Cementitious Materials for Radioactive Waste Management, New York, Nov. 1, 1984.

36. M. P. Gillen et al., *Strength and Elastic Properties of Concrete Exposed to Long-Term Moderate Temperatures and High Radiation Fields*, RHO-RE-SA-55 P, Rockwell Hanford Operations, Richland, Wash., 1984.

37. H. Kagami et al., "Properties of Concrete Exposed to Sustained Elevated Temperatures," *3rd Int'l. Conf. on Structural Mechanics in Reactor Technology*, Paper H1/5, London, 1975.

38. A. F. Milovanov and G. D. Salmanov, "The Influence of High Temperature Upon the Properties of Reinforcing Steels and Upon Bond Strength between Reinforcement and Concrete," Issledovanija po zharoupornym betonu i zhelezobetonu, pp. 203–23 (1954) (in Russian).

39. C. B. Oland and J. P. Callahan, *Bond Between Concrete and Steel Reinforcement at Temperatures to 149°C (300°F)*, ORNL/TM-6086, Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., April 1978.

40. U. Diederichs and U. Schneider, "Bond Strength at High Temperature," *Mag. Concr. Res.* 35(115), 75–84 (June 1981).

41. E. G. Peterson, *Shielding Properties of Ordinary Concrete as a Function of Temperature*, HW-65572, Hanford Atomic Products Operation, Richland, Wash., Aug. 2, 1960.

42. S. Miyasaka et al., "Gamma-Ray Leakage Through a Slit in Concrete Shield," *Nippon Genshiryaku Gakkaishi* 11, 2–8 (1969) (in Japanese).

43. F. Seboek, *Shielding Effectiveness of Cracked Concrete*, Kerntechnik 12, 496–501 (November 1970).

44. H. K. Hilsdorf et al., "The Effects of Nuclear Radiation on the Mechanical Properties of Concrete," SP-55, Paper 55-10, pp. 223–51 in *Douglas McHenry, International Symposium on Concrete and Concrete Structures*, American Concrete Institute, Detroit, 1978.

45. C. A. Negin et al., *Extended Life Operation of Light-Water Reactors: Economic and Technological Review*, EPRI NP-2418, vols. 1 and 2, Electric Power Research Institute, Palo Alto, Calif., June 1982.

46. "Code for Concrete Reactor Vessels and Containments," Sect. III-Div. 2, Nuclear Power Plant Components, *American Society of Mechanical Engineers Boiler and Pressure Vessel Code*, New York, 1983.

47. *Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors*, BS5975, British Standards Institution, London, July 1973.

48. V. B. Dubrovskij et al., "The Effect of Neutron Irradiation on Certain Properties of Refractory Concretes," *At. Energ.* 21, 108–12 (1966).

49. M. R. Elleuch, F. Dubois, and J. Rappenau, "Behavior of Special Shielding Concretes and Their Constituents Under Neutron Irradiation," *Fourth United Nations International Conference on the Peaceful Uses of Atomic Energy*, 1971.

50. B. S. Gray, "The Effect of Reactor Radiation on Cements and Concrete, Commission of the European Communities, Luxembourg," pp. 17–39, 1972.

51. T. C. Hansen and L. Erikson, "Temperature Change Effect on Behavior of Cement Paste, Mortar and Concrete Under Load," *Am. Concr. Inst. Proc.* 63(4), 489–504 (April 1966).

52. J. A. Houben, *DeBestraling van Mortelproefstukken* [*Radiation of Mortar Specimens*], Commission of the European Communities, Brussels, 1969, pp. 170–83.

53. B. T. Kelly and I. Davidson, "Irradiation Effects on Concrete," Group C Paper 16, pp. 173–5 in *Conference on Prestressed Concrete Pressure Vessels at Church House, Westminster, S.W.I., Institution of Civil Engineers, London, March 13–17, 1967*, 1968.

54. W. Meyer and H. K. Al-Dujaili, "Radiation Effects on Concrete and Carbonate Compounds," *Second Cairo Solid State Conference*, Cairo, 1973.

55. A. Pederson, *Radiation Damage in Concrete — Measurements on Miniature Specimen of Cement Mortar*, Commission of the European Communities, Luxembourg, Belgium, 1971, pp. 5–16.

56. B. T. Price, C. C. Horton, and K. T. Spinney, "Radiation Shielding," pp. 276–78 in *International Series of Monograph on Nuclear Energy*, Pergamon Press, New York, 1975.

57. C. R. Van der Schaaf, *Effect of Heating and Radiation on Some Properties of Mortar and Concrete Specimens with Different Compositions*, Commission of the European Communities, Brussels, 1967, pp. 42–71.

58. J. F. Sommers, "Gamma Radiation Damage of Structural Concrete Immersed in Water," *Health Physics*, Pergamon Press, Vol. 16, 1969, pp. 503–8.

59. B. Stoces et al., *The Effect of Radiation on the Mechanical Properties of Concrete*, Ceskoslovenska Akademie, Translated from the Czech; Oak Ridge National Laboratory, Purchase Order: 34B-83481, Letter Release No.: T81, STS No.: 14087.

60. R. L. Walker and M. Grotenhuis, *A Summary of Shielding Constants for Concrete*, ANL-6443, Argonne National Laboratory, Argonne, Ill., November 1961.

61. M. F. Eleuch and F. Dubois, "Les Experiences d'Irradiation des Betons et de leurs Constituants," Commission of the European Communities, Brussels, 1972, pp. 71–96.

62. S. Granata and A. Montaginini, "Behavior of Plastic Mortar Samples Under Temperature and Radiation Conditions," *Information Exchange*

Meeting Concerning the Results of Concrete Irradiation Programmes, 18 pp., 1971.

63. B. T. Kelley et al., "The Effects of Reactor Radiation on Concrete," pp. 237–66 in *Proceedings of the Second Conference on PCPVs and Their Thermal Isolation*, Commission of the European Communities, Brussels, November 1969.

64. D. C. McDowall, *The Effect of Gamma Irradiation on the Creep Properties of Concrete*, Euratom Report No. EUR 4751, pp. 55–69, Commission of European Communities, Luxembourg, 1971.

65. R. G. Clark, *Radiation Damage to Concrete*, HW-56195, Hanford Atomic Products Operation, Richland, Wash., Mar. 31, 1958.

66. T. V. Blosser et al., *A Study of the Nuclear and Physical Properties of the ORNL Graphite Reactor Shield*, ORNL-2195, Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., August 1958.

67. M. F. Elleuch et al., "Effects of Neutron Radiation on Special Concretes and Their Components," SP-34, vols. 1–3, Paper 34-51, pp. 1071–1108 in *Concrete for Nuclear Pressure Vessels*, American Concrete Institute, Detroit, 1972.

68. H. S. Davis, "N-Reactor Shielding," SP-34, vols. 1–3, Paper 34-52, pp. 1109–61 in *Concrete for Nuclear Pressure Vessels*, American Concrete Institute, Detroit, 1972.

69. S. Granata and A. Montagnini, "Studies of Behavior of Concretes Under Irradiation," SP-34, vols. 1–3, Paper 34-53, pp. 1163–72 in *Concrete for Nuclear Pressure Vessels*, American Concrete Institute, Detroit, 1972.

70. S. C. Alexander, *Effects of Irradiation on Concrete, Final Results*, Atomic Energy Research Establishment, Harwell, England, 1963.

71. A. W. Ch. Batten, *Effect of Irradiation on Strength of Concrete*, Atomic Energy Research Establishment, Harwell, England, 1960.

72. G. Christiana et al., *Behavior of Plastic Mortar Samples Under Temperature and Radiation Conditions*, Commission of European Communities, Luxembourg, 1972.

73. E. Crispino et al., "Behavior of Concrete in the Presence of Thermal Stresses and Radiation," *2nd Information Meeting on Prestressed Concrete Reactor Pressure Vessels and Their Thermal Isolation*, Commission of the European Communities, Brussels, November 1969.

74. E. Crispino et al., "Irradiation Effects on Reactor Structural Materials," *Fourth United Nations International Conference on Peaceful Uses of Atomic Energy*, 1971.

75. Danish Atomic Energy Commission, *Radiation Damage in Concrete — Measurements on Miniature Specimens of Cement Mortar*, Communications Research Establishment Riso, Reactor Engineering Department, Roskilde, Denmark, 1971.

76. R. N. White and P. Gergeley, "Prestressed Concrete Pressure Vessels," short course, Cornell University, Ithaca, N.Y., June 16—20, 1969.

77. P. K. Mehta and B. C. Gerwick, Jr., "Cracking-Corrosion Interaction in Concrete Exposed to Marine Environments," *Concr. Int.* 4(10), 45—51 (October 1982).

78. D. Darwin et al., "Debate: Crack Width, Cover and Corrosion," *Concr. Int.* 7(5), 20—35 (May 1985).

79. A. W. Beeby, "Cracking, Cover and Corrosion of Reinforcement," *Concr. Int.* 5(2), 35—40 (February 1983).

80. T. Z. Harmathy and W. W. Stanzack, *Elevated Temperature Tensile and Creep Properties of Some Structural and Prestressing Steels*, ASTM STP 464, American Society for Testing and Materials, Philadelphia, 1970.

81. A. Cowan and R. W. Nichols, "Effect of Irradiation on Steels Used in Pressure Vessels," Group D Paper 20, pp. 229—35 in *Prestressed Concrete Pressure Vessels*, Marilyn S. Udall, ed., The Institute of Civil Eng., London, 1968.

82. C. P. Tan, *Prestressed Concrete in Nuclear Pressure Vessels, A Critical Review of Current Literature*, ORNL-4277, Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., May 1968.

83. *Technical Report — An International Survey of In-Service Inspection Experience with Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors*, FIP/3/5, FIP Commission on Concrete Pressure and Storage Vessels, Federation Internationale de la Precontrainte, Wexham Springs, Slough, U.K., 1982.

84. D. J. Naus, *An Evaluation of the Effectiveness of Selected Corrosion Inhibitors for Protection of Prestressing Steels in PCPVs*, ORNL/TM-6479, Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., March 1979.

85. H. Ashar and D. J. Naus, "Overview of the Use of Prestressed Concrete in U.S. Nuclear Power Plants," Paper H1/1, *7th International Conference on Structural Mechanics in Reactor Technology*, Chicago, August 1983.

86. W. Podolny, Jr. and T. Melville, "Understanding the Relaxation in Prestressing," *PCI J.* 14(4) (August 1969).

87. J. R. Libby, *Modern Prestressed Concrete, Design Principles and Construction Methods*, Von Nostrand Reinhold Co., New York, 1971.

88. T. Cahill and G. D. Branch, "Long-Term Relaxation Behavior of Stabilized Prestressing Wires and Strands," Group D, Paper 19, pp. 219–28 in *Prestressed Concrete Pressure Vessels*, Marilyn S. Udall, ed., The Institute of Civil Eng., London, 1968.

89. S. Freedman, "Properties of Materials for Reinforced Concrete," *Handbook of Concrete Engineering*, ed. M. Fintel, Van Norstrand Reinhold Co., New York, 1974.

90. *Product Data Sheet, Visconorust 2090P-4 Casing Filler*, Viscosity Oil Company, Chicago, 1975.

91. R. R. Funk, "Testing Anchorage to Concrete," *Civ. Eng. Nucl. Power* 1, 6-1-1 to 6-1-17 (Sept. 15–17, 1980).

92. D. F. Meinheit and F. D. Heidbrink, "Behavior of Drilled-In Expansion Anchors," *Concr. Int.* 7(4), 62–66 (April 1985).

5. CURRENT TECHNOLOGY FOR DETECTION OF CONCRETE AGING PHENOMENA

Tests are conducted on concrete to assess future performance of a structure as a result of (1) noncompliance of strength tests; (2) inadequacies in standards for placing, compacting, or curing of concrete in the structure; (3) damage resulting from overload, fatigue, frost, abrasion, chemical attack, fire, explosion, or weathering;* and (4) concern about the capacity of the structure to withstand design, actual, or projected loading conditions.[1] Item (4) of this list is of interest to the present study because it pertains to life extension considerations.

Information presented in Chap. 4 indicates that the ability of a concrete component to continue to meet its functional and performance requirements over an extended period of time is dependent on the durability of its constituents. Techniques for detection of concrete component degradation should, therefore, address evaluation of the concrete, mild steel reinforcing, prestressing system, and anchorage embedments. In the following sections, the various methods for inspection of concrete materials are described, and recommendations are provided on techniques to be used in the evaluation of light-water reactor (LWR) concrete components.

5.1 Evaluation of Concrete Materials

Sources of distress that are present or can occur in concrete materials include (1) cracking, voids, and delamination and (2) strength losses. Although not an aging-related phenomenon, whether the concrete was cast having the specified mix composition could also become a life extension consideration.

5.1.1 Detection of concrete cracking, voids, and delamination

Discontinuities in concrete structures can be detected by visual inspection, nondestructive testing, or examination of cores.

5.1.1.1 Visual inspection. Periodic visual examinations of exposed concrete provide a rapid and effective method for identifying and defining areas of distress (i.e., cracking, spalling, volume change, or cement/aggregate interaction). By locating, marking, identifying by type and orientation, and measuring and recording conditions associated with the cracks (seepage, differential movement, edge spalling, etc.), a history that will be of assistance in identifying the cause and establishing whether a crack is active or dormant can be established. A crack comparator capable of width determinations to an accuracy of 0.025 mm can be used to establish cracks that are above a critical size required to permit the entry of hostile environments to attack either the concrete or

---

*Prolonged exposure to elevated temperature and irradiation conditions should be added to this list for nuclear applications.

its steel reinforcement.*[2,3] Subsurface cracking, delaminations and voids, and the extent of cracking, however, cannot be established through visual examinations.

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

Ultrasonic and stress wave methods. Detection of cracks or voids in concrete by using ultrasonic through-transmission measurements is based on the principle that the amplitude and direction of travel of ultrasonic compressional pulses propagating through concrete will be changed when they encounter a crack.[4] The ultrasonic pulses are emitted by a transducer, and the transit time to a receiver is measured by electronic means in terms of either transit time (microseconds) or path length. Crack widths >0.0254 mm are detectable because transmission across air-filled voids of this size has been shown to be negligible.[5] Large internal flaws in concrete can be detected by an abnormally long transmission time and/or a large decrease in amplitude of the ultrasonic pulses as they pass around a crack. Primary advantages of the technique are that it is an excellent method for rapidly estimating the quality and uniformity of concrete and that a low level of user expertise is required to make measurements. Disadvantages are that sound transmissions through concrete are influenced by a number of conditions (Fig. 43, Ref. 6), and quantitative interpretation of results is difficult.

Sonic coring, a form of ultrasonic testing, has been proposed as a method for detecting construction faults in concrete pressure vessels and for detecting faults in concrete shields.[7] The method consists of lowering transmitter and receiver probes to the bottom of adjacent tubes (i.e., prestressing tendon conduits or drilled core cavities), filling the tubes with water for coupling, and slowly raising the probes (20 cm/s) with the signals continuously monitored by photographic means, using modulation of light intensity to represent signal intensity. An area of bad concrete will be indicated by deflection of the first wave. Although this method has been successfully used for pile and diaphragm wall construction quality control, its application to LWR concrete components is presumed unlikely because of the large number of tendons and the requirement to fill tubes with water as a couplant and because the scattering effects of multiple layers of reinforcement would make data interpretation extremely difficult.

The pulse-echo technique, which provides an alternative to the through-transmission methods,[8] is based on monitoring the interaction of acoustic (or stress) waves with the internal structure of an object.[9] An

---

*Critical crack widths for entry of a hostile environment vary significantly depending on exposure conditions. Reference 2 notes that corrosion of steel does not occur in concrete exposed to severe conditions having crack widths <0.2 mm and in protected conditions where the crack width is <0.3 mm. Additional information on tolerable crack widths as a function of exposure condition is presented in Ref. 3.

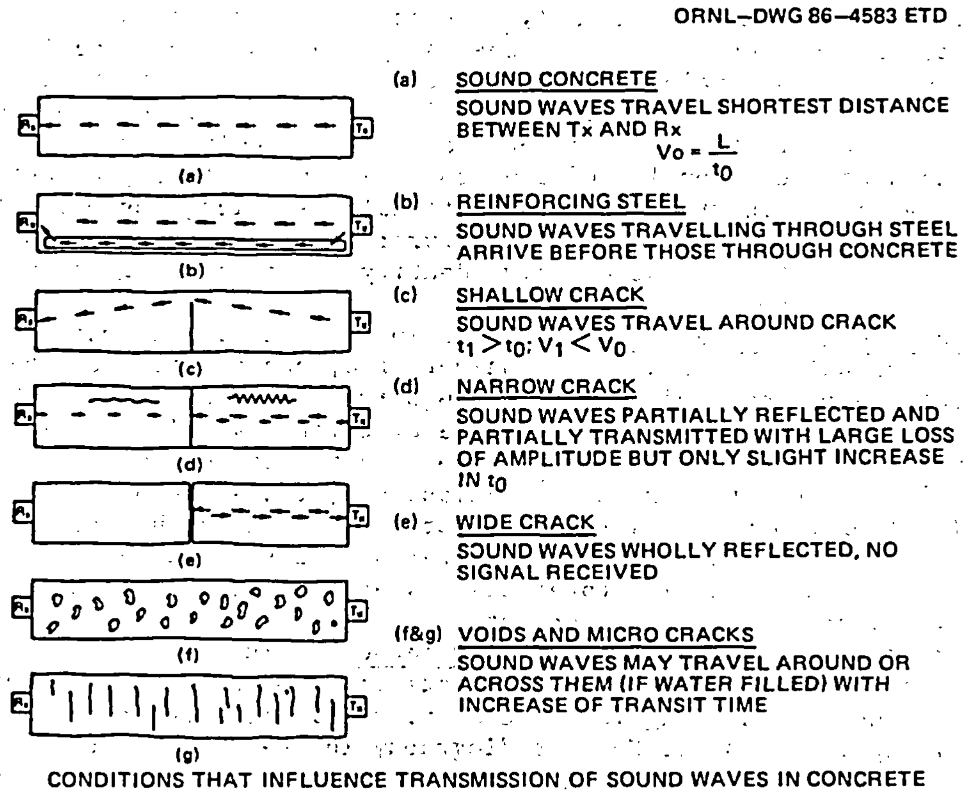


Fig. 43. Conditions that influence transmission of sound in concrete. *Source*: V. R. Stirrup et al., "Pulse Velocity as a Measure of Concrete Compressive Strength," *In Situ/Nondestructive Testing of Concrete*, SP-82, Paper 11, American Concrete Institute, Detroit, 1984.

acoustic pulse is introduced into the test object by either an electromechanical transducer (pulse repeatability good) or mechanical impact (simple). Concrete defects are detected by a reduction in the penetrating ability of the high-frequency waves. The primary advantage of this technique relative to the through-transmission is that only one face of the member needs to be accessible because the reflected signal is used. Principal difficulties in application of the technique to concrete are that the concrete heterogeneity prevents direct application of the methods developed for inspecting metallic structures, a transducer producing both a highly penetrating and relatively narrow ultrasonic beam has not been perfected, and interpretation of results can be difficult.

Acoustic emissions are small-amplitude elastic stress waves generated during material deformation resulting from a mechanical or thermal stimulus. The stress waves are detected by transducers as small displacements on the specimen surface. Acoustic emission has been applied to concrete for almost 30 years to detect (and locate) distress (cracking) in concrete components.[10] Because acoustic emissions are indicators of increasing stress levels in, and potential subsequent deformation of, a structure, they can potentially be used to nondestructively determine the degree of damage that a structure has experienced; that is, the method, therefore, can potentially be used in evaluating the remaining integrity of a structure that has been subjected to an extreme loading condition or in estimating the in-service ability of a structure to carry new loads in excess of those anticipated during its original design.[11] Quantitative application of the technique is difficult, however, because it requires an understanding of the basic mechanisms that generate micro-seismic waves within structures and materials; knowledge by which the disturbances propagate through a structure; and development of sophisticated instrumentation to accurately identify the nature, severity, and location of the source.

Acoustic impact methods. Acoustic impact methods, in which the concrete surface is struck with a hammer, rod, chain, etc., can be used to detect the presence of defects through frequency and damping characteristics of the "ringing." A portable electronic version developed by the Texas Highway Department is capable of detecting delaminations up to 66 mm below a concrete surface.[12] Advantages of the technique are that a low level of expertise is required for use and the method does not involve complicated electronic instrumentation. Disadvantages are that experience is required to interpret results and results are affected by geometry and mass of the test object.

Radiography and penetrating radar methods. The radioactive methods (X- and gamma-ray techniques) are potentially promising for determining concrete density, locating reinforcement, and identifying concrete honeycombing. Applications of X-ray radiography in the field, because of its relatively high initial cost and limited mobility of testing equipment, have been limited to establishing rebar location, investigating bond stress in prestressed concrete, and showing concrete density variations. Gamma-ray radiography, because of its use of less costly portable equipment and its ability to make measurements up to concrete thicknesses of 450 mm, has been more widely used to determine position and condition of reinforcement, voids in grouting of posttensioned prestressed concrete, voids in concrete, and variable compaction in concrete.[13] Advantages of gamma-ray radiography are that it uses portable and relatively inexpensive equipment (relative to X-ray radiography) and can detect internal defects in a number of materials. Disadvantages are that the radiation intensity cannot be adjusted (thus, long exposures may be required), it uses potentially dangerous radiation, and operators must be highly trained and licensed.

Penetrating radar using electromagnetic energy in the 100- to 1200-MHz frequency range can be used for nondestructive evaluations of concrete. The waves propagate through the concrete until a boundary (materials with different dielectric properties) is intercepted; then part of the incident energy is reflected, picked up by a receiver, and indicated by a change in wave shape. Radar traces are easily able to indicate voids and severely deteriorated material.[14] Advantages of the technique are that large areas of concrete can be rapidly surveyed and internal construction details and defects identified. Disadvantages are that where material differences are small — such as a crack in sound material or a contact delamination — transmission differences are hard to detect and evaluate and material permittivity must be known to determine the interface depth.

Thermal methods. Heat-sensing devices are used to detect irregular temperature distributions caused by the presence of flaws or inhomogeneities in a material or component that has different impedances to heat flow. Contours of equal temperature (thermography) or temperature levels (thermometry) are measured over the test surface with contact or noncontact detection devices. A common detection device is an infrared scanning camera. Advantages of the technique are that it is portable, a permanent record can be made, testing can be done without direct access to the surface, and large areas can be rapidly inspected. Disadvantages are that the equipment is costly, reference standards are required, and moderate to extensive operator expertise is required.

5.1.1.3 Examination of cores. Visual and nondestructive testing methods are effective in identifying areas of concrete exhibiting distress but often cannot quantify the extent or nature of the distress. Cores obtained from these areas provide the only direct means to evaluate the width and depth of cracking or the extent of voids.

5.1.2 In-situ concrete strength determinations

In conventional civil engineering structures little attention is given to the in-situ concrete strength because 28-d (or older) moist-cured control specimens are used to indicate the correct strength in a particular structure,* and very few concrete structures actually fail.[15,16] However, for a structure that is being considered for extension beyond its designed service life, especially a structure that has been subjected to a less than ideal operating environment, the in-situ strength of the concrete takes on a new meaning. Available methods used to evaluate the strength of concrete in a structure include both direct (testing of core specimens) and indirect techniques (ultrasonic pulse velocity, surface hardness, rebound, penetration, pullout, and breakoff).

5.1.2.1 Direct techniques. Testing of core samples in conformance with Ref. 17 requirements provides a direct method for obtaining the in-situ concrete strength. The effects of various factors (core diameter, slenderness ratio, location, etc.) are presented in Refs. 18 and 19. As noted in Ref. 20, current American Concrete Institute statistical standards relative to the number of tests required to ensure that the probability of obtaining a strength less than desired is below a certain level are not applicable for in situ tests. Such standards should be developed to achieve reliability of in-situ strength results.

5.1.2.2 Indirect techniques. Indirect techniques measure some property of concrete from which an estimate of the strength is made

---

*In-situ strength of 28-d concrete is normally significantly less (20 to 25%) than 28-d standard control specimen strength of the same concrete because of different compaction and curing conditions.[1] Also, systematic variation of concrete occurs in a structure because of segregation that can reduce concrete strength at the top of a lift by 15 to 30% (Ref. 16).

through correlations that have been developed.*[21] Nondestructive testing techniques considered as indirect measures of strength include those based on surface hardness, penetration resistance, pullout resistance, break-off resistance, and ultrasonic pulse velocity.

Surface hardness methods. Three test methods (Williams testing pistol, Frank spring hammer, and Einbeck pendulum) have been developed in which the increase in hardness with age of concrete is used to indicate compressive strength. These methods are all based on the principle of impacting the concrete surface by using a given mass activated by a given energy and then measuring the size of the indentation. Although all of these methods are simple to use and provide a large number of readings in a short time, frequent calibration is required, cement type may affect results, and strength can generally be determined with an accuracy of only 20 to 30%.

Rebound methods. The Schmidt rebound hammer is basically a form of surface hardness tester in which a spring-loaded weight is impacted against the concrete surface and a rebound number is obtained. Concrete strength is then determined from a manufacturer-supplied chart or from a laboratory-generated calibration chart. The primary usefulness of the device is in assessing concrete uniformity in situ, delineating zones (or areas) or poor quality or deteriorated concrete in structures, and indicating changes with time of concrete characteristics. Test requirements are contained in ASTM C 805 *Standard Test Method for Rebound Number of Hardened Concrete*.[22] Advantages of the technique are that user expertise requirements are minimal and a large amount of data can be developed quickly and inexpensively. Disadvantages are that test results are affected by concrete surface conditions and the technique only provides a "rough" indication of compressive strength.

Penetration methods. This type of test, as described in ASTM C 803 *Tentative Test Method for Penetration Resistance of Hardened Concrete*,[23] involves measurement of the resistance of concrete to penetration by a steel probe driven by a given amount of energy. The most common device of this type is the Windsor Probe, consisting of a powder-activated driving unit that propels a hardened alloy probe into the concrete and a depth gage for measuring penetration. Compressive strength is determined through calibration curves. Advantages and disadvantages of the technique are essentially the same as for the rebound methods.

Pullout resistance methods. Pullout tests, in conformance with requirements provided in ASTM C 900 *Tentative Test Method for Pullout Strength of Hardened Concrete*.[24] involve a determination of the force required to pull a steel insert out of concrete. Essentially, the method provides a measure of the shear strength of concrete, which is converted to tensile or compressive strength through correlations. Most of the pullout methods, such as the LOK-test, require embedment of the metal pullout insert in fresh concrete.[20] Testing of hardened concrete is done

---

*The primary application of these tests is to indicate differences in concrete quality from one part of a structure to another, thus indicating areas requiring closer examination through drilling cores and conducting petrographic studies.[21]

using the CAPO test, which is similar to the LOK-test except that the insert is drilled and expanded wherever required in situ.[25] Hardened concrete can also be tested by using a circular probe bonded by epoxy resin to either a cored or uncored concrete surface[26] and by the BRE internal fracture test[27] in which a 6-mm-diam hole is drilled into the concrete, a wedge anchor placed into the hole, and the torque required to pull the anchor bolt out of the concrete determined. Advantages of these methods are that they are one of the only nondestructive methods that directly measure in-place strength and they are economical and rapid. Disadvantages are that they do not measure the interior strength of mass concrete and they result in the requirement for minor concrete surface repairs.

Breakoff resistance methods. In-situ concrete compressive strength can be determined from the breakoff strength of concrete cores formed either by plastic inserts while the concrete was plastic or by drilling hardened concrete. Bending force applied at right angles to the top of the core at the point of rupture is taken as a measure of the concrete flexural strength, which is then related through calibration curves to the compressive strength.[28] Small cores can then be taken to the laboratory for further examination. Advantages and disadvantages are similar to those for the pullout resistance methods.

Ultrasonic pulse velocity methods. Ultrasonic pulse velocity methods are based on the fact that the velocity of sound in a material is related to the elastic modulus and material density. Because the pulse velocity depends only on the elastic properties of the material and not on the geometry, it is a very convenient technique for evaluating concrete quality (i.e., concrete quality proportional to pulse velocity). Procedures and apparatus for determining pulse velocity through concrete are contained in ASTM C 597 *Standard Test Method for Pulse Velocity Through Concrete.*[29] Advantages of the technique are that it is a rapid and cost-effective method for measuring in-situ concrete uniformity, the method is totally nondestructive, and it can be utilized to "estimate" in-situ concrete strength within 15 to 20% if a good correlation curve has been developed.* Disadvantages are that results are affected by contact surface smoothness, pulse velocity is somewhat path dependent, pulse velocity can be affected by temperatures outside the range of 5 to 30°C, the presence of steel bars parallel to transmission path affects results, and for a given pulse velocity the compressive strength is higher for older specimens.[30,31]

5.1.3 Mix composition analysis of hardened concrete

Questions concerning whether the concrete in a structure was cast using the specified mix composition can be answered through examination of core samples.[30,32] By using a "point count" method described in ASTM C 457 *Standard Recommended Practice for Microscopical Determination of Air-Void Content,*[33] the nature of the air void system (volume and spacing) can be determined by examining under a microscope a polished section

---

*Utilization of ultrasonic pulse velocity to detect cracks and voids in concrete was discussed in Sect. 5.1.1.2.

of the concrete. An indication of the type and relative amounts of fine and coarse aggregate, as well as the amount of cementitious matrix, can be determined by using ANSI/ASTM C 856 *Standard Recommended Practice for Petrographic Examination of Hardened Concrete*.[34] Cement content can be determined chemically by using ANSI/ASTM C 85 *Standard Test Method for Cement Content of Hardened Portland Cement Concrete*.[35] Determination of the original water-cement ratio is not covered by an ASTM standard but can be estimated by using a British Standard (BS 1881, Part 6)[36] that determines the volume of capillary pores originally filled with water and the combined water (original water is the sum of these two). A standard method also does not exist for determining either the type or amount of chemical admixtures used. With respect to mix composition for concretes that have aged considerably, the determinations are more difficult, especially if the concrete has been subjected to leaching by chemical attack or carbonation.

5.2 Evaluation of Mild Steel Reinforcing Materials

The primary source of distress to which mild steel reinforcement could be subjected would be corrosive attack.* Implications of safety and serviceability of structures undergoing appraisal as a result of rebar corrosion should consider effects on three levels: (1) effect on rebars themselves (cross section or property reductions), (2) development of fine hairline cracks in concrete cover parallel to rebars (indicates deterioration), and (3) structural cracking or voids (preferential corrosion sites). Safety implications of reinforcement corrosion depend primarily on the structural form or system of construction; second, on the way in which the geometry of the structural components may be affected; and, third, to a lesser extent, on the total amount of corrosion of the rebars.[37] Techniques available for corrosion monitoring and inspection of steel in concrete include (1) visual inspection, (2) mechanical and ultrasonic tests, (3) core sampling and chemical and physical tests, (4) potential mapping, and (5) rate of corrosion probes.[38,39]

5.2.1 Visual inspection

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

---

*Mild steel reinforcement in LWR concrete safety-related components under normal operating conditions probably would not be subjected to levels of irradiation or elevated temperatures sufficiently high to produce a reduction in properties.

5.2.2 Mechanical and ultrasonic tests

Surface tapping using techniques described previously for detection of delamination can be used to define the area potentially affected by corrosion. If a rebound or Schmidt hammer is used to impact the surface, comparative information may also be obtained on concrete quality.

Ultrasonic pulse velocity measurements also may be used to detect areas experiencing cracking or delamination caused by corrosion.

5.2.3 Core sampling and chemical and physical tests

Cores obtained from areas indicating distress as determined by either of the two previous techniques provide a direct method for examining and assessing the extent of corrosion. A pachometer, or cover meter,* can be used to detect the presence, and in some cases the depth or size of reinforcement, so that the core can be obtained without further damaging the steel.

Chemical analysis for chloride or sulfate distribution can be conducted on samples obtained by coring or from dust obtained by drilling. Measurement of diffusion parameters for oxygen and chloride ions provides an indication of the ease with which contaminants enter the concrete. Areas (depths) that are alkaline and, thus, able to protect the reinforcement can be identified by using phenolphthalein.

Concrete composition and performance can also be indicated through electrical resistivity measurements. A high value of resistivity (>12000 Ω-cm) indicates that corrosion from galvanic effects is of reduced threat.[39]

5.2.4 Potential mapping

Information on the passivity of reinforcing steel can be obtained through corrosion potential measurements by using a reference electrode placed on the concrete surface and connected by means of a high-impedance voltmeter ($>10^9$ Ω) to the reinforcement. The probability of an area exhibiting corrosion decreases as the half-cell potential measurement approaches zero. More details are provided in ANSI/ASTM C 876 *Standard Test Method for Half Cell Potentials of Reinforcing Steel in Concrete*.[40]

5.2.5 Rate of corrosion probes

Two types of probes that can be embedded into concrete to provide an indication of the rate of corrosion are available.[41] The first type uses two or three electrically isolated short sections of steel wire or reinforcing steel and linear polarization techniques. The second device uses

---

*The presence of steel affects the magnetic field of a probe with the effect increasing with proximity of the steel.

a steel wire or hollow cylinder embedded into concrete to provide cumulative rate of corrosion data from periodic measurements. The primary application of these devices has been to evaluate the effect of rehabilitation procedures on the corrosion rate.

5.3 Evaluation of Prestressing Steel Materials

The ability of a prestressed concrete containment to withstand the loadings that would develop as a result of a loss-of-coolant accident depends on the continued integrity of the tendons. In the United States the condition and functional capability of unbonded posttensioning systems must be periodically assessed. This is accomplished, in part, systematically through an in-service inspection program that must be developed and implemented for each containment. Requirements for containment tendon surveillance programs in the United States are presented in (1) Regulatory Guide (RG) 1.35 (proposed revision 3),[42] (2) RG 1.35.1 (proposed),[43] (3) ASME (proposed Subsection IWX),[44] and (4) U.S. NRC Standard Technical Specification for Tendon Surveillance.[45]

The present basis for conducting tendon inspections is presented in RG 1.35 *Inservice Inspections of Ungrouted Tendons in Prestressed Concrete Containment Structures (Rev. 2)*.[46] The intent of RG 1.35 is to provide the utilities with a basis for developing inspection programs and to provide reasonable assurance, when properly implemented, that the structural integrity of the equipment was being maintained. Basic components covered in the RG include sample selection, visual inspection, prestress monitoring tests, tendon material tests and inspections, and inspection of filler grease.

Tendon sample selection criteria are specified for typical prestressed concrete containments. If no problems are uncovered during the first three surveillances (scheduled 1, 3, and 5 years after the initial structural integrity test), then the criteria for sample selection are relaxed. In all cases, the tendons are to be selected on a random but representative basis.

Anchorage assembly hardware (stressing washers, shims, wedges, and bearing plates) of all tendons selected for inspection are to be examined visually. (During the integrated leak rate test, while the containment is at its maximum test pressure, visual examination of the exterior of the concrete is also performed to detect areas of widespread concrete cracking, spalling, or grease leakage.)

Stress levels of each of the tendons in the sample selected for inspection are monitored by performing lift-off or other equivalent tests. These tests include measurement of the tendon force level by using properly calibrated jacks and the simultaneous measurement of elongations. Acceptance criteria for the results state that the prestress force measured for each tendon should be within the limits predicted for the time of the test.

Previously stressed tendon wires or strands from one tendon of each type are to be removed from the containment for examination over its entire length to determine if there is evidence of corrosion or other deleterious effects. At least three samples are cut from each wire or strand (each end and midlength) and tensile tests conducted. At successive inspections, samples should be selected from different tendons.

A sample of grease from each tendon in the surveillance is to be analyzed (impurities and amounts) and the results are to be compared with the original grease specification. Also, the presence of voids in the grease is to be noted.

Additional information on in-service inspections of ungrouted tendons in prestressed concrete containment structures and containment leak rate testing criteria can be obtained from Refs. 47 and 48, respectively.

5.4 Evaluation of Anchorage Embedments

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

5.5. Recommended Techniques for Concrete Component Inspection

In the previous sections, available techniques for inspection of concrete materials were discussed and their advantages and limitations presented. Generally, evaluation of concrete components will involve a combination of several techniques (i.e., a visual examination followed by coring in areas exhibiting distress). Table 6 presents a summary of nondestructive evaluation techniques that are recommended for investigation of concrete components properties. Once an area of distress is identified, core samples should be obtained to provide quantitative information on the extent of degradation, cause, and need for repair. Recommended and alternate methods for structural integrity monitoring of concrete components are also presented in the table.

Table 6. Recommended nondestructive evaluation methods for inspection of concrete materials

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

[a]Limited to concrete thickness <450 mm.

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

References

1. W. E. Murphy, "The Interpretation of Tests on the Strength of Concrete in Structures," SP-82, Paper 19, pp. 377–92 in *In Situ/ Nondestructive Testing of Concrete*, American Concrete Institute, Detroit, 1984.

2. A. E. Komendant, *Contemporary Concrete Structures*, McGraw-Hill Book Co., New York, 1972.

3. ACI Committee 224, *Control of Cracking in Concrete Structures*, Report 224R-80, American Concrete Institute, Detroit, 1980.

4. L. I. Knab et al., "Laboratory Evaluation of Ultrasonics for Crack Detection in Concrete," *Proc. J. Am. Concr. Inst.* 80(1), 17–27 (January–February 1983).

5. J. R. Leslie and W. J. Cheesman, "An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures," *Proc. J. Am. Concr. Inst.* 46(1), 17–36 (September 1949).

6. V. R. Stirrup et al., "Pulse Velocity as a Measure of Concrete Compressive Strength," SP-82, Paper 11, pp. 201–28 in *In Situ/ Nondestructive Testing of Concrete*, American Concrete Institute, Detroit, 1984.

7. S. A. Robertson et al., "Method of Detecting Construction Faults in Concrete Pressure Vessels," Paper 179/75, pp. 539–44 in *Experience in the Design, Construction, and Operation of Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors*, Institution of Mechanical Engineers, London, Sept. 8–12, 1975.

8. D. M. Alexander, *Development of Procedures for Nondestructive Testing of Concrete Structures: Report 2, Feasibility of Sonic Pulse-Echo Techniques*, Miscellaneous Paper No. C-77-11, U.S. Army Waterways Experiment Station, Vicksburg, Miss., 1980.

9. N. J. Carino, "Laboratory Study of Flaw Detection in Concrete by Pulse-Echo Method," SP-82, Paper 28, pp. 557–80 in *In Situ/ Nondestructive Testing of Concrete*, American Concrete Institute, Detroit, 1984.

10. D. J. Naus, *Applications of Acoustic Emission to Concrete Material Systems and Structures with Reference to Its Applicability to Prestressed Concrete Pressure Vessels (PCRVs)*, ORNL/GCR-81/19, Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., October 1981.

11. P. F. Mlakar et al., "Acoustic Emission Behavior of Concrete," SP-82, Paper 31, pp. 619–38 in *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, Detroit, 1984.

12. D. G. Manning and F. B. Holt, "Detecting Delamination in Concrete Bridge Decks," *Concr. Int.* 2(11), 34–41 (November 1980).

13. J. A. Forrester, "Application of Gamma Radiography to Concrete," *The Eng.* 205 (Feb. 28, 1958).

14. T. R. Cantor, "Review of Penetrating Radar as Applied to Nondestructive Evaluation of Concrete," SP-82, Paper 29, pp. 582–602 in *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, Detroit, 1984.

15. R. D. Browne and R. Blundell, "The Behavior of Concrete in Prestressed Concrete Pressure Vessels," Paper H1/1, *First International Conference on Structural Mechanics in Reactor Technology*, Berlin, Germany, September 1971.

16. *Concrete Core Testing for Strength*, Publication 51.071, Technical Report No. 11, Concrete Society, London, 1976.

17. "Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," ANSI/ASTM C42, Part 14, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 30–33.

18. J. G. L. Munday and R. Dhir, "Assessment of In Situ Concrete Quality by Core Testing," SP-82, Paper 20, pp. 393–410 in *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, Detroit, 1984.

19. J. Bungey, "Determination of Concrete Strength by Using Small Diameter Cores," *Mag. Concr. Res.* 31(107), 91–98 (June 1979).

20. K. Hindo and W. R. Bergstrom, "Statistical Evaluation of In-Place Compressive Strength of Concrete," *Concr. Int.* 7(2), 44–48 (February 1985).

21. "Report of RILEM Working Group on Nondestructive Testing of Concrete," *RILEM Bull.* New Series No. 27 (June 1965).

22. "Standard Test Method for Rebound Number of Hardened Concrete," ASTM C 805, Part 14, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 494–96.

23. "Tentative Test Method for Penetration Resistance of Hardened Concrete," ASTM C 803, Part 14, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 491–93.

24. "Tentative Test Method for Pullout Strength of Hardened Concrete," ASTM C 900, Part 14, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 564–70.

25. C. G. Petersen, "LOK-test and CAPO-test Development and Their Applications," Technical Note 382, *Proc. Institution Civil Engrs.*, Part 1, Vol. 76, May 1984.

26. A. E. Long and A. M. Murray, "The 'Pull-Off' Partially Destructive Test for Concrete," SP-82, Paper 17, pp. 327–50 in *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, Detroit, 1984.

27. A. P. Keiller, "Assessing the Strength of In Situ Concrete," *Concr. Int.* 7(2), 15–21 (February 1985).

28. M. Carlsson et al., "Field Experience in the Use of the 'Break-Off' Tester," SP-82, Paper 14, pp. 277–92 in *In Situ/Nondestructive Testing of Concrete*, American Concrete Institute, Detroit, 1984.

29. "Standard Test Method for Pulse Velocity Through Concrete," ASTM C 597, Part 14, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 353–56.

30. S. Mindess and J. F. Young, *Concrete*, Prentice-Hall, Englewood Cliffs, N.J., 1981.

31. H. W. Chung, "Effect of Embedded Steel Bars Upon Ultrasonic Testing of Concrete," *Mag. Concr. Res.* 30(102), 19–25 (March 1978).

32. K. Mather, "Preservation Technology: Evaluating Concrete in Structures," *Concr. Int.* 7(10), 33–41 (October 1985).

33. "Standard Recommended Practice for Microscopical Determination of Air-Void Content," ASTM C 457, Part 14, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 465–69.

34. "Standard Recommended Practice for Petrographic Examination of Hardened Concrete," ANSI/ASTM C 856, Part 14, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 514–32.

35. "Standard Test Method for Cement Content of Hardened Portland Cement Concrete," ANSI/ASTM C 85, Part 14, *Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 41–44.

36. "Methods of Testing Concrete," BS 1881, Part 6, British Standards Institution, London, 1983.

37. R. J. Currie, "The Implication of Reinforcement Corrosion for Safety and Serviceability of Structures," pp. 11–18 in *Proceedings of a Meeting on Corrosion of Reinforcement in Concrete Construction*, ed. A. P. Crane, Ellis Harwood Ltd. Publishers, Chichester, England, 1983.

38. J. L. Dawson, "Corrosion Monitoring of Steel in Concrete," pp. 175–92 in *Proceedings of a Meeting on Corrosion of Reinforcement in Concrete Construction*, ed. A. P. Crane, Ellis Harwood Ltd. Publishers, Chichester, England, 1983.

39. P. G. Cavalier and P. R. Vassie, *Proc. Institution of Civil Engrs.*, Vol. 70, 461–80, 1981.

40. "Standard Test Method for Half Cell Potentials of Reinforcing Steel in Concrete," ANSI/ASTM C 876, Part 14, *1979 Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1979, pp. 539–43.

41. ACI Committee 222, "Corrosion of Metals in Concrete," ACI 222R-85, pp. 3–32 in *Proc. Am. Concrete Inst.*, V.82, N.1, Detroit (January–February 1985).

42. "Inservice Inspection of Ungrouted Tendons in Prestressed Concrete Containments," Regulatory Guide 1.35 (proposed revision 3), Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., April 1979.

43. "Determining Prestressing Forces for Inspection of Prestressed Concrete Containments," Regulatory Guide 1.35.1 (proposed), Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., April 1979.

44. "Inservice Inspection of Concrete Pressure Components," Subsection IWX-2400 Examination of Prestressed Concrete Containment with Unbonded Tendons, Section XI Working Group, American Society for Mechanical Engineers, New York, 8/81 (draft), *ASME Boiler and Pressure Vessel Code*, Section XI Rules for Inservice Inspection of Nuclear Power Plant Components.

45. J. F. Fulton, "Tendon Surveillance Requirements — Average Tendon Force," *Nucl. Eng. Des.* 72(3), 303–308 (October 1982).

46. "Inservice Inspections of Ungrouted Tendons in Prestressed Concrete Containment Structures," Regulatory Guide 1.35 (rev. 2), Office of Standards Development, U.S. Nuclear Regulatory Commission, Washington, D.C., January 1976.

47. J. R. Dougan, *Evaluation of Inservice Inspections of Greased Prestressing Tendons*, ORNL/TM-8278, Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., September 1982.

48. J. R. Dougan, *Evaluation of Containment Leak Rate Testing Criteria*, NUREG/CR-3549 (ORNL/TM-8909), Union Carbide Corp. Nuclear Div., Oak Ridge Natl. Lab., March 1984.

6. REMEDIAL MEASURES FOR REPAIR OR REPLACEMENT OF DEGRADED CONCRETE COMPONENTS*

Objectives of remedial work are to restore the component's structural integrity, to arrest the mechanism producing distress, and to ensure, as far as possible, that the cause of distress will not recur. 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.[1] With respect to these components, materials for repair, preparation of concrete for repair, and repair techniques† will be discussed. Also, several examples of structural component performance before and after repair will be presented to demonstrate the effectiveness of repair procedures.

6.1 Materials for Repair of Concrete

A wide variety of materials is available for the repair and maintenance of concrete. They range from low-viscosity polymers (epoxies and polyesters) for sealing fine cracks, to very rapid-setting cements (calcium aluminate and regulated set) for repairs in the presence of water, to special concretes (fibrous, latex modified, and polymer) for overlays, to portland cement mortar or concrete. Established remedial measures generally involve the use of one or more of the following materials: epoxy resins, shotcrete, preplaced aggregate concrete, epoxy ceramic foam, replacement mortar or concrete, wedge anchors and additional reinforcement, and miscellaneous sealant materials.[2,3]

6.1.1 Epoxy resins

Epoxy resins have a number of advantageous properties that make them ideally suited for use in the repair of concrete. They provide a wide range of viscosities and cured physical properties, and they provide excellent bond strength, even in the presence of moisture. Limitations of epoxies include thermal expansion and modulus of elasticity properties

---

*Remedial measures pertain primarily to the concrete material systems. Mild steel reinforcement repair will be addressed at appropriate points, but repair of prestressing systems is not addressed because these systems are designed to be inspectable and replaceable.

†Retrofitting to increase the load-carrying capability of a component through strengthening of existing elements or element addition is also considered. Although in the strict sense this is not a repair technique, there are situations where retrofitting may be required either in conjunction with a repair procedure or separately.

significantly different from concrete; susceptibility to creep; and elevated temperature exposure, which can significantly reduce the strength of some formulations. Epoxies used in repair work are generally two-component systems with mixing done at the time of usage. Although their pot life varies with formulation and temperature, their effective usage period normally runs from 5 to 30 min. Problems occurring with the use of epoxies generally result from improper proportioning, contaminated substratum, excessive exotherm, or moisture. American Concrete Institute (ACI) Standards relating to bonding of hardened concrete by using a multicomponent epoxy adhesive include: ACI 503.1-79, *Standard Specification for Bonding Hardened Concrete, Steel, Wood, Brick, and Other Materials to Hardened Concrete With a Multi-Component Epoxy*;[4] ACI 503.2-79, *Standard Specification for Bonding Plastic Concrete to Hardened Concrete with a Multi-Component Epoxy Adhesive*;[5] ACI 503.3-79, *Standard Specification for Producing a Skid-Resistant Surface on Concrete by Use of a Multi-Component Epoxy System*;[6] and 503.4-79, *Standard Specification for Repairing Concrete with Epoxy Mortars*.[7] Additional information also can be obtained from ACI 503R-80, *Use of Epoxy Compounds with Concrete*.[8]

6.1.2 Shotcrete

Shotcreting, or gunniting, is concrete that is applied pneumatically by spraying it from a nozzle by means of compressed air. Application may be by means of either a dry-mix or wet-mix process. The dry-mix process involves premixing the cement and sand* and transferring it to the work site through a hose in a stream of compressed air. The water is injected and mixed with the material as it exists. In the wet-mix process all ingredients are thoroughly mixed, material is introduced into the chamber of the delivery equipment, mix is metered into the delivery hose and conveyed to the nozzle, additional air is injected at the nozzle to increase the velocity, and the material is jetted from the nozzle at high velocity. Because the dry mix can be placed at lower water contents resulting in higher strengths and lower shrinkage, it is most commonly used. A properly installed dry mix will develop good bond strength and can obtain compressive strengths to 55 MPa (a strength of 27.6 MPa is commonly obtained). Advantages of shotcreting are that it is an ideal method for placement of concrete on vertical or steeply sloped surfaces, formwork is not required, and shrinkage is virtually eliminated. Disadvantages are that the quality of the material applied is highly dependent on the nozzleman, and about one-fourth to one-half of the material can rebound on impact. ACI 506-66 *Recommended Practice for Shotcreting*[10] presents additional information on materials, equipment, and applications.

6.1.3 Preplaced aggregate concrete

Preplaced aggregate concrete basically involves packing the forms with a well-graded coarse aggregate and injecting structural mortar or

---

*Concrete for shotcreting is generally made with fairly fine aggregate (<10 mm) and sand, but aggregates up to 20-mm maximum size have been used. Fibrous concrete can also be applied by shotcreting.[9]

grout into the mass to fill the voids. Because the coarse aggregate particles are in intimate contact with one another and are generally present in greater quantities, preplaced aggregate concrete exhibits only about one-half the drying shrinkage of conventional concrete. Also, high bond strengths develop with the existing concrete because of the self-stressing effect of the grout. Utilization of the method, however, requires skill and experience to ensure complete filling of the voids. Additional information on preplaced aggregate concrete is contained in Ref. 11.

6.1.4 Epoxy ceramic foams

Epoxy ceramic foams are a two-component formulation that, when properly mixed, will initiate foam generation within <1 min and expand in volume 7 to 20 times if unrestrained.[2] An advantage of the epoxy ceramic foams relative to conventional expansive resins is that even when completely constrained, the maximum pressure developed is only about 14 kPa. Strengths obtained are a function of the formulation, application procedures, and expansion permitted and generally will range from 0.7 to 34.5 MPa where expansion of <50% is permitted. In addition to the low pressure buildup on expansion, advantages of epoxy ceramic foams include extraordinary bond strength to most materials, stability even at high temperatures, and ability to penetrate crack widths >0.25 mm. Optimal injection of the epoxy ceramic foams, however, requires a high shear mixer and heating.

6.1.5 Replacement mortar or concrete

Portland cement grout or mortar materials are used for localized patching. The material is provided as dry as possible consistent with good compaction or pumping.

Machine-mixed concrete of suitable consistency and proportions is utilized for areas where concrete replacement* is required.[12] To provide compatibility with the substratum concrete, it is best to use materials and mix proportions as close as possible to those used in the original construction.

6.1.6 Wedge anchors and additional reinforcement

Often in the repair or rehabilitation of concrete structures, areas exist where inadequate shear connection between concrete and steel may be

*Improved performance of replacement mortars and grouts can be obtained by replacing up to 33% of the mix water with a latex emulsion (polyvinylidene, styrene-butadiene copolymer, or polyacrylate copolymer). Use of the latex emulsion improves workability; provides increased compressive, flexure, and tensile strengths; provides excellent bond with existing concrete; reduces shrinkage cracking and absorption; and provides increased freeze—thaw resistance.[12] Cost of the latex emulsion system is its major limitation.

present. Wedge-type anchors or grouted anchors may be utilized for this purpose. Pullout and shear resistance data for the various wedge anchor systems is provided by the manufacturer based on static load tests. Where seismic loadings may occur, however, the capacity of these systems needs to be lowered. Results presented in Ref. 13 indicate that the average failure value of anchors under dynamic conditions is ~15% lower than the value for static loadings.

Conventional mild steel and prestressing steel materials can be utilized in the repair of cracked reinforced concrete construction. The reinforcement is used to provide load transfer across a crack. Prestressing steel is utilized where a major portion of the member must be strengthened or where cracks must be closed.

6.1.7 Miscellaneous sealant materials

Information on coating and joint sealant materials is provided in Refs. 14 and 15.

6.2 Preparation of Concrete for Repair

The effectiveness of a repair to concrete is directly related to the care that was exercised in preparing the substratum. Deteriorated or defective concrete must be removed to expose sound concrete by chipping, sawing, drilling, scarifying, planing, or using a water jet. Reinforcing steel that is corroded or has been mechanically damaged should be removed and replaced. After removal of defective materials, the exposed concrete surface should be thoroughly cleaned by flushing with high-pressure water or vacuum cleaning to remove particles or dust. Where formwork is required, the forms should be constructed and installed in accordance with ACI 347-68 *Recommended Practice for Concrete Formwork*.[16]

6.3 Techniques for Repair of Concrete

Selection of the technique for repair of a concrete structure depends to a large degree on the size, depth, and area of repair required. Choice of a repair procedure is also predicated on meeting at least one of the following objectives: restore or increase strength, restore or increase stiffness, improve functional performance, provide watertightness, improve appearance of concrete surface, improve durability, and prevent access of corrosive materials to reinforcement.[3] Types of distress requiring repair that could occur in light-water reactor (LWR) safety-related concrete components include: cracking, spalling or delamination, nonvisible voids, and fracturing or shattering. In addition, situations could occur in which a component could require retrofitting because of either a change in performance requirements or overload.

6.3.1 Crack repair methods

Nine potential methods for the repair of cracks are identified:[3] (1) epoxy injection, (2) routing and sealing, (3) stitching and additional reinforcement, (4) drilling and plugging, (5) flexible sealing, (6) grouting, (7) dry packing, (8) polymer impregnation, and (9) autogenous healing.

6.3.1.1 Epoxy injection. The use of pressure-injected low-viscosity epoxy resin can bond cracks as narrow as 0.025 mm. After cleaning to remove deleterious substances, the cracks are sealed at the surface by using thixotropic epoxy, thermosetting wax, or cementitious materials. Injection of epoxy is performed sequentially through pre-formed plastic injection ports or through unsealed portions of the crack. Either an automated proportioning pump in-head mixing device or batch mixing followed by injection from a pressurized vessel procedure is used. Complete and proper injection of through-cracked members requires sealing and installation of ports on both sides of the member being injected. Appearance of epoxy material at all port locations ensures complete filling of a crack. Epoxy injection is generally limited to cracks with a maximum width of ~6 mm.

6.3.1.2 Routing and sealing. Routing and sealing is used primarily to prevent the entry of hostile environments into dormant cracks. The procedure consists essentially of enlarging the crack along its exposed face by using a concrete saw, hand tools, or a pneumatic tool and sealing with a suitable joint sealant. The technique has application to both fine pattern cracks and larger isolated defects.

6.3.1.3 Stitching and additional reinforcement. Stitching provides a method for reestablishing tensile force transfer across a major crack. Holes are drilled on both sides of the crack and U-shaped metallic units spanning the crack are inserted and grouted. Where crack watertightness is required the crack should be sealed prior to stitching.

An additional technique for reestablishing the integrity of cracked sections is to seal the crack, drill holes at about a 90° angle to the crack plane, fill the hole and crack plane with epoxy by injection at low pressure, and place a rebar into the hole. This technique bonds the crack surfaces together and also provides reinforcement. Where additional strengthening or crack closure is required, prestressing strand or bars can be used to apply a compressive force. The prestressing force is generally applied through additional anchors that must be provided.

6.3.1.4 Drilling and plugging. Drilling and plugging are utilized to repair cracks that run in reasonably straight lines and are accessible only at one end, for example, vertical cracks in retaining walls. The technique involves drilling a hole, centered on and following the crack, of sufficient size to intersect the crack along its full length and to provide room for sufficient material to structurally take the loads. After cleaning and sealing, the hole is filled with a grout material to form a shear key.

6.3.1.5 Flexible sealing. Active cracks can be routed out, cleaned, and filled with a suitable flexible sealant. A bond breaker is provided at the bottom of the slot to allow the sealant to change shape. This repair technique is applicable to areas that are not subject to traffic or mechanical abuse.

6.3.1.6 Grout injection. Wide cracks in mass concrete structures that cannot be repaired by epoxy injection can be repaired by injection of either portland cement grout or a chemical grout. The procedure consists of cleaning the crack surfaces, installing grout nipples, sealing the crack, flushing the crack to clean, checking the seal, and injecting the grout under pressure. Portland cement grout mixtures consist of cement and water or cement plus sand and water (larger cracks). Chemical grouts consist of solutions of two or more chemicals that combine to form a gel, a solid precipitate, or a foam.

6.3.1.7 Dry packing. The dry pack method has a distinct advantage because it does not require special equipment. Dry packing is used for defects that have a high ratio of depth to area and dormant cracks that have been slotted. After cleaning, a low water—cement mortar is placed into the defect and compacted by tamping or rodding. Because the patching material has a low water—cement ratio, its shrinkage is negligible so that the patch remains tight.

6.3.1.8 Polymer impregnation. Monomer systems can also be used for effective repair of cracks. Systems that are used for impregnation contain a catalyst or inhibitor and a monomer or combination of monomers. Polymerization can be effected by catalytic action, irradiation, or heat, with heat being the general method used in conjunction with concrete repair. The technique involves drying the concrete surface, flooding it with monomer, and polymerization in place. Large voids or broken areas of structures in compression zones (beams) can be repaired by first filling with fine and coarse aggregate and then flooding the area with monomer.

6.3.1.9 Autogenous healing. Autogenous healing is a natural process of crack repair that can occur in the presence of nonflowing moisture and absence of tensile stress (inactive crack). Healing occurs through carbonation of the calcium hydroxide in the cement paste by carbon dioxide. The crystals that form interlace and twine, producing a mechanical bonding effect supplemented by chemical bonding between adjacent crystals and between the crystals and the surfaces of the paste and aggregate. The effectiveness of this technique decreases with age of the crack.

6.3.2 Spalling or delamination repair methods

Spalling or delaminated areas are satisfactorily repaired by a concrete overlay provided procedures are taken to provide good bond to the substratum.* The process consists of mechanically removing the damaged or unsound concrete, thoroughly cleaning the surface, permitting the surface to dry, applying a thin grout layer or bonding agent, and applying a high-quality portland-cement-based material or an epoxy-based mortar or concrete. Where the area is relatively deep and shear transfer is required between the substratum and repair material, reinforcement dowels may be grouted into the substratum prior to placement of the repair material. The use of mesh or additional reinforcement may also be desirable.

---

*Techniques for repair of areas exhibiting scaling are similar except the depth of repair is much less.

6.3.3 Nonvisible void repair methods

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

6.3.4 Fractured or shattered concrete repair methods

Where the concrete has been badly fractured or shattered, the defective material (concrete and possibly rebars) must be removed and replaced. Either (1) machine-mixed concrete of suitable consistency and proportions to become integral with the base concrete, (2) shotcrete, or (3) preplaced aggregate materials may be utilized to effect the repair. Type K shrinkage-compensating cement also is frequently used. Supplemental reinforcement and dowels are used to make the repair self-sustaining and to anchor it to the sound concrete.

6.3.5 Retrofitting (strengthening) methods

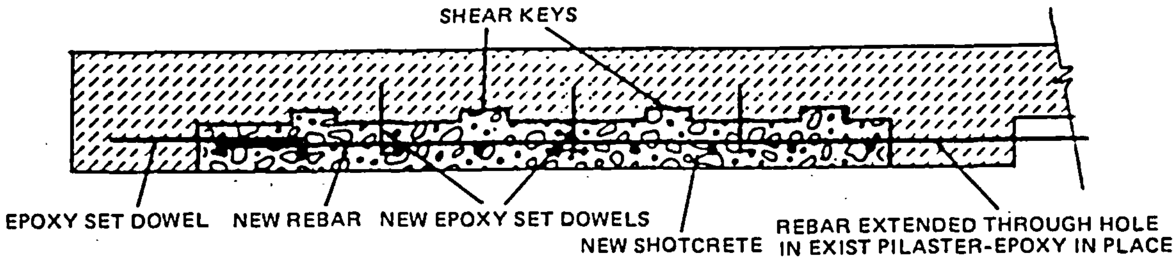
Existing structural components can become inadequate due to either a change in performance requirements or occurrence of an overload condition (intense seismic event). Under these conditions retrofitting may be required to reestablish serviceability. As noted in Ref. 2, this can be accomplished by either strengthening of existing elements, addition of new force-resisting elements, a combination of element strengthening and addition, or use of supplemental connecting devices.

Strengthening of existing elements can be accomplished by increasing their shear resistance or cohesiveness by grout or adhesive injection, encasement, or addition of section. In cases using encasement or section addition, careful attention must be paid to providing shear transfer and bond development. This can be effected through roughening the host concrete surface and the use of shear transfer devices such as grouted dowels or wedge anchors. Figures 44–47 present examples of methods utilized for strengthening existing shear walls, columns, beams, and foundations.[2]

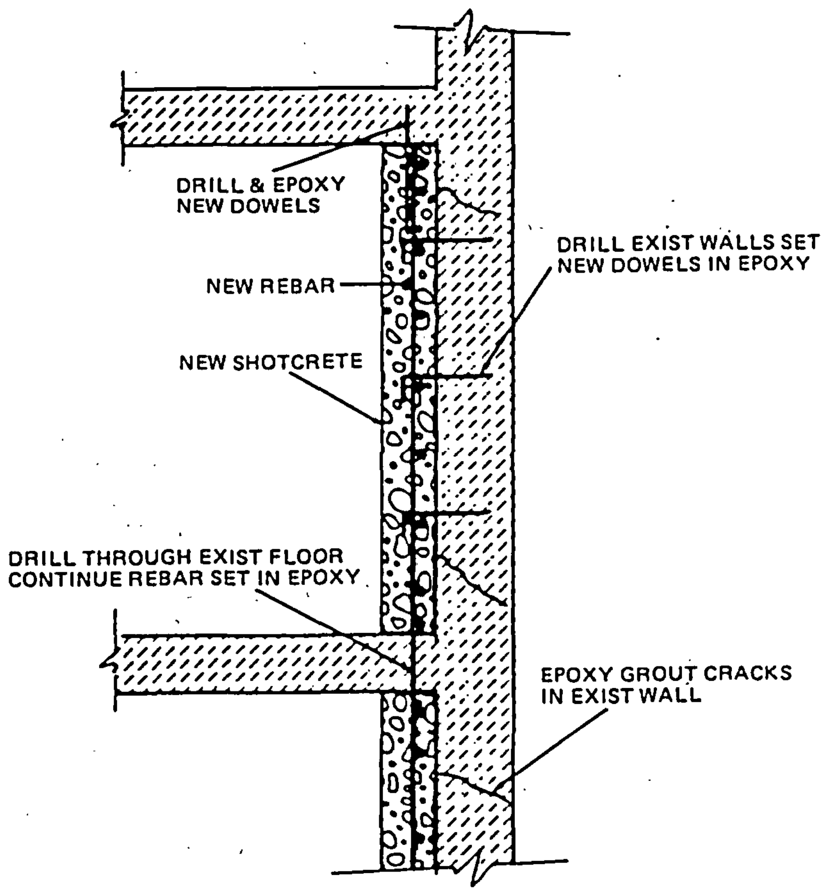
New elements or the replacement of existing elements also can provide component structural strengthening. Techniques that can be used include replacement of nonstructural building interior walls with walls designed to impart shear resistance, addition of new floor and roof diaphragms, or foundation augmentation (new elements, additional piles). Figure 46 presents an example where a new collector member was added.

Continuity or fixity of nonstructural elements can be provided by direct bolting or placement of supplemental steel straps that are bolted in place. Parapets, towers, overhanging cornices, or support fixtures also can be braced to restore structural integrity by use of structural

ORNL–DWG 86–4584 ETD



PLAN



SECTION

Fig. 44. Typical shear wall strengthening. *Source*: J. Warner, "Methods for Repairing and Retrofitting (Strengthening) Existing Buildings," Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11–15, 1977.

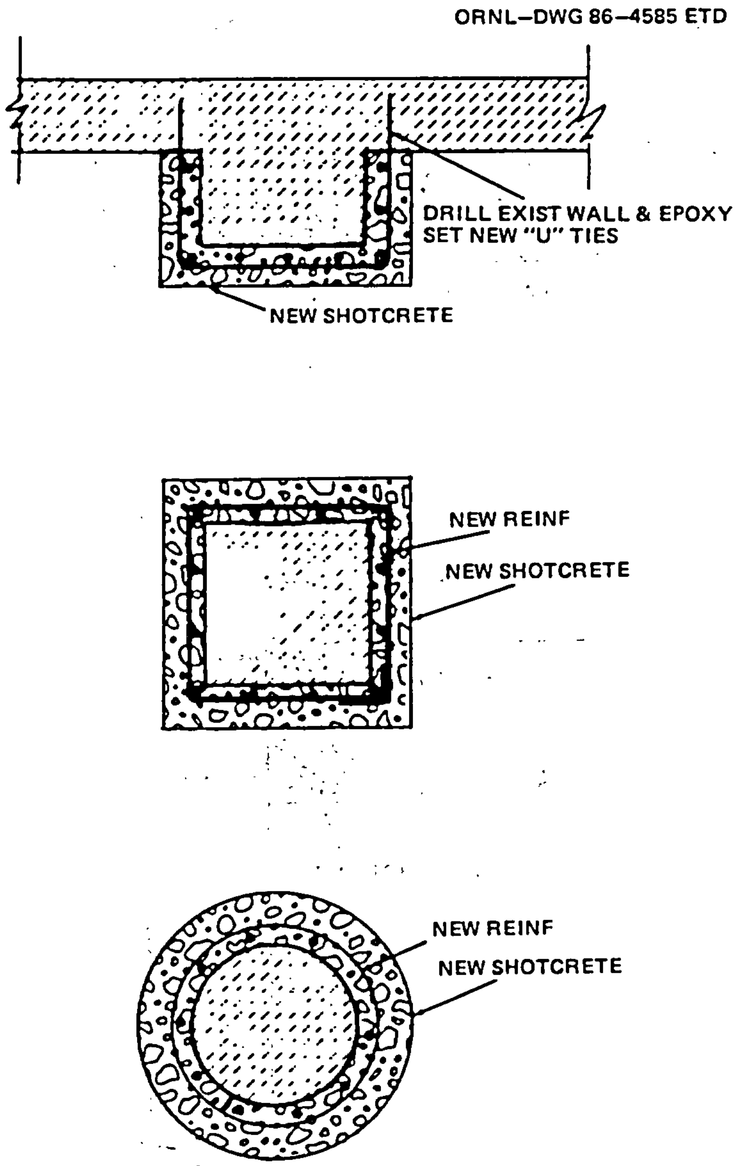


Fig. 45. Typical methods for column strengthening. *Source:* J. Warner, "Methods for Repairing and Retrofitting (Strengthening) Existing Buildings," Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11–15, 1977.

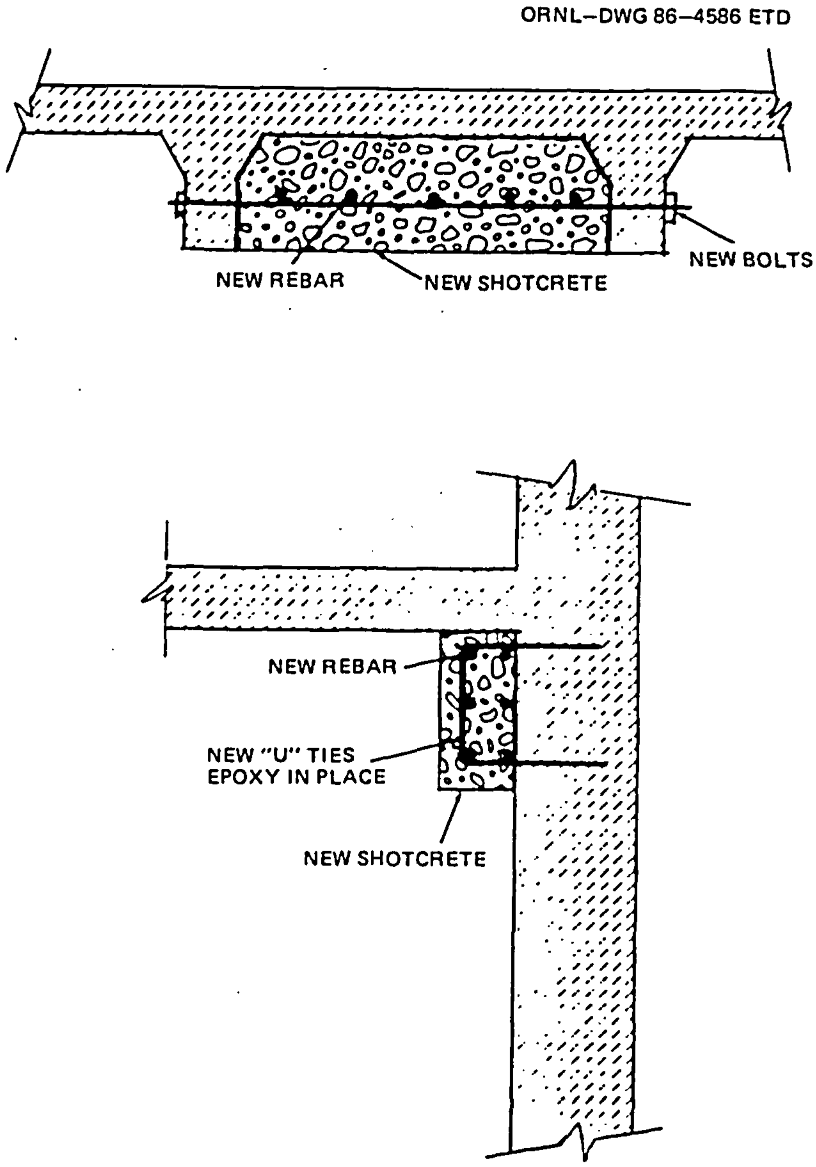


Fig. 46. Typical methods for strengthening beams and new collector members. *Source*: J. Warner, "Methods for Repairing and Retrofitting (Strengthening) Existing Buildings," Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11–15, 1977.

ORNL-DWG 86-4587 ETD

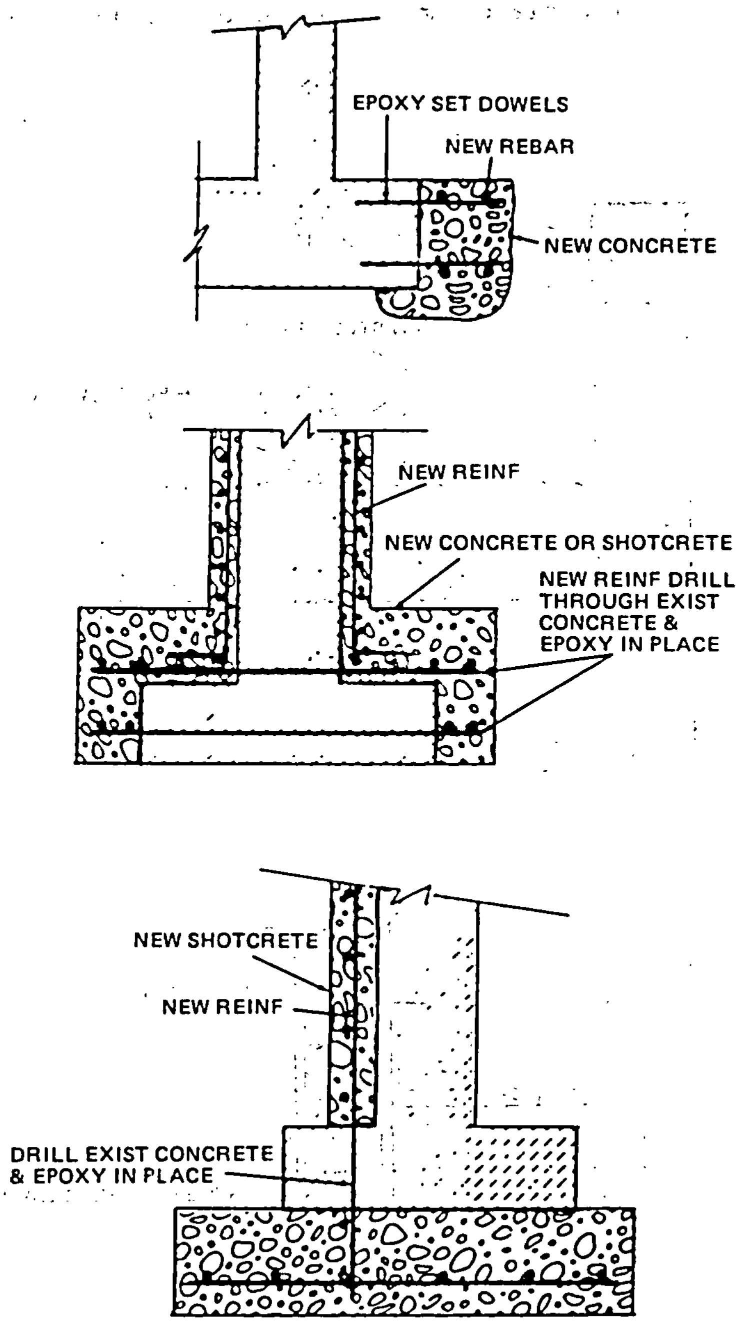


Fig. 47. Typical foundation augmentation. *Source*: J. Warner, "Methods for Repairing and Retrofitting (Strengthening) Existing Buildings," Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11–15, 1977.

steel members that are either bolted in place, secured by embedment anchor systems, or embedded in replacement mortar or a polymer-based material. Figure 48 presents an example of an anchorage for parapets and cornices.[2]

ORNL-DWG 86-4588 ETD

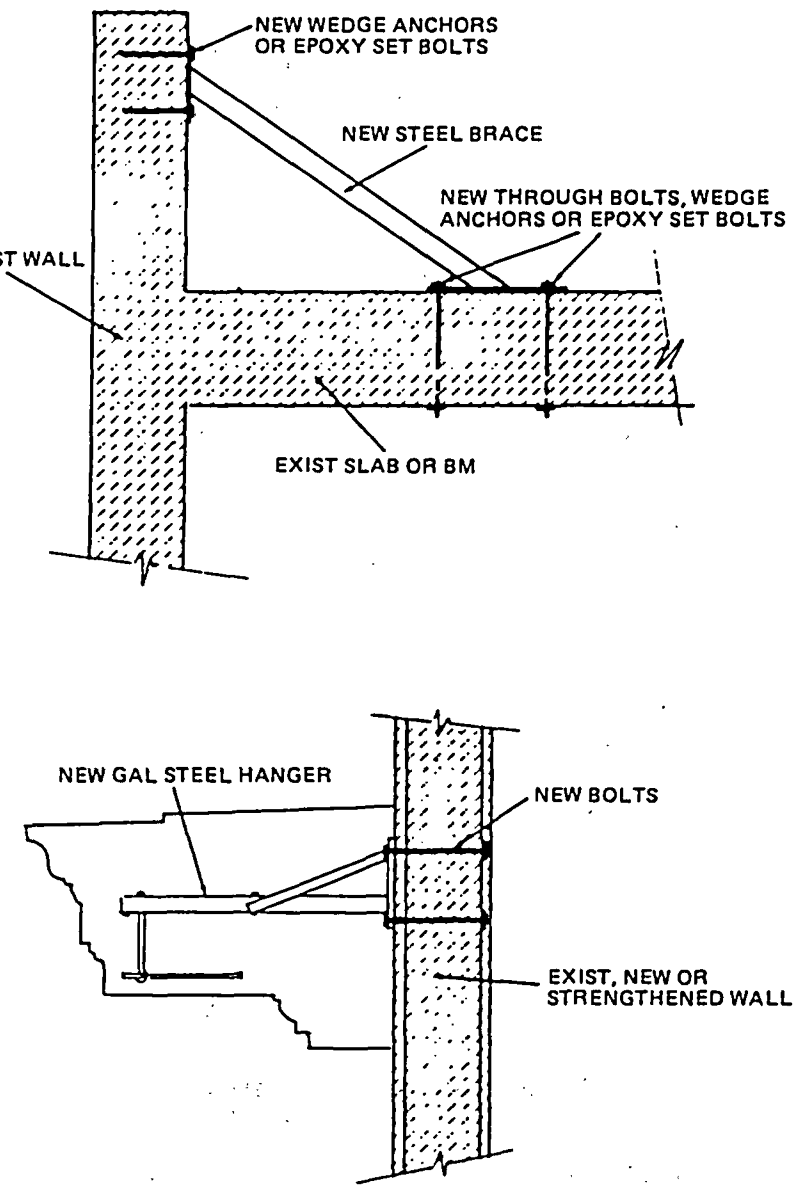


Fig. 48. Typical anchorage for parapets and cornices. *Source:* J. Warner, "Methods for Repairing and Retrofitting (Strengthening) Existing Buildings," Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11–15, 1977.

6.4 Effectiveness of Repairs to Concrete Structural Components

An indication of the effectiveness of techniques used in the repair of concrete structural components can be provided by examining the performance of several components before and after repair. Pertinent examples from the literature that compare prerepair and postrepair performance include: (1) concrete-rebar bond, (2) reinforced concrete beams statically and cyclically loaded, (3) concrete joints under static and dynamic loading, (4) concrete shear walls under fire exposure, and (5) earthquake-resistant structural wall.

6.4.1 Concrete-rebar bond

Tests were conducted on pullout specimens and reinforced concrete beams (shown schematically in Fig. 49) to investigate the effectiveness of epoxy injection in repairing the bond between steel and concrete in reinforced concrete members.[17] The pullout specimens consisted of 150-mm concrete cubes containing a single Grade 60 deformed bar having a diameter of either 12, 16, or 20 mm. The specimens were tested by applying a tensile force to the bar while the concrete cube was restrained and measuring slip of the bar at the free end. After testing, the perimeter of each concrete crack was sealed with a rapid-setting epoxy adhesive, a structural adhesive was pressure injected into the cracks, the specimen was permitted to cure under ambient conditions for about 7 d, and the specimen was then retested. To supplement the pullout data, two reinforced concrete beams 200 mm × 300 mm × 2 m long and containing a single Grade 60 deformed bar of 25-mm diameter were loaded in flexure. To determine steel strains during the test, strain gages were attached at two

ORNL–DWG 86–4589 ETD

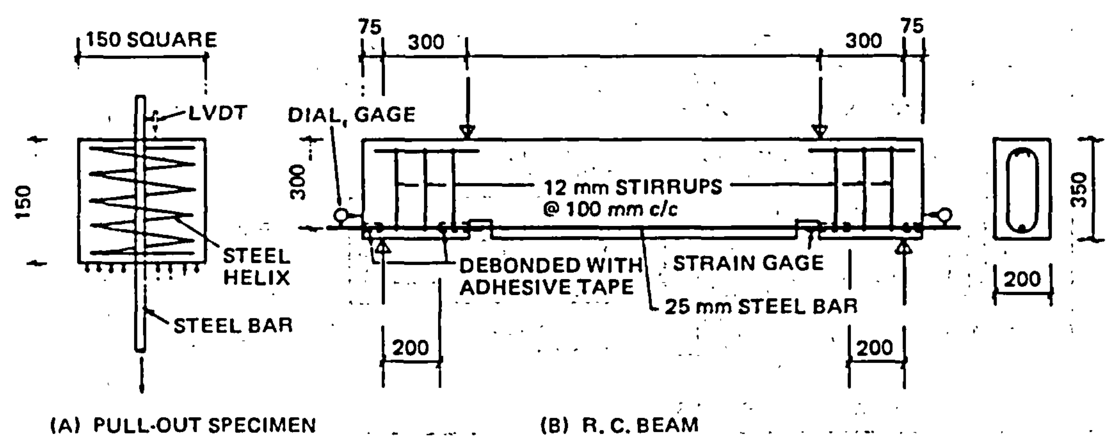


Fig. 49. Details of pull-out and reinforced concrete beam test specimens. *Source:* H. W. Chung, "Epoxy Repair of Bond in Reinforced Concrete Members," *Proc. J. American Concrete Institute* 78(1), January–February 1981.

notches located in the beam ~300 mm from each end. Also part of the bar between the notch and end of the beam was debonded to ensure that a bond failure occurred. After beam failure in shear with substantial rebar slip, it was repaired using the same procedure as for the pullout specimens. Although the damaged surfaces of rebar embedment were not totally penetrated by the epoxy, the bond strength of the repaired concrete was not less than the original concrete, and the repaired concrete could resist the same bond stress with less slip than experienced by the original concrete.

6.4.2 Reinforced concrete beams

Six reinforced concrete beams (Fig. 50) containing a large rectangular opening were loaded eccentrically.[18] The size of the rectangular opening differed either in length or depth for each beam. At failure numerous cracks developed, and concrete crushing occurred at all four corners of the opening. The beams were repaired by restoring their shape (straightness), removing all loose concrete, replacing the crushed concrete with epoxy mortar, sealing exposed cracks, and pressure-injecting a mixture of low-viscosity resin/hardener through nipples that had been attached during crack sealing. The repaired beams were then retested in the same manner as the original beams. Results showed that all repaired cracks did not reopen on reloading (repaired sections were stronger than adjacent concrete), crack widths in repaired beams were generally smaller

ORNL–DWG 86–4590 ETD

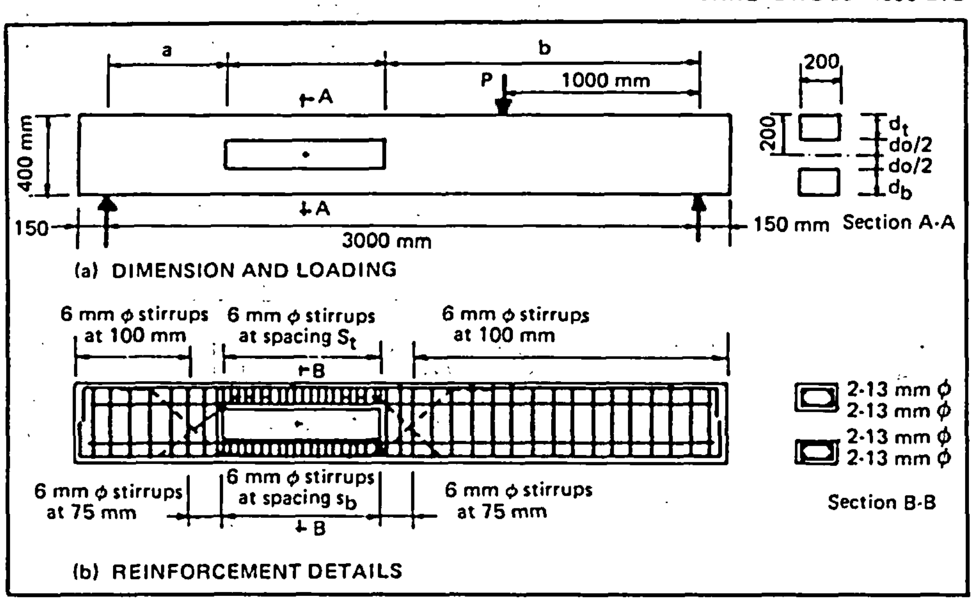


Fig. 50. Dimensions, loadings, and reinforcement details of reinforced concrete beams containing a large rectangular opening. *Source*: M. A. Mansur and K. C. G. Ong, "Epoxy-Repaired Beams," *Concr. Int'l.* 7(10), American Concrete Institute, Detroit, October 1985.

than those originally occurring, repaired beams exhibited reduced stiffness because of the presence of hairline cracks that could not be injected, and the repaired beams were stronger than the original beams.

Beams (shown schematically in Fig. 51) were designed to fail either in tension or shear to establish the repair capability and consequences of synthetic resin injection.[19] Unidirectional and reversed (cyclic) loading sequences were applied to the beams. The cyclic loading was applied to establish the effect of an interposed resin layer within cracked concrete, resulting from wedging and possible resin fatigue. Prior to retesting, the cracks were repaired by sealing at the concrete surface and injecting a synthetic resin (epoxide). Conclusions from the investigation were that badly cracked reinforced concrete beams can be reinstated to load deflection and ultimate load behavior at least as good as that for an unfailed beam; the repair technique has limitations if the crack widths are either too great (repair will not hold) or too small (<0.1 mm); and for the cyclic tests there were no signs, from the performance of either the resin or the shear-cracked beams, that the repair worsened the situation by creating new cracks as a result of wedging.

6.4.3 Concrete joints

Shear tests were conducted on concrete pushoff specimens that were 125 × 200 × 660 mm in length.[20] As shown in Fig. 52, each specimen was composed of two parts: a precast part of 38 MPa and a cast-in-place part of 32-MPa concrete. The interface between the parts was a rough surface produced by exposing the coarse aggregate on the precast part to flowing water before the concrete had set. Specimens either had no reinforcement across the joint or two 5-mm-diam mild steel stirrups were provided across the joint. The specimens were first loaded axially to produce a shearing effect along the plane of the joint with slip along the interface monitored. After failure, the specimens were repaired by clamping the detached parts tightly together, sealing the perimeter of the joint with rapid-setting adhesive leaving holes for epoxy injection and air relief, and pressure-injecting epoxy into the crack. After curing 3 or 4 d under ambient conditions, the specimen was reloaded to failure. Investigation results showed that the shear resistance of the repaired joint was at least equivalent to that of the original joint, deformation capability of original and repaired joints were equivalent, and shear stresses up to as high as 5 MPa could be tolerated by the repaired joint (failure may occur in the adjacent concrete, however).

Dynamic shear tests were also conducted using the concrete pushoff specimen shown in Fig. 52.[21] Specimens were fabricated, tested, and repaired using the same procedures as described in the previous paragraph, except the specimens were loaded by axial impact using a specially built jack operated by compressed air rather than loaded statically. Load cells placed on the top and bottom of the specimen were used to measure impact loads that produced a rate of stressing of ~12,500 MPa/s. The dynamic tests lead to the following conclusions: dynamic shear strength of repaired joint was at least equivalent to that of the original joint; the repaired joint can absorb the same amount of impulse as the original joint; and, provided the repair is properly done, the repaired joint is stronger in shear than the adjacent concrete.

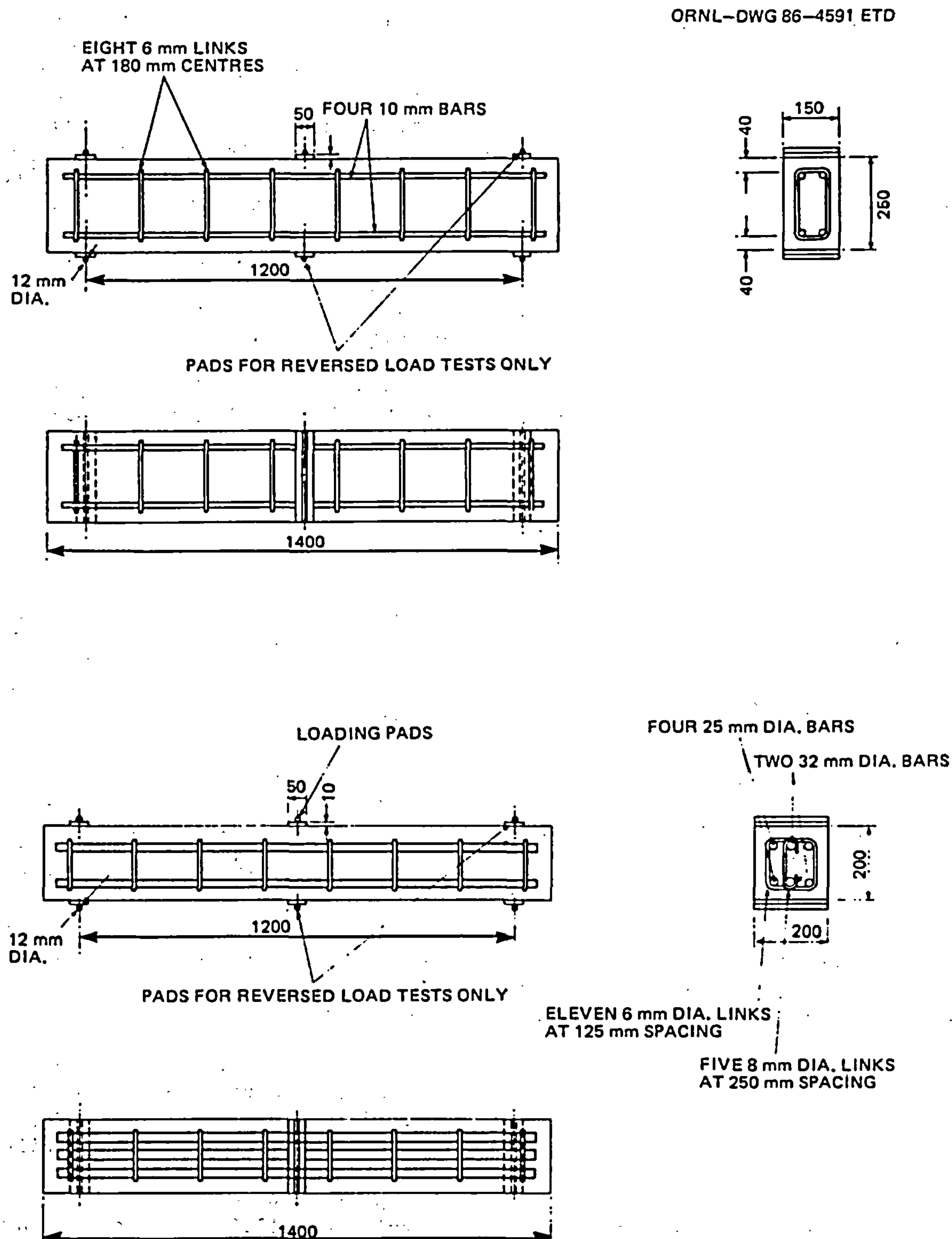


Fig. 51. Schematics of tension and shear crack-type reinforced concrete beam test specimens. *Source:* P. C. Hewlett and J. G. D. Morgan, "Static and Cyclic Response Reinforced Concrete Beams Repaired by Resin Injection," *Mag. Concr. Res.* 34(118) (May 1982).

ORNL–DWG 86–4592 ETD

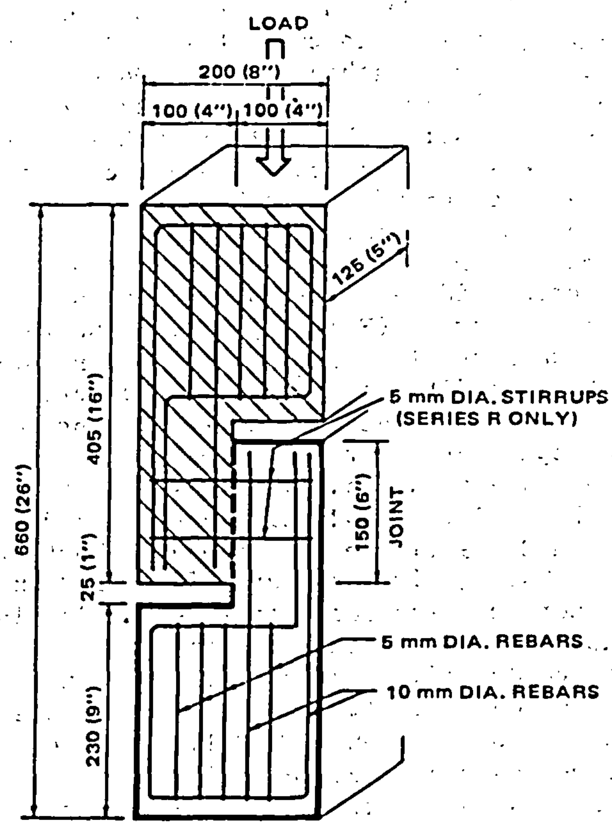


Fig. 52. Details of concrete pushoff test specimens. *Source*: H. W. Chung and L. M. Lui, "Epoxy-Repaired Concrete Joints," *Proc. J. Am. Concr. Inst.* 74(6) (June 1977).

6.4.4 Concrete walls under fire exposure

Results presented in the previous paragraphs indicate that when a structural component is repaired properly with an epoxy-based system, it will exhibit equivalent or superior performance characteristics relative to the original structure. Extremely low-probability environments (loss-of-coolant accident), however, could occur in an LWR plant in which a concrete component repaired by epoxy injection may be required to meet its functional and performance requirements under less than ideal conditions, for example, elevated temperature. Because epoxies, like most materials, exhibit strength reductions on elevated temperature exposure, the performance of epoxy-repaired structural components under these conditions needs to be established. Some insight into this problem can be found in Ref. 22, which presents results of the effects of elevated temperature exposure on "basic" structural epoxy systems and the behavior of epoxy-repaired concrete shear walls during "pseudo-fire" exposures.

Pure epoxy adhesive specimens 12.7-mm diam by 25.4 mm long were placed into a preheated electric oven for a period of 1 h at the specified temperature and then tested in compression immediately upon removal from the oven (hot strengths). Companion tests were also conducted in which the specimens were permitted to cool at room temperature for about 7 d before testing (residual strengths). Above 204°C the epoxy hot strength was found to be negligible due to cracking and the rubberlike specimen behavior. Residual strengths up to ~149°C exposure were reduced <25%, but beyond 204°C the specimens cracked and became rubberlike, exhibiting a strength reduction (~40% strength reduction at 204°C).

Small- (356- by 457-mm), intermediate- (864- by 1016-mm), and large-scale (2286- by 2591-mm) prismatic specimens of varying wall thickness (152.4 to 254 mm) and crack widths (1.27 to 6.35 mm) were fabricated using 28.6-MPa ready-mix concrete (Fig. 53). Crack surfaces were simulated by breaking each wall specimen as a beam. After curing for 90 d under standard laboratory conditions, the specimens were epoxy injected to reestablish integrity. Six structural epoxies, representing materials that had been used to repair structures damaged by the San Fernando earthquake, were used in the investigation. The epoxy-repaired shear wall specimens were then subjected to pseudo-fire exposures designed to simulate a 2-h duration ASTM E-119 fire exposure and a short-duration high-intensity (SDHI) fire. During fire exposure (face ABCD in Fig. 53), the small-scale specimens were not subjected to external loadings, but upon completion of fire exposure, hot (within 10 min) and residual strength compression tests were conducted. The intermediate- and large-scale test specimens were nominally loaded to 1.51 and 0.8 MPa, respectively, during and after fire exposure. For the 2-h ASTM E-119 and the 1-h SDHI fires, the properties of epoxy-repaired concrete walls 152 to 254 mm in thickness were reduced to levels below original design stress levels. Residual strength properties of most of the epoxy materials subjected to

ORNL-DWG 86-4593 ETD

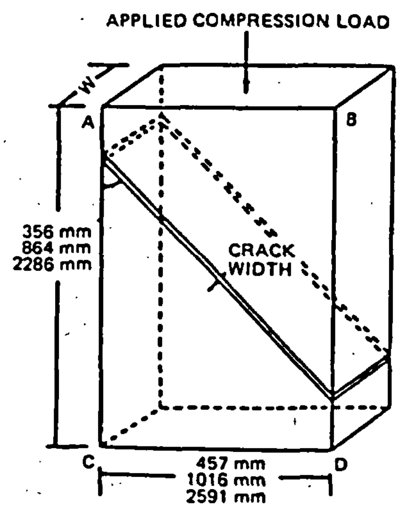


Fig. 53. General configuration of small-scale expoxy-repaired shear wall specimens. *Source*: J. M. Plecnik et al., "Epoxy-Repaired Concrete Walls Under Fire Exposure," *ASCE J. Str. Div.* 108(STP8) (August 1982).

elevated temperatures were increased more than 50% as a result of post-curing. The duration and intensity of fire exposure were found to have great significance on strength and behavior of epoxy-repaired concrete walls both during and after fire exposure; that is, compressive strength properties after SDHI fire exposure were about two times greater than for the ASTM E-119 fire. Also, the orientation of the epoxy-repaired crack in relation to applied stress was found to have a significant effect on the strength properties of epoxy-repaired components during fire exposure with cracks subjected to parallel shear stresses exhibiting lowest strengths.

6.4.5 Earthquake-resistant structural wall

One-third-scale specimens, representing five-story walls, were loaded laterally through the top slab until web damage occurred (Fig. 54).[23] Of

ORNL–DWG 86–4594 ETD

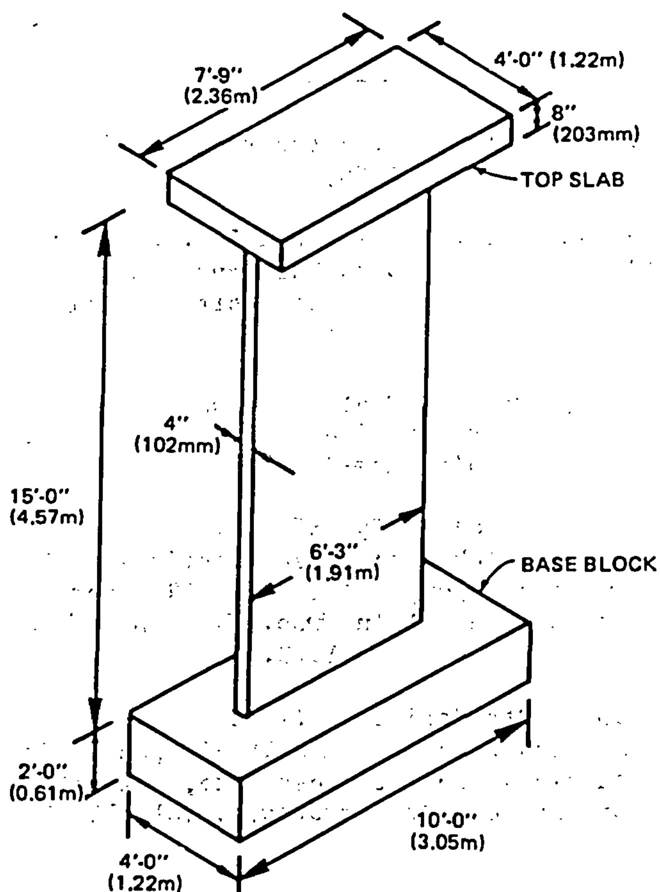


Fig. 54. Nominal dimensions of one-third-scale five-story wall test specimens. *Source*: A. E. Fiorato et al., "Behavior of Earthquake Resistant Structural Walls Before and After Repair," *J. American Concrete Institute* 80(5) (September–October 1983).

the three specimens tested, only one was loaded axially during testing. A different repair procedure was investigated for each wall: damaged web was replaced to its original thickness with new concrete, web thickness was increased as part of the repair, and supplementary reinforcement was added to the web (diagonal bars) prior to replacement of web concrete to its original thickness. The specimens were then retested and results compared with initial wall performance. Conclusions from the results presented were that replacement of damaged concrete in webs of structural walls is an effective and simple repair procedure that yields strength and deformation capacities equivalent to the original walls; initial stiffnesses of repaired walls were ~50% those of original walls (important for dynamic loadings); for the specimen repaired with a thickened web, deformation capacity of the wall was increased, nominal shear stresses at equivalent loads were reduced, and the capacity of diagonal compression struts that form under lateral load reversals were increased; and addition of diagonal reinforcement within the hinging region of the base of the repaired wall reduced shear distortions and increased deformation capacity.

References

1. J. J. Waddell, "Basic Steps of a Concrete Repair Program," *Concr. Int.* 2(9), 30–33 (September 1980).

2. J. Warner, "Methods for Repairing and Retrofitting (Strengthening) Existing Buildings," Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11–15, 1977.

3. ACI Committee 224, "Causes, Evaluation and Repair of Cracks in Concrete Structures," Report No. 224.R-84, pp. 211–30 in *Proc. J. Am. Conc. Inst.* 81(3), Detroit, May–June 1984.

4. ACI Committee 503, "Standard Specification for Bonding Hardened Concrete, Steel, Wood, Brick, and Other Materials to Hardened Concrete with a Multi-Component Epoxy," ACI 503.1-79, American Concrete Institute, Detroit, pp. 437–41 in *Proc. J. of American Concrete Institute*, Vol. 75, No. 9, September 1978.

5. ACI Committee 503, "Standard Specification for Bonding Plastic Concrete to Hardened Concrete with a Multi-Component Epoxy Adhesive," ACI 503.2-79, American Concrete Institute, Detroit, pp. 442–47 in *Proc. J. of American Concrete Institute*, Vol. 75, No. 9, September 1978.

6. ACI Committee 503, "Standard Specification for Producing a Skid-Resistant Surface on Concrete by Use of a Multi-Component Epoxy System," ACI 503.3-79, American Concrete Institute, Detroit, pp. 448–53 in *Proc. J. of American Concrete Institute*, Vol. 75, No. 9, September 1978.

7. ACI Committee 503, "Standard Specification for Repairing Concrete with Epoxy Mortars," ACI 503.4-79, American Concrete Institute, Detroit, pp. 454–59 in *Proc. J. of American Concrete Institute*, Vol. 75, No. 9, September 1978.

8. ACI Committee 503, "Use of Epoxy Compounds with Concrete," ACI 503R-80, American Concrete Institute, Detroit, Part 5, Masonry, Precast Concrete, Special Processes, *ACI Manual of Concrete Practice*, 1985.

9. S. Mindess and J. F. Young, *Concrete*, Prentice-Hall, Englewood Cliffs, N.J., 1981.

10. ACI Committee 506, "Recommended Practice for Shotcreting," ACI 506-66, American Concrete Institute, Detroit, Part 3, Products and Processes, *ACI Manual of Concrete Practice*, 1977.

11. ACI Committee 304, "Preplaced Aggregate Concrete for Structural and Mass Concrete," pp. 785–97 in *Proc. J. of American Concrete Institute*, Vol. 66, No. 10, Detroit, October 1969.

12. P. C. Robery, "Structural Repairs," *Concrete* 17(2) (February 1983).

13. *Static and Dynamic Loadings of 5/8 Inch Concrete Anchors*, Report 7745.10-72, Department of Engineering Research, Pacific Gas and Electric Co., San Francisco, Aug. 10, 1972.

14. ACI Committee 515, *Guide for the Protection of Concrete Against Chemical Attack by Means of Chemical Coatings and Other Corrosion-Resistant Materials, Part 3, Products and Processes, ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, 1977.

15. ACI Committee 504, "Guide to Joint Sealants for Concrete Structures," *Part 3, Products and Processes, ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, 1977.

16. ACI Committee 347, "Recommended Practice for Concrete Formwork," ACI 347-68, American Concrete Institute, Detroit, 1968, Part 1 Materials and Properties of Concrete, Construction Practices and Inspection, Pavements and Slabs, *ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, 1976.

17. H. W. Chung, "Epoxy Repair of Bond in Reinforced Concrete Members," pp. 79–82 in *Proc. J. Am. Concr. Inst.* 78(1) (January–February 1981).

18. M. A. Mansur and K. C. G. Ong, "Epoxy-Repaired Beams," *Concr. Int.* 7(10), 46–50 (October 1985).

19. P. C. Hewlett and J. G. D. Morgan, "Static and Cyclic Response of Reinforced Concrete Beams Repaired by Resin Injection," *Mag. Concr. Res.* 34(118), 5–17 (March 1982).

20. H. W. Chung and L. M. Lui, "Epoxy-Repaired Concrete Joints," *Proc. J. Am. Concr. Inst.* 74(6), 264–67 (June 1977).

21. H. W. Chung and L. M. Lui, "Epoxy-Repaired Concrete Joints Under Dynamic Loads," *Proc. J. Am. Concr. Inst.* 75(7), 313–16 (July 1978).

22. J. M. Plecnik et al., "Epoxy-Repaired Concrete Walls Under Fire Exposure," *ASCE J. Str. Div.* 108(STP8), 1894–1908 (August 1982).

23. A. E. Fiorato et al., "Behavior of Earthquake Resistant Structural Walls Before and After Repair," *J. Am. Concr. Inst.* 80(5), 403–13 (September–October 1983).

7. CONSIDERATIONS FOR DEVELOPMENT OF A DAMAGE METHODOLOGY TO ASSESS DURABILITY FACTOR DETERIORATION RATES AND TO PREDICT STRUCTURAL RELIABILITY

Information previously presented indicates that the performance of concrete components in both nuclear and non-nuclear applications has been very good. Where the concrete in these components has been fabricated with close attention to the factors related to production of good concrete (Fig. 21), the concrete will exhibit infinite durability; however, where there has been a breakdown in one of these factors or the component was subjected to an extreme environmental stressor, distress can occur. Review of the various nondestructive and destructive techniques for identifying and indicating the magnitude of distress in concrete has shown these techniques to be capable of locating regions subjected to deteriorating influences. Also, remedial measures that can lead to successful repair and replacement of concrete have been shown to be available, provided a procedure such as that shown schematically in Fig. 55 is followed. Where the system breaks down, however, is that a damage methodology to provide a quantitative measure of the ability of a structure to meet potential future performance requirements [e.g., loss-of-coolant accident (LOCA)] does not presently exist. Three areas, however, that would provide significant input toward quantifying the ability of a light-water reactor (LWR) safety-related concrete component to meet its functional and performance requirements at some future time, based on its performance history or present status, can be addressed: (1) development of a representative material property data base, (2) establishment and evaluation of an accelerated aging methodology for concrete materials;

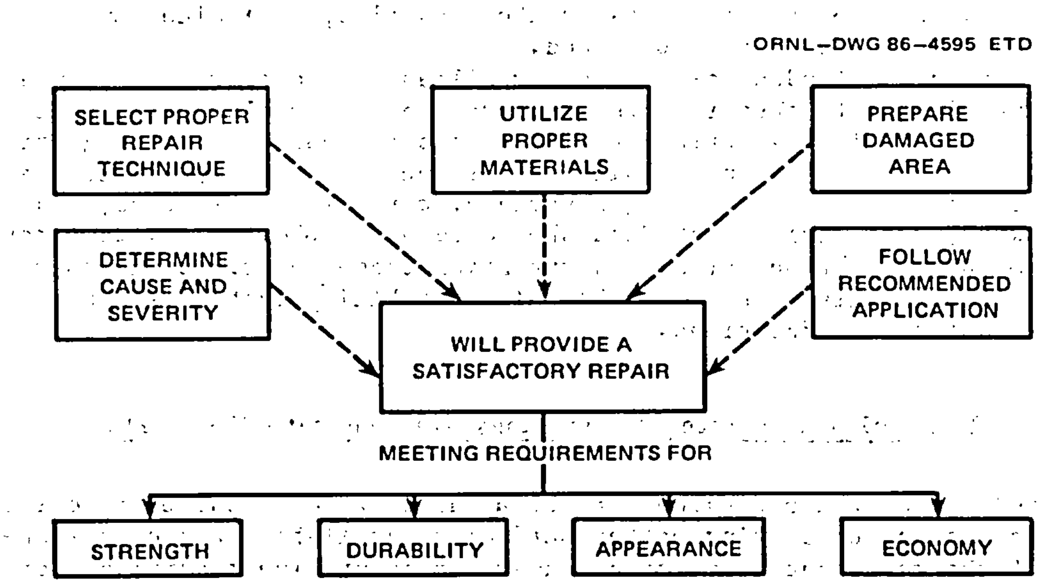


Fig. 55. Factors influencing the successful repair of a concrete component. *Source:* Modification of J. J. Waddell, "Basic Steps of a Concrete Repair Program," *Concr. Int'l.* 2(9), American Concrete Institute, Detroit, September 1980.

and (3) formulation of a methodology to provide a quantitative measure of structural reliability and of residual life.

7.1 Development of Representative Material Property Data Base

Overall performance of a structure is largely dependent on (1) the quality of the materials used, which, in turn, is affected by the standard of workmanship, and (2) for concrete, a structure's function, position, and the environmental stressors to which it may be subjected. Under normal operating conditions a high level of confidence can be placed in traditional material performance based on past experience. However, for concrete material systems used in LWR applications where operating conditions are not necessarily considered normal because of potential elevated temperature and irradiation exposure over a protracted time, the confidence level will not be as high as for the more traditional applications. This is not the result of obvious deteriorating influences operating on these structures, but rather from the lack of a historical material property data base that can be used to form the basis for life extension considerations.

Three plants that are currently shut down (Dresden 1, Humbolt Bay, and Shippingport),* however, provide an opportunity for making major contributions to the material property data base relative to aging effects. Baseline information on the concrete materials and control specimen strength results should be available as part of the quality assurance (QA) documentation. By obtaining concrete core samples at pertinent locations in one or more of these plants and conducting petrographic examinations and load-to-failure tests on these samples, an indication of the significance of aging can be obtained.

Prestressing tendon in-service surveillance reports and containment integrated leak-rate test reports also provide a valuable data source. Results obtained from scrutinizing these reports would provide significant information useful in trending performance of not only the concrete materials, but also prestressing materials, corrosion inhibitors, seals and gaskets, etc. Sufficient data should be available to provide information on deterioration parameters affecting these materials and to provide at least a first cut at establishing durability factors for use in life extension considerations.

7.2 Accelerated Aging Methodology for Concrete Materials

Prediction of the service life of a building component or material is dependent on there being either sufficient available data on performance of the component or material under representative conditions for

---

*Dresden 1 = 200 MW(e) boiling-water reactor (BWR).
Humbolt Bay = 63 MW(e) BWR.
Shippingport = 72 MW(e) pressurized-water reactor—light-water breeder reactor (PWR-LWBR).

the time period of interest or accelerated testing methods that can be used with confidence to develop the required data within a reasonable time. Although, as noted in Sect. 7.1, valuable data on aging effects can be obtained from plants that have been shut down, the data in all likelihood will be somewhat plant specific and probably will not be representative for either all safety-related concrete components or potential environmental stressors. A possible alternative approach that can be used to develop the required data base is to use accelerated aging test techniques. Either deterministic or probabilistic analyses can then be applied to the data to predict service life.

Accelerated aging tests have been used for many materials, such as insulation, paints, glasses, polymers, etc., to predict useful remaining life or to aid in predicting service life. To a limiting degree, tests of this type have also been applied to predict, at an early age, the 28-d strength of concrete (accelerated strength testing),[1] to predict potential concrete strength at any age,[2] to predict long-term service life of concretes in a sulfate environment,[3] and to evaluate resistance of concrete to freezing and thawing.[4] The analytical-experimental program proposed would be based mostly on the American Society for Testing and Materials (ASTM) practice for developing accelerated aging tests to aid in the prediction of building component service life (shown schematically in Fig. 56, Ref. 5). The program would involve three major phases: (1) problem definition (characterization of material or component, identification of pertinent degradation factors and their method(s) of simulation, and definition of test performance requirements), (2) design and performance of predictive service life tests (experimental studies in which pertinent degradation factors would be simulated at an accelerated rate and predictive service life tests would be compared to long-term tests under service conditions), and (3) mathematical model development (compare rates of change in predictive service life tests with those from in-service tests; Table 7 presents several mathematical models used in aging studies). Table 8 presents the major steps and systematic diagrams similar to those that would be used for analysis of data obtained from a prototype accelerated life test.[13]

Results obtained from this study will aid in describing and understanding the phenomena of potential deterioration with the passage of time, assist in determining the residual service life of materials and components in conjunction with actual degradation condition, and help in establishing maintenance or remedial measure programs that will assist in either prolonging a component's service life or improving the probability of the components surviving an extreme event, such as a LOCA.

7.3 Methodology to Provide a Quantitative Measure of Structural Reliability

Assessment of the functional and performance characteristics of concrete components is an important consideration in the extension of the operating life of nuclear facilities. Given the complex nature of the various environmental stressors that can exert deteriorating influences on the concrete components, a systems approach is probably best in addressing the evaluation of a structure for life extension considerations.

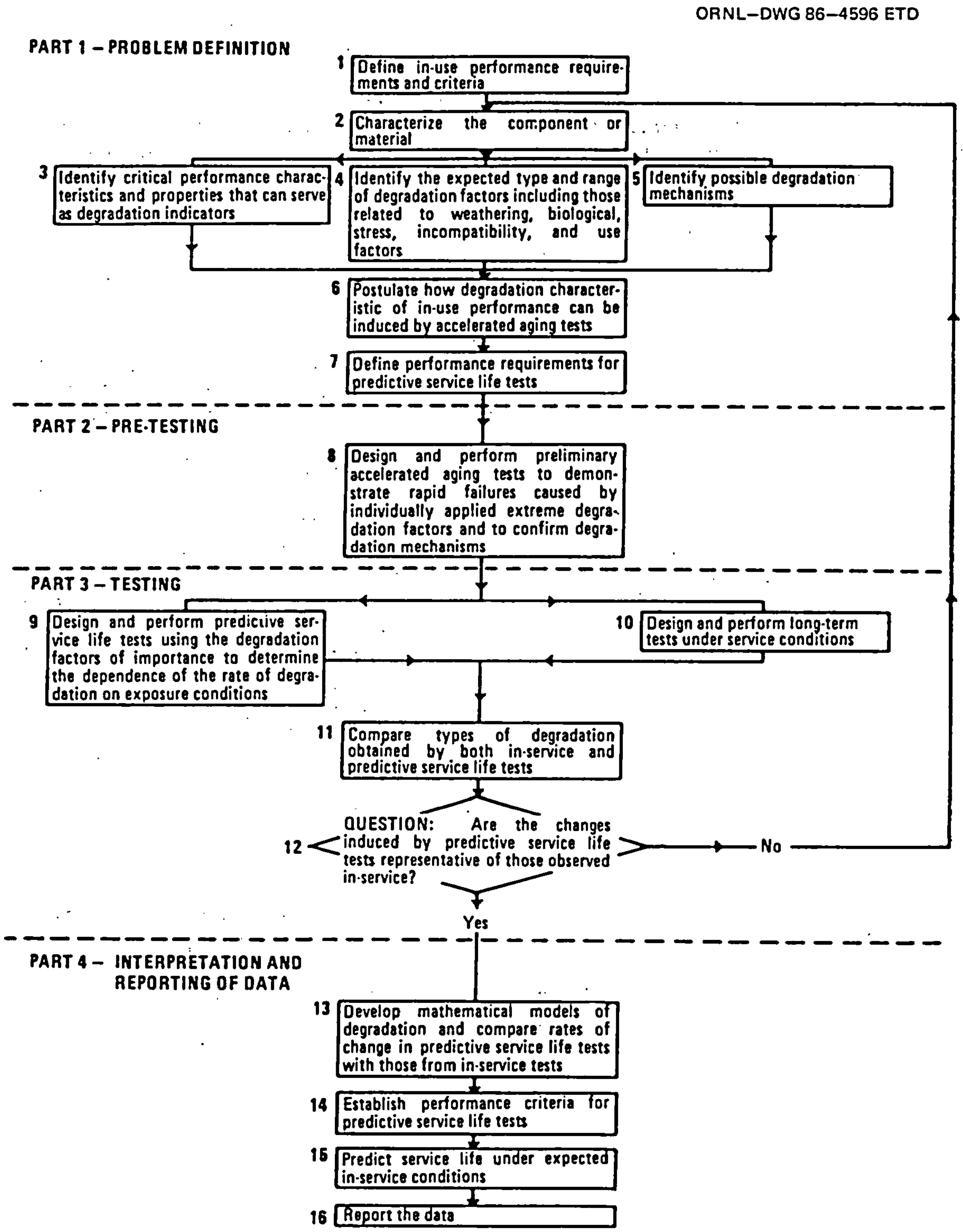


Fig. 56. ASTM E 632 recommended practice for developing predictive service life tests. *Source*: "Standard Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials," ASTM E 632, Part 4, Concrete and Mineral Aggregates, *Annual Book of Standards*, American Society for Tests and Materials, Philadelphia, 1979.

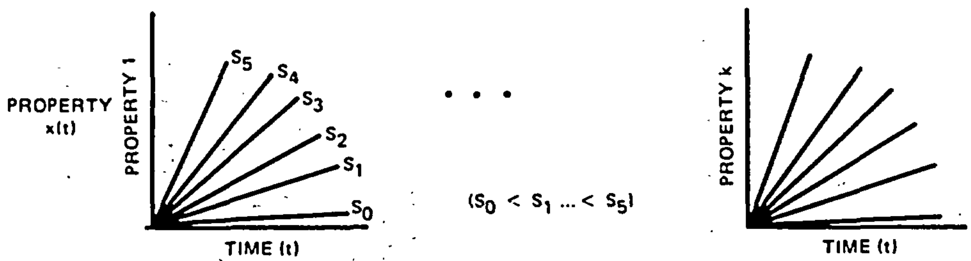
Table 7. Several mathematical models used in accelerated aging studies

Model	Description
$P = b_1 \exp\left\{-\left[\left(\frac{t + b_2}{b_3}\right)^{b_4}\right]\right\} + b_5$	Model for weathering of plastic materials[6]
$P = b_0 + b_1X_1 + b_2X_2 + b_{11}X_1^2 + b_{22}X_2^2 + b_{12}X_1X_2$	Study of temperature and irradiation effects on a composite[7]
$P = b_0 + \sum_{i=1}^{4} b_iX_i + \sum_{j=1}^{4}\sum_{i=1}^{4} b_{ij}X_iX_j$	Study of irradiation, temperature, water, and exposure time effects on a polyethylene and PVC[8]
$\log P = b_0 + b_1 (t - 250)$	Study of weatherometer testing of polystyrene, PVC, and cross-linked polyester[9]
$\log \frac{P_t}{P_0} = k_t$; $k = A \exp(-B/RT)$	Study of heat aging of polyacrylonitrite and polychloroprene elastomer system[10]
$P = P_0 + k \log t$	Study of propellent life[11]
$P = P_0 + b_1t$	Study of potting compounds, adhesives, spiralloy, and pressure seals[12]
$f'_c = A + B \log M$	Accelerated strength testing of concrete[2]

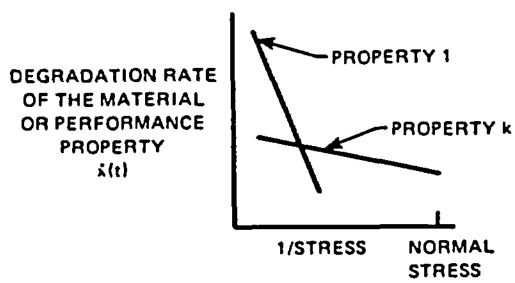
Table 8. Major steps and schematic diagrams for analysis of data obtained from a prototype accelerated life test[a]

---

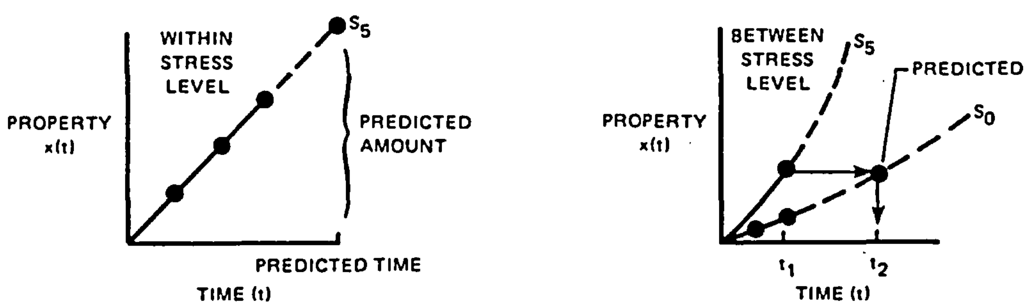
1. Measure degradation rate for each quality $x_i(t)$ associated with observed changes in material properties and performance characteristics for generalized stress levels S_0, S_1 ... S_n where each successive stress level is of higher magnitude than previous stress level.



2. Relate degradation rates to measures of environmental and/or operational stresses (i.e., Arrhenius type plot).



3. Predict amount of degradation of each quality expected to be observed at next measurement time.



4. Verify predicted degradation amounts using data obtained from subsequent measurement times

---

[a]Adapted from: G. B. Gaines et al., *Final Report on Methodology for Accelerated Aging Tests for Predicting Life of Photovoltaic Arrays*, ERDA/JPL 954328-77/1, Battelle Columbus Laboratories, Feb. 1, 1977.

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) the structural reliability techniques to estimate the ability of a structure or component to meet potential future requirements, such as a LOCA.

7.3.1 Component classification scheme

Considerable work toward development of a classification scheme has been done by the School of Civil and Mining Engineering at the University of Sydney.[14] In the study, flow charts are used to categorize types of structure, elements, and causes and effects of durability problems. Structures are initially categorized according to use (e.g., thermal power station) and then broken down by structural classification (e.g., building), structural element classification (e.g., wall), element subclassification (e.g., shear wall), durability factors (e.g., metallic corrosion), and types of deterioration phenomena (e.g., rebar general corrosion). Figure 57 presents an example of the detail that can be realized with the University of Sydney categorization system. Measurement, intensity, and distribution factors for each of the deterioration effects were also developed. Results of this study should have direct application to LWR nuclear-safety-related concrete components, particularly if the consequences of component failure are also factored into the study.

7.3.2 Methodology for conducting a quantitative assessment of the presence of active deteriorating influences and their effects

Detection of age-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-service inspection (ISI) requirements are imposed on nuclear plants through documents such as the following: 10 CFR 50; Nuclear Regulatory Commission (NRC) Regulatory Guides; Plant Technical Specifications; Inspection and Enforcement (I&E) Bulletins; NRC letters; and American Society of Mechanical Engineers (ASME) *Boiler and Pressure Vessel Code*.[15] However, because each nuclear plant has a different construction permit docket date, construction permit issue date, and operating license issue date, each plant could potentially have a different set of minimum ISI requirements. Therefore, to simplify life extension of nuclear-safety-related concrete components, having a standardized ISI program that could not only be used to identify but also to quantify any deteriorating influences would be advantageous.

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: for example, *Guide for Making a Condition Survey of Concrete in Service*,[16] *Strength Evaluation of Existing Concrete in Service*,[17] *Practices for Evaluation*

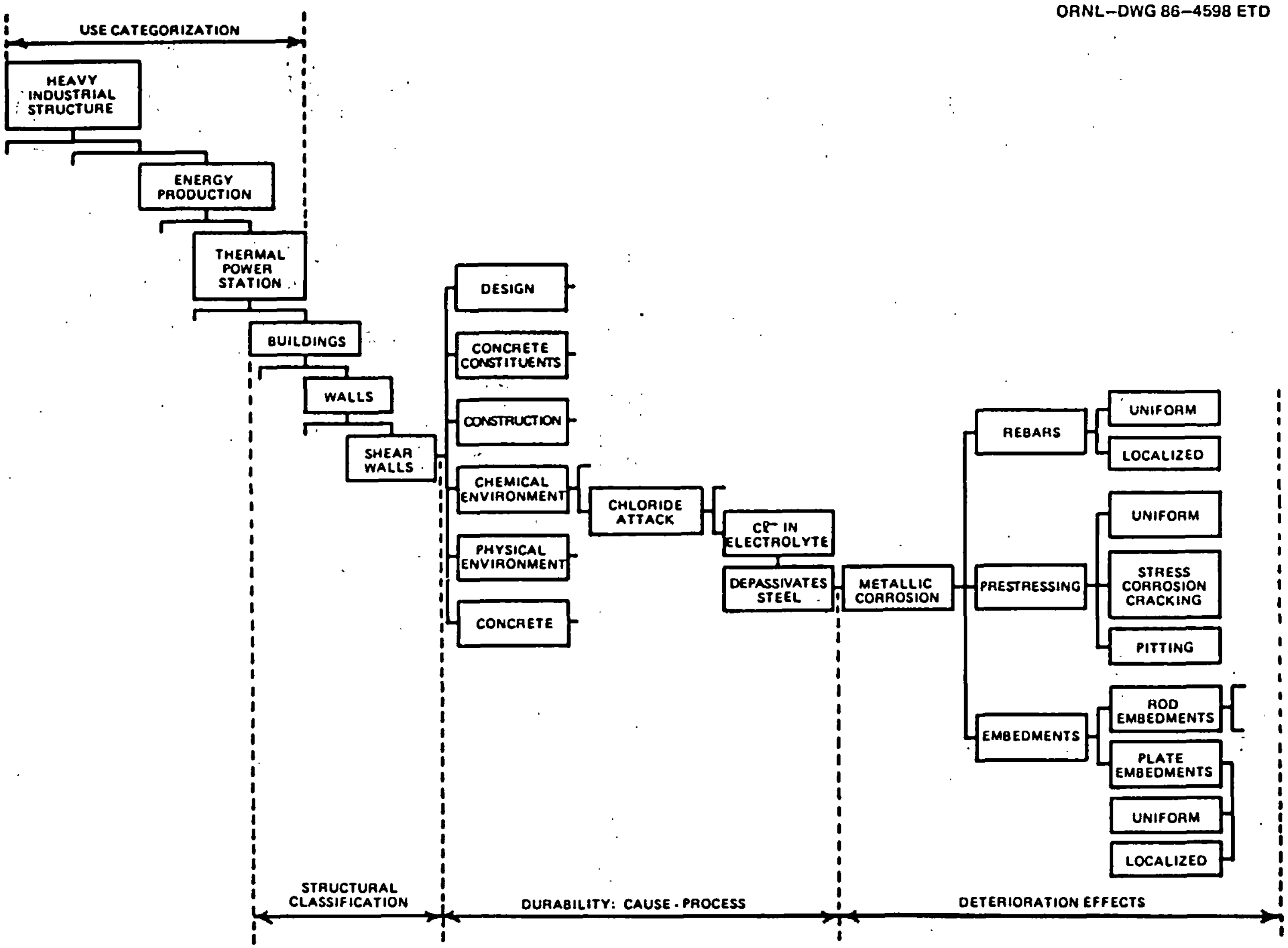


Fig. 57. Systematic evaluation method for determination of structural durability factors. *Source*: Based on material presented in H. Roper, D. Baweja, and G. Kirby, "Towards a Quantitative Measure of Durability of Concrete Structural Members," Paper SP 82-32, *In Situ/Nondestructive Testing of Concrete*, Publication SP-82, American Concrete Institute, Detroit, October 1984.

of Concrete in Existing Massive Structures for Service Conditions,[18] and *Guide for Concrete Inspection*.[19] Additional information is also contained in Refs. 20–22. The application of requirements presented in these documents to nuclear-safety-related structures being considered for life extension, however, needs to be evaluated.

A methodology similar to that presented in Fig. 58* needs to be developed, and criteria need to be established for application to nuclear-safety-related concrete components. Quantification of durability factors, such as those presented in Ref. 14, needs to be addressed and input into the methodology for evaluating the structural condition of concrete components. Limits need to be placed on magnitudes of the deterioration factors (e.g., crack sizes), and probabilistic techniques should be applied to account for random variations and uncertainties in the measured parameters that can affect loadings and material strengths. Once this procedure has been developed, the issue of determining the reliability of the structure to meet potential future requirements and/or prediction of component service life can be addressed.

7.3.3 Structural reliability technique development for life extension evaluations

Once it has been established that a component has been subjected to environmental stressors that have resulted in deteriorating influences,† the effects of these influences must be related to a structural reliability assessment, especially if the component is being considered for an extended service life. A methodology for conducting such an assessment presently does not exist. One approach, however, might be to calculate the reliability of the particular component by using a safety index factor in conjunction with a damage probability matrix that would characterize the probabilistic nature of the damage that had occurred or is expected to occur over the component's anticipated service life.

Shinozuka and Tan have used the damage probability matrix approach to estimate the reliability of a seismically damaged concrete structure when subjected to a future earthquake.[23] Damage states are defined in the study, and conditional, as well as initial, damage probability matrices are introduced in such a manner that the definition of damage is consistent with the kind of accuracy achieved when the extent of structural damage is estimated through field inspections. The initial damage probability matrix indicates the probabilities that an undamaged structure will experience various degrees of damage, representative of the corresponding states of damage, after it is subjected to an earthquake of specified intensity. The conditional damage probability matrix used is essentially a Markovian transition matrix that describes the transition probability with which a structure in a certain state of damage will reach another state after being subjected to an earthquake of a

*Adaptation of a procedure presented in Ref. 22.

†Or for that matter, even a structure that exhibited no signs of deteriorating influences would have to be evaluated for life extension considerations.

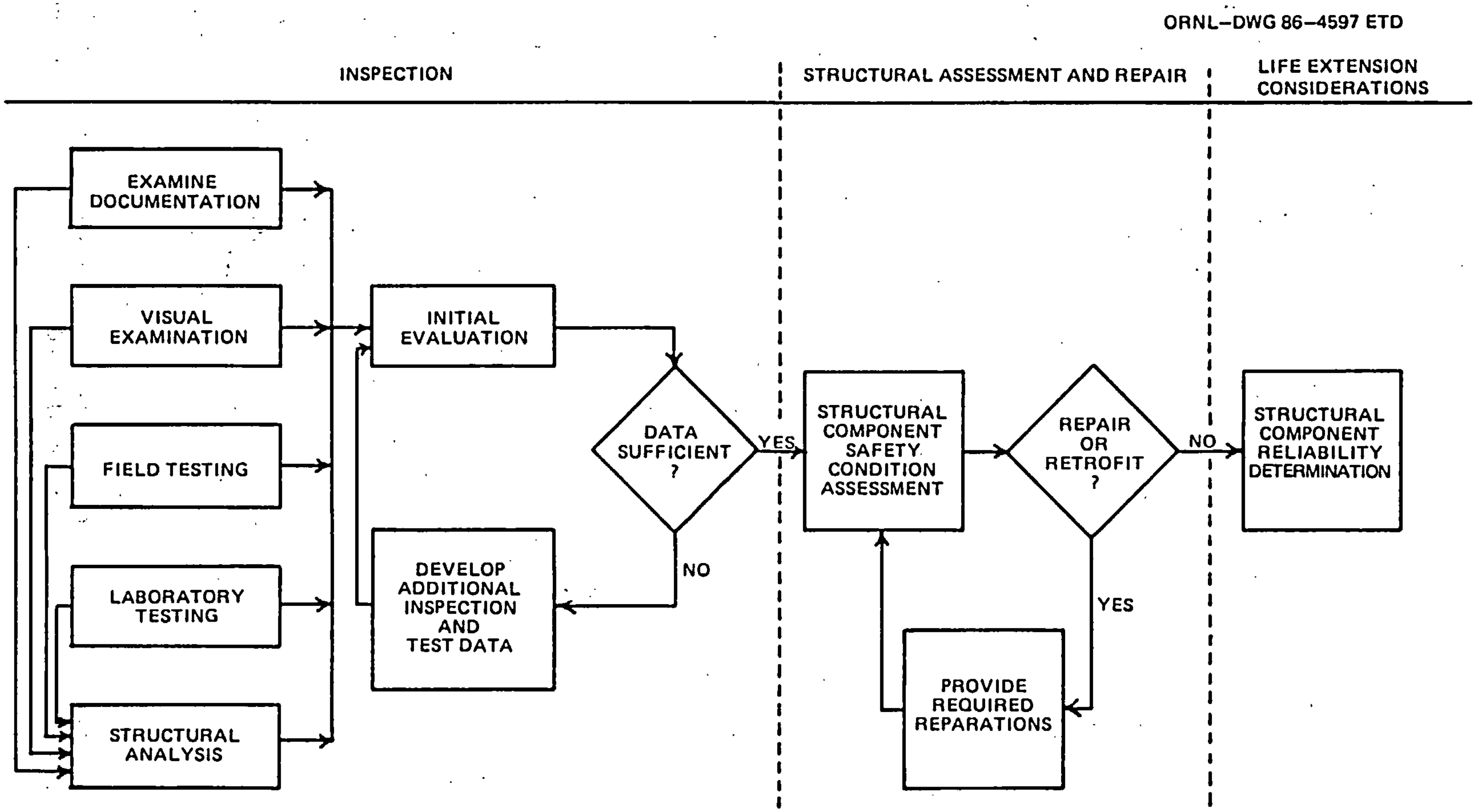


Fig. 58. LWR concrete component evaluation methodology. *Source*: Adaptation of a procedure presented in T. L. Rewarts, "Safety Requirements and the Evaluation of Existing Buildings," *Concr. Int'*l. 7(4), American Concrete Institute, Detroit, April 1985.

given intensity. The validity of using probability damage matrices is demonstrated analytically by considering a vertically standing, symmetrically reinforced concrete cantilever beam subjected to a horizontal ground acceleration (modeled as a nonstationary random process). Crack widths that develop near the beam fixed end are related to damage ratios (stiffness change) that develop under different earthquake intensities. Further verification is provided by a Monte Carlo simulation with the aid of a nonlinear dynamic structural analysis involving artificially generated earthquakes.

Results obtained in the above investigation are sufficiently encouraging that the use of a damage probability matrix approach* should be considered as a method for addressing future structural reliability determinations. Various deteriorating effects (e.g., cracking) resulting from environmental stressors could be modeled and their effect on structural performance determined under simulated LOCA conditions.

---

*Concepts of fracture mechanics as applied to cracked reinforced concrete structures also may merit consideration.

References

1. V. Ramakrishman and J. Dietz, *Accelerated Strength Testing*, SP-56, Paper 7, American Concrete Institute, Detroit, 1978, pp. 95–116.

2. H. S. Lew and T. W. Reichard, *Prediction of Strength of Concrete from Maturity*, SP-56, Paper 14, American Concrete Institute, Detroit, 1978, pp. 229–48.

3. G. L. Kalousek et al., "Concrete for Long-Time Service in Sulfate Environment," *Cem. Concr. Res.* 2(1) (1972).

4. "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing," ANSI/ASTM C666, Part 14, Concrete and Mineral Aggregates, *Annual Book of Standards*, American Society for Testing and Materials, Philadelphia, 1979.

5. "Standard Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials," ASTM E 632, Sect. 4, *1985 Annual Book of ASTM Standards*, American Society for Testing and Materials, Philadelphia, 1985.

6. National Bureau of Standards and Manufacturing Chemists Association, *Outdoor Performance of Plastics I through X*, 1968–1972.

7. M. E. Leikina and G. O. Talevos'yan, "Planning an Experiment for Studying the Light Aging of Polyethylene Film, *Sov. Plast.* (7), 73 (1970).

8. M. E. Leikina et al., "Use of Mathematical Experiment Planning Method to Investigate the Influence of Artificial Weather Factors on Certain Properties of Polymer Films," *Sov. Plast.* (2), 41 (1971).

9. M. R. Kamal, "Effect of Variables in Artificial Weathering on the Degradation of Selected Plastics," *Polym. Eng. Sci.* 6, 333 (1966).

10. D. K. Thomas and R. Sinnott, "Prediction of Shelf Storage Life from Accelerated Heat Aging Tests," *J. IRI*, p. 163 (1969).

11. G. E. Myers, *Chemical Structural Aging Tests*, MANCP Report No. 297, 1974.

12. E. M. Dalaba, *Minuteman Stage III Component Report*, AFRPL-TR-74-58, 1974.

13. G. B. Gaines et al., *Final Report on Methodology for Designing Accelerated Aging Tests for Predicting Life of Photovoltaic Arrays*, ERDA/JPL 954328-77/1, Battelle Columbus Laboratories, Columbus, Ohio, Feb. 1, 1977.

14. H. Roper et al., *Durability Studies of Concrete Structures Classification and Quantification*, Research Report No. R465, School of Civil and Mining Engineering, University of Sydney, Australia, January 1984.

15. D. D. Davis, "Optimizing an Inservice Inspection Program Plan," *Nuclear Plant Safety*, 37–40 (May–June 1985).

16. ACI Committee 201, "Guide for Making a Condition Survey of Concrete In-Service," pp. 905–18 in *Proc. J. Am. Concr. Inst.* 65(11) (November 1968).

17. ACI Committee 437, "Strength Evaluation of Existing Concrete Buildings," *J. Am. Concr. Inst.* 64(11), 705–10 (November 1967).

18. ACI Committee 207, "Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions," *Concr. Int.* 1(3), 47–61 (March 1979).

19. ACI Committee 311, "Guide for Concrete Inspection," *Concr. Int.* 2(10), 81–85 (October 1980).

20. B. Bresler, *Essential Steps in Adaptation of Old Buildings*, Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11–15 1977.

21. D. Kaminetzky, *Verification of Structural Adequacy*, Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, University of California, Berkeley, July 11–15, 1977.

22. T. L. Rewarts, "Safety Requirements and the Evaluation of Existing Concrete Buildings," *Concr. Int.* 7(4), 50–55 (April 1985).

23. M. Shinozuka and R. Y. Tan, "Seismic Reliability of Damaged Concrete Beams, *J. Struct. Div.* 109(7) 1617–34 (July 1983).

8. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

8.1 Summary

The objectives of the study were to (1) expand upon the work that was initiated in the first two Electric Power Research Institute studies relative to longevity and life extension considerations of safety-related concrete components in light-water reactor (LWR) facilities and (2) provide background that will logically lead to subsequent development of a methodology for assessing and predicting the effects of aging on the performance of concrete-based materials and components. These objectives are consistent with Nuclear Plant Aging Research (NPAR) Program goals to: (1) identify and characterize aging and service wear effects that, if unchecked, could cause degradation of structures, components, and systems and, thereby, impair plant safety; (2) identify methods of inspection, surveillance, and monitoring, or of evaluating residual life of structures, components, and systems, that will ensure timely detection of significant aging effects before loss of safety function; and (3) evaluate the effectiveness of storage, maintenance, repair, and replacement practices in mitigating the rate and extent of degradation caused by aging and service wear.[1]

Applications of safety-related concrete components to LWR technology were identified, and pertinent components (containment buildings, containment base mats, biological shield walls and buildings, and auxiliary buildings), as well as the materials of which they are constructed (concrete, mild steel reinforcement, prestressing systems, embedments, and anchorages), were described. Historical performance of concrete components was established through information presented on concrete longevity, component behavior in both LWR and high-temperature gas-cooled reactor applications, and a review of problems with concrete components in both general civil engineering and nuclear power applications. The majority of the problems identified in conjunction with nuclear power applications were minor and involved either concrete cracking, concrete voids, or low concrete strengths at early ages. Five incidences involving LWR concrete containments considered significant were described in detail from occurrence and detection through remedial measures used to restore structural integrity or continuity. These incidences were related to design, construction, or human error and involved two dome delaminations, voids under tendon-bearing plates, anchor head failures, and a breakdown in quality control and construction management.

Potential environmental stressors and aging factors to which LWR safety-related components could be subjected were identified and discussed in terms of durability factors related to the materials used to fabricate the components (e.g., concrete, mild steel reinforcement, prestressing systems, and embedments). The current technology for detection of concrete aging phenomena was also presented in terms of methods applicable to the particular material system that could experience deteriorating effects. Remedial measures for the repair or replacement of degraded concrete components were discussed, and examples of prerepair and postrepair structural performance were presented to indicate the effectiveness of these measures. Finally, considerations relative to development of a damage methodology for assessment of durability factor deterioration rates and prediction of structural reliability were discussed.

8.2 Conclusions

Based on the results of this investigation, the following conclusions can be derived:

1. The performance of concrete-based components in both general civil engineering and nuclear power applications has been exemplary. Distress that has occurred was generally due to construction or material errors.
2. Techniques for detecting effects of environmental stressors on concrete materials are sufficiently developed to provide qualitative data. However, quantitative interpretation can be complicated because of either (a) the requirement for development of correlation curves; (b) embedment (rebars, anchorages, etc.) effects on measured quantities, such as time of ultrasonic wave transmission; or (c) accessibility. Also, a methodology for application of this technology to provide required data for either structural reliability or life extension assessments needs development.
3. Remedial measures for repair of degraded concrete components are capable of completely restoring structural integrity when proper techniques and materials are used. However, results obtained from shear wall components subjected to fire exposure after being repaired by structural epoxies indicate that some additional work on development of more temperature-resistant epoxies may be merited.
4. The durability of concrete constructions is affirmed by the presence of many structures that have been in existence for periods of time ranging from several decades to several millenia; however, well-documented data on concrete longevity that can be used as a basis for life extension considerations is almost nonexistent.
5. Primary effects that could lead to a loss of serviceability of concrete components in LWR plants include concrete cracking and loss of strength resulting from environmental stressors; however, severity criteria (e.g., statistically-based crack width tolerances and corrosion inhibitor impurity levels) for degradation of these components need to be established.
6. A damage methodology to provide a quantitative measure of the durability of a structure with respect to meeting potential future requirements [e.g., loss-of-coolant accident (LOCA)] does not presently exist.

8.3 Recommendations

The following recommendations are made:

1. Existing facilities that have been shut down after an extended period of service (e.g., Shippingport, Dresden 1, and Humbolt Bay) should be used to obtain aging-related data for concrete materials. Also, these

facilities can be used to evaluate the applicability of various techniques for detecting the effects of environmental stressors (primarily elevated temperature and irradiation) on the concrete materials. By comparing results from the nondestructive examination/tests with those obtained from core tests, considerable insight can be gained toward evaluating the ability of these tests to provide quantitative data useful for residual life assessments.

2. Accelerated aging techniques should be investigated as a method for supplementing the extremely limited data base on concrete aging. This technique would also have application to other materials used in conjunction with concrete.

3. Available prestressing tendon in-service inspection records and data obtained during containment integrated leak-rate tests should be examined as potential sources of information for trending concrete component behavior. Also, for plants that are likely candidates for life extension considerations (e.g., plants with lengthy construction periods), consideration should be given to increased emphasis on in-service inspections to provide trending information that could potentially shorten the process required for life extension evaluations.

4. Criteria on durability factor* significance need to be established.

5. A methodology needs to be developed to provide a quantitative measure of structural reliability either now or later. Such a methodology would use a systems approach and encompass component classification, techniques for quantitative determination of presence and magnitude of deteriorating influences, and structural reliability assessments. By using trending of environmental stressor data (concrete aging), the scheme would enable (a) an assessment of the ability (probability) of various safety-related concrete components to meet their design requirements (e.g., LOCA) later and (b) prediction of a component's residual life. For example, an estimation can be made of the time when the influence of an environmental stressor would produce a decrease in concrete strength to a value below that specified in the design as necessary to ensure that the structural component meets normal operating and accident condition requirements.

Reference

1. B. M. Morris and J. P. Vora, *Nuclear Plant Aging Research (NPAR) Program Plan*, NUREG-1144, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, Washington, D.C., July 1985.

---

*Identification of the various deterioration phenomena acting on a particular structure and the assignment of a weighting factor to each of the phenomena on its significance relative to life extension considerations.

Appendix A

LICENSED U.S. POWER REACTORS AS OF APRIL 30, 1985
[From *Nuclear Safety* 26(4), July–August 1985]

Reactor	Docket No.	Reactor[a] type (designer)	Design power MW(t)	Design power MW(e)	Operating license	Containment type[b]
Arkansas 1	50-313	P(B&W)	2568	850	1974	PC — shallow dome, 3 buttresses
Arkansas 2	50-368	P(CE)	2815	912	1978	PC — shallow dome, 3 buttresses
Beaver Valley 1	50-334	P(West)	2652	852	1976	RC — subatmospheric
Big Rock Point	50-155	B(GE)	240	75	1964	S — spherical
Browns Ferry 1	50-259	B(GE)	3293	1065	1973	S — MKI
Browns Ferry 2	50-260	B(GE)	3293	1065	1974	S — MKI
Browns Ferry 3	50-296	B(GE)	3293	1065	1976	S — MKI
Brunswick 1	50-325	B(GE)	2436	821	1976	RC — MKI
Brunswick 2	50-324	B(GE)	2436	821	1974	RC — MKI
Byron 1[c]	50-454	P(West)	3425	1120	1984	PC — shallow dome, 3 buttresses
Callaway 1	50-483	P(West)	3411	1120	1984	PC — hemispherical dome, 3 buttresses
Calvert Cliffs 1	50-317	P(CE)	2560	845	1974	PC — shallow dome, 6 buttresses
Calvert Cliffs 2	50-318	P(CE)	2560	845	1976	PC — shallow dome, 6 buttresses
Catawba 1[c]	50-413	P(West)	3411	1145	1984	S — ice condenser
Cook 1	50-315	P(West)	3250	1054	1974	RC — ice condenser
Cook 2	50-316	P(West)	3391	1060	1977	RC — ice condenser
Cooper	50-298	B(GE)	2831	778	1974	S — MKI
Crystal River 3	50-302	P(B&W)	2560	802	1976	PC — shallow dome, 6 buttresses
Davis-Besse 1	50-346	P(B&W)	2772	906	1977	S — cylindrical
Diablo Canyon 1[c]	50-275	P(West)	3338	1084	1984	RC — hemispherical dome
Dresden 1[d]	50-10	B(GE)	700	200	1960	S — spherical
Dresden 2	50-237	B(GE)	2527	794	1969	S — MKI
Dresden 3	50-249	B(GE)	2527	794	1971	S — MKI
Duane Arnold	50-331	B(GE)	1593	538	1974	S — MKI
Farley 1	50-348	P(West)	2652	829	1977	PC — shallow dome, 3 buttresses
Farley 2	50-364	P(West)	2652	829	1980	PC — shallow dome, 3 buttresses
Fermi 2	50-341	B(GE)	3292	1093	1985	S — MKI
Fitzpatrick	50-333	B(GE)	2436	821	1974	S — MKI
Fort Calhoun	50-285	P(CE)	1420	457	1973	PC — partial prestress, 0 buttresses
Fort St. Vrain	50-267	HTGR(GAT)	842	330	1973	PC
Ginna	50-244	P(West)	1520	490	1969	PC — hemispherical dome, 0 buttresses
Grand Gulf 1[c]	50-416	B(GE)	3833	1250	1982	RC — MKIII
Haddam Neck	50-213	P(West)	1825	575	1974	RC — hemispherical dome
Hatch 1	50-321	B(GE)	2436	786	1974	S — MKI
Hatch 2	50-366	B(GE)	2436	795	1978	S — MKI
Humboldt Bay[d]	50-133	B(GE)	220	63	1969	S — cylindrical
Indian Point 2	50-247	P(West)	2758	873	1971	RC — hemispherical dome
Indian Point 3	50-286	P(West)	2760	873	1975	RC — hemispherical dome

Reactor	Docket No.	Reactor[a] type (designer)	Design power MW(t)	Design power MW(e)	Operating license	Containment type[b]
Kewaunee	50-305	P(West)	1650	535	1973	S — cylindrical
La Crosse	50-409	B(A-C)	165	50	1973	S — cylindrical
La Salle 1	50-373	B(GE)	3323	1078	1982	PC — MKII
La Salle 2	50-374	B(GE)	3323	1078	1984	PC — MKII
Limerick 1[e]	50-352	B(GE)	3293	1065	1984	RC — MKII
Maine Yankee	50-309	P(CE)	2560	790	1972	RC — subatmospheric
McGuire 1	50-369	P(West)	3411	1180	1981	S — ice condenser
McGuire 2	50-370	P(West)	3411	1180	1983	S — ice condenser
Millstone Point 1	50-245	B(GE)	2011	660	1970	S — MKI
Millstone Point 2	50-336	P(CE)	2560	830	1975	PC — shallow dome, 3 buttresses
Monticello	50-263	B(GE)	1670	545	1971	S — MKI
Nine Mile Point 1	50-220	B(GE)	1850	610	1974	S — MKI
North Anna 1	50-338	P(West)	2775	898	1977	RC — subatmospheric
North Anna 2	50-339	P(West)	2775	907	1980	RC — subatmospheric
Oconee 1	50-269	P(B&W)	2568	887	1973	PC — shallow dome, 6 buttresses
Oconee 2	50-270	P(B&W)	2568	887	1973	PC — shallow dome, 6 buttresses
Oconee 3	50-287	P(B&W)	2568	887	1974	PC — shallow dome, 6 buttresses
Oyster Creek	50-219	B(GE)	1930	650	1969	S — MKI
Palisades	50-255	P(CE)	2200	805	1971	PC — shallow dome, 6 buttresses
Palo Verde 1[e]	50-528	P(CE)	3817	1250	1984	PC — hemispherical dome, 3 buttresses
Peach Bottom 2	50-277	B(GE)	3293	1065	1973	S — MKI
Peach Bottom 3	50-278	B(GE)	3293	1065	1974	S — MKI
Pilgrim 1	50-293	B(GE)	1998	655	1972	S — MKI
Point Beach 1	50-266	P(West)	1518	497	1970	PC — shallow dome, 6 buttresses
Point Beach 2	50-301	P(West)	1518	497	1971	PC — shallow dome, 6 buttresses
Prairie Island 1	50-282	P(West)	1650	530	1973	S — cylindrical
Prairie Island 2	50-306	P(West)	1650	530	1974	S — cylindrical
Quad Cities 1	50-254	B(GE)	2511	789	1971	S — MKI
Quad Cities 2	50-265	B(GE)	2511	789	1972	S — MKI
Rancho Seco	50-312	P(B&W)	2772	918	1974	PC — shallow dome, 3 buttresses
Robinson 2	50-261	P(West)	2200	700	1970	PC — RC hemispherical dome, 0 buttresses
Salem 1	50-272	P(West)	3423	1090	1976	RC — hemispherical dome
Salem 2	50-311	P(West)	3423	1115	1980	RC — hemispherical dome
San Onofre 1	50-206	P(West)	1347	430	1967	S — spherical
San Onofre 2	50-361	P(CE)	3410	1100	1982	PC — hemispherical dome, 3 buttresses
San Onofre 3	50-362	P(CE)	3410	1100	1982	PC — hemispherical dome, 3 buttresses
Sequoyah 1	50-327	P(West)	3423	1148	1980	S — ice condenser

Reactor	Docket No.	Reactor[a] type (designer)	Design power MW(t)	Design power MW(e)	Operating license	Containment type[b]
Sequoyah 2	50-328	P(West)	3423	1148	1981	S — ice condenser
St. Lucie 1	50-335	P(CE)	2560	802	1976	S — cylindrical
St. Lucie 2	50-389	P(CE)	2560	810	1983	S — cylindrical
Summer 1	50-395	P(West)	2775	900	1982	PC — shallow dome, 4 buttresses
Surry 1	50-280	P(West)	2441	822	1972	RC — subatmospheric
Surry 2	50-281	P(West)	2441	822	1973	RC — subatmospheric
Susquehanna 1	50-387	B(GE)	3293	1050	1982	RC — MKII
Susquehanna 2	50-388	B(GE)	3293	1050	1984	RC — MKII
Three Mile Island 1	50-289	P(B&W)	2535	819	1974	PC — shallow dome, 6 buttresses
Three Mile Island 2[d]	50-320	P(B&W)	2772	906	1978	PC — shallow dome, 6 buttresses
Trojan	50-344	P(West)	3411	1130	1975	PC — hemispherical dome, 3 buttresses
Turkey Point 3	50-250	P(West)	2200	693	1972	PC — shallow dome, 6 buttresses
Turkey Point 4	50-251	P(West)	2200	693	1973	PC — shallow dome, 6 buttresses
Vermont Yankee	50-271	B(GE)	1593	514	1972	S — MKI
Washington NP 2	50-397	B(GE)	3323	1100	1984	S — MKII
Waterford 3[e]	50-382	P(CE)	3410	1113	1984	S — cylindrical
Wolf Creek 1	50-482	P(West)	3411	1150	1985	PC — hemispherical dome, 3 buttresses
Yankee Rowe	50-29	P(West)	600	175	1961	S — spherical
Zion 1	50-295	P(West)	3250	1040	1973	PC — shallow dome, 6 buttresses
Zion 2	50-304	P(West)	3250	1040	1973	PC — shallow dome, 6 buttresses

[a]P = pressurized-water reactor
B = boiling-water reactor
B&W = Babcock and Wilcox
CE = Combustion Engineering
West = Westinghouse
GE = General Electric
GAT = GA Technologies Inc.
A-C = Allis Chalmers

[b]PC = prestressed concrete
RC = reinforced concrete
S = steel

[c]In power ascension phase.

[d]Operating license but shut down indefinitely.

[e]Licensed for low-power testing.

Appendix B

ANNOTATED LISTING OF PROBLEM AREAS ASSOCIATED WITH CONCRETE COMPONENTS IN LWR APPLICATIONS

Reactor plant	Docket No.	Year of commercial operation	Year of occurrence	Summary description
Yankee Rowe	50-29	1961	1967	A 4.6-m (15-ft) shrinkage crack, covered with fiberglass and recoated
San Onofre 1	50-206	1968	1976	Voids at 14 locations in diesel generator building center wall; areas from 0.09 m^2 (1 ft^2) with 7- to 10-cm (3- to 4-in.) penetration to several square meters (square feet) with full penetration; repaired with dry pack, grout, or concrete
Ginna	50-244	1970	1981	Excessive loss of prestressing, tendons retensioned with no recurrence noted in subsequent inspections
Indian Point 2	50-247	1974	1974	Concrete temperature local to hot penetration >66°C (150°F) but <93°C (200°F), no safety problem due to relatively low periods of exposure
Turkey Point 3	50-250	1972	1968	Voids below containment wall and near reactor pit, repaired with high-strength grout, guniting, or dry packing
			1970[a]	Dome delamination; delaminated concrete removed, additional rebars provided, concrete replaced
			1974	Grease leakage from 110 of 832 tendons at casing, tendon casings repaired and refilled
			1975	Concrete spalling at horizontal joint at containment ring girder with cavities 3 to 5 cm (1 to 2 in.) wide by 7 to 10 cm (3 to 4 in.) deep, no threat to structural integrity, repaired by dry packing
			1982	Small void under equipment hatch barrel, no threat to structural integrity, repaired by grouting
Turkey Point 4	50-251	1973	1981	Approximately 0.1 m^3 (0.4 ft^2) of concrete with inadequate fines, area removed and refilled with concrete
Palisades	50-255	1971	1975	Sixty-three out of 3780 buttonheads inspected found split, no threat to structural integrity
Fort St. Vrain	50-267	1979	1984[b]	Tendon wire failures noted because of tendon corrosion caused by microbiological attack of corrosion inhibitor, analysis revealed sufficient tendons intact to provide structural integrity, and surveillance increased and tendons inerted by nitrogen blanket
Oconee 2/3	50-270/287	1974	1982	During final reactor building interior inspection, two vertical tendons in secondary shield wall of unit 2 were found failed and some tendons in units 2 and 3 were exhibiting corrosion near stressing washers; tendons are not required to meet shield wall functions but were replaced and bottom grease caps redesigned to permit water drainage; surveillance was increased
			1983	Four tendons in reactor building found ungreased, tendons inspected and grease applied
Peach Bottom 2/3	50-277/278	1974	1969	Aluminum pipe used to place concrete caused concrete strength reduction up to 50%, low-strength concrete in biological shield wall and floor slab of turbine building replaced
Surry 1	50-280	1972	1979	Cracking in concrete supports for two heat exchangers caused by thermal expansion of heat exchanger shells, cracks repaired and supports modified

Reactor plant	Docket No.	Year of commercial operation	Year of of occurrence	Summary description
Three Mile Island 1	50-289	1974	1975	Two of six concrete footings for rigid pipe supports cracked due to design deficiency, footings were replaced using a new design
			1974	Cracking <0.02 cm (<0.010 in.) wide in containment building ring girder and around tendon bearing plates, cracks repaired and monitored during subsequent surveillance
Zion 1	50-295	1973	1972	Excessive pitting observed in some tendon wires of unit 2 during installation, cause was outdoor storage in conjunction with high precipitation and inadequate protection, defective tendons replaced
Crystal River 3	50-302	1978	1974	Twenty-eight-day concrete strength was low due to failure of cement to meet specifications; design review revealed strength attained to be adequate; cement inspection increased
			1976[a]	Dome delaminated over ~32-m-diam (105-ft) area due to low concrete properties, radial tension due to prestressing, and biaxial failure criterion; upper delaminated section removed, additional rebars provided, concrete replaced, dome retensioned, and structural integrity test conducted
Salem 2	50-311	1981	1974	Incomplete concrete pour near equipment hatch due to use of wrong concrete mix, voids repaired with high-strength nonshrink grout
Rancho Seco	50-312	1975	1974	Concrete surface temperature >66°C (150°F) during initial power escalation
Cook 1/2	50-315/316	1975/1978	1974	Cracking in spent fuel pit wall and slabs framing into pit walls, cause was thermal expansion and hydrostatic pressure, no threat to structural integrity
Calvert Cliffs 1/2	50-317/318	1975/1977	1971/1972[a]	Eleven of top bearing plates of units 1 and 2 depressed into concrete because of voids; 190 plates of each containment exhibited voids upon inspection; tendons detensioned, plates grouted and tendons retensioned
Three Mile Island 2	50-320	1978	1974	Four of six sets of compression cylinders had low f'_c because of mishandling and inventory control at cement silo, 90-d strengths were acceptable and concrete in-place determined to have adequate strength; cement storage and sampling techniques improved
			1975	Void ~7 cm (3 in.) high × 1.8 m (6 ft) wide × 0.9 to 1.5 m (3 to 5 ft) deep occurred in south exterior wall of fuel-handling building, cause was improper placement, void determined not to be a threat to structural or shielding effectiveness; void refilled
			1976	Void 0.9 to 1.2 m (3 to 4 ft) into concrete 0.4 m (1.5 ft) high by 1.8 to 2.4 m (6 to 8 ft) wide in north exterior wall of fuel transfer canal, void repaired, no structural or shielding effectiveness threat
Hatch 1	50-321	1975	1981	Cracks in concrete wall around base plate
			1981	Concrete in pedestal for several recirculation line snubbers exhibited spalling and cracking due to design deviation, 2.5-cm (1-in.) plates with four wedge anchors installed on top of existing plates
Shoreham	50-322	UC[c]	1974	Unconsolidated and honeycombed areas in first lift of reactor support pedestal, voids repaired after determining that they were not a threat to structural integrity, placement procedures improved

Reactor plant	Docket No.	Year of commercial operation	Year of of occurrence	Summary description
Brunswick 1/2	50-324/325	1977/1975	1974	Voids occurred behind liner during construction of suppression chamber, grout injected into voids through holes drilled in liner, some grout in unit 1 did not harden but was left in place to provide limited resistance
Sequoyah 2	50-328	1982	1978	Concrete in outer 2.5 to 5 cm (1 to 2 in.) of unit 2 shield building was under-strength because of exposure to freezing temperatures at early concrete age, determined not to affect shield building capability
Midland 2	50-330	c[d]	1975	Rebar spacing deficiencies in reactor containment building, determined error not significant enough to affect safety
			1977	Leaking water pipe in exterior wall caused bulging of liner plate up to 0.6 m (2 ft) inwards over an area of about 195 m^2 (2100 ft^2) producing concrete spalling of 7.5 to 25.4 cm (3 to 10 in.) deep, bulged liner plate and concrete removed
Duane Arnold	50-331	1974	1974	Hairline cracks in floor under torus, cracks permitted to self heal
Fitzpatrick	50-333	1975	1973	Horizontal crack from hairline to 0.9 cm (3/8 in.) wide in reactor pedestal extending into concrete 0.2 to 0.7 m (9 to 30 in.), cause believed to be welding procedure causing tension; structural integrity of pedestal not impaired, crack sealed by epoxy injection
Beaver Valley 1	50-334	1977	1982	Void ~0.9 m (37 in.) long × 0.9 m (3 ft) deep in outer containment wall in concrete ring around equipment hatch, no threat to structural integrity, void repaired
St. Lucie 1	50-335	1976	1974	Concrete spalled because of scaffolding fire in annulus between containment vessel and shield building, area affected ~3.4 m (11 ft) × 0.6 m (2 ft) × 2.5 cm (1 in.), temperature reached 148 to 177°C (300 to 350°F) inflicting only superficial damage, spalled area replaced
			1978	Hairline crack ~1 mm (1/24 in.) wide by 1 m (39 in.) long in east wall of reactor containment refueling canal near embedded steel plate, crack repaired by grouting and column added to support platform girder
North Anna 2	50-339	1980	1974	Cracks >1.6 mm (1/16 in.) in containment floor slab occurred around neutron shield tank anchor bolts, following pressure testing of seal chambers, cause was inadvertent pressurization, cores showed cracks extended into concrete vertically, cracks no structural threat, routed and sealed to prevent fluid penetration
Fermi 2	50-341/342	1985	1972	Cracks <0.8 mm (1/32 in.) wide in basement floor slab permitted groundwater to seep into building, cracks caused by shrinkage, cracks repaired by pressure grouting after determining that they were no threat to structural integrity
			1984	Voids detected around one of auxiliary building watertight doors, defective concrete removed by chipping and area grouted, other doors inspected
Davis Besse 1	50-346	1977	1982	Two concrete expansion anchors and upper part of base plate pulled from wall ~1 cm (3/8 in.) because of improper installation, anchors replaced and torque checked
Farley 1	50-348	1977	1985[a]	Cracks detected in six containment tendon anchors during refueling outage
			1980	Portions of unit 1 walls had areas where vertical reinforcing and grout were missing, corrective action taken
San Onofre 3	50-362	1984	1983	Tendon liftoff force in excess of maximum value listed in technical specifications, cause was lower relaxation rate than expected, no threat to structural integrity

Reactor plant	Docket No.	Year of commercial operation	Year of of occurrence	Summary description
Farley 2	50-364	1981	1985[a]	Three anchor heads on bottom ends of vertical tendons failed and 18 cracked with several tendon wires fractured, occurred about 8 years after tensioning, cause attributed to hydrogen stress cracking, all tendons and anchor heads from same heat inspected with no further problems noted, 20 tendons replaced
Hatch 2	50-366	1979	1979	Approximately 10% failures occurred during testing of 183 anchor bolts because of improper installation, failed bolts replaced with wedge anchors
			1982	Main steam pipe hangers had significant concrete spalling around embedded plate with concrete missing ~5 cm (2 in.) adjacent to plate, cause was defective concrete material or faulty placement, plate redesigned
McGuire 1	50-369	1981	1976	Two buttonheads failed during stressing of CRDM missile shield holddown tendons at underside of bottom plate and two wires failed in another tendon near base anchor, additional failed wires found during checking, cause was excessive corrosion, design modified to replace tendons with 3.5-cm-diam (1-3/8-in.) threaded rods that were grouted into place
La Salle 1/2	50-373/374	1982/1984	1976	Low concrete strength at 90 d, in-place strength determined acceptable from cores and cement contents for future pours increased, strength low in only small percent of pours so did not threaten structural integrity
Waterford 3	50-382	1984	1976	Improper concrete placing sequences used in foundation mat forming a cold joint and not achieving stepped bedding planes, core drilling revealed fine cracks and honeycombed areas, defective concrete removed and replaced, supervision and inspection increased.
			1976	Improper placement of concrete in reactor auxiliary building interior wall resulted in honeycombed areas, voids, and cold joints; unsound concrete removed and repaired
			1977	Crane boom fell during construction on common foundation structure wall causing concrete cracking and spalling over area 0.3 m (12 in.) × 10 cm (4 in.) × 2.5 cm (1 in.), rebars and concrete removed and replaced over entire height of damaged area for a length of 9.5 m (31 ft)
			1977	Low concrete compressive strength in 4.2 m^3 (5.5 yd^3) of concrete in wall contiguous with portion of condensate storage pool wall and wall of refueling water pool
			1977	Low concrete strength in reactor auxiliary building slab, cores yielded satisfactory strength, amount of sand in future mixes increased as well as mixing requirements
			1984	Spalled concrete observed in corbel exposing and displacing rebars and cracking in plane of anchor bolts, no loss of structural support, area repaired
Susquehanna 1/2	50-387/388	1983/1984	1976	Coarse aggregate with excessive fines used because of quality control deficiency, concrete strength exceeded requirements so structural integrity not affected, aggregate material for future batches replaced
Summer 1	50-395	1984	1976	Voids located behind liner plate of reactor containment building wall, windows cut in liner revealed voids up to 22 cm (8.5 in.) deep, cause was use of low slump concrete with insufficient compaction, voids chipped and cleaned to sound concrete, filled with nonshrink grout and liner repaired with all welds leak tested

Reactor plant	Docket No.	Year of commercial operation	Year of of occurrence	Summary description
Summer 1 (continued)			1977	Excessive heat from welding caused liner attached to concrete on inside face of concrete primary shield wall cavity to buckle and fail stud anchors and crack concrete, liner and concrete to depth of 15 cm (6 in.) removed, new liner plate welded in place and space filled with high-strength grout
Hanford 2	50-397	1966	1973	Deficiency in vertical cadweld splice sleeves in reactor building mat
Catawba 2	50-414	UC[c]	1976	Cement used in reactor building base slab had been contaminated by fertilizer, 7-d strengths exceeded 28-d design values, cement feed transferred to another silo
Grand Gulf 1/2	50-416/417	1984/UC[c]	1975	Seven of 19 cylinders for control building base slab concrete did not meet 28-d design strength, 90-d values were acceptable
			1976	Voids found beneath drywell wall embed and shear key because of too stiff a concrete mix; holes drilled through embed and used to fill voids with high-strength grout; voids below shear key repaired by removing central portion of plate, chipping to good concrete, adding rebars, replacing concrete and liner, and leak testing liner
Bellefonte 1/2	50-438/439	UC[c]	1984	Expansion shell anchor failures occurred in control building concrete because of low surface concrete strength, anchors replaced by more deeply embedded bolts or grouted anchors
			1976[a]	Eight rock anchor heads failed during construction because of possible stress corrosion cracking, anchor heads replaced with cleaner steel
Seabrook 1/2	50-443/444	UC[c]/C[d]	1983	Cracking occurred in walls at end of stiffening slabs separating pump cells in category 1 service water and circulating water pumphouse, cause was shrinkage and temperature variations, stiffening slabs were modified
Commanche Peak 1/2	50-445/446	UC[c]	1975	Cold joint formed in reactor mat, concrete removed, rebars exposed and new joint poured
			1976	Voids 10 to 16 mm (3/8 to 5/8 in.) found under sump plates in concrete base mat, voids filled with neat cement grout using holes drilled through plates
			1976	Concrete not properly compacted around one of valve isolation embeds forming a void, faulty material removed by chipping and replaced by mortar or concrete
			1976	Inadequate concrete compaction under containment wall for 58 m (190 ft) at 1.8 to 2.1 m (6 to 7 ft) below top of mat, 3.7 by 6.1 m (12 by 20 ft) area south of reactor pit, 1.8 by 3.7 m (6 by 12 ft) area south of north sump and 1.2 by 1.8 m (4 by 6 ft) area north of north sump; core holes drilled for inspection in conjunction with analytical evaluations revealed base mat was adequate for all loading conditions; cores filled with mortar and interconnecting voids grouted
			1976	Excessive mortar used in concrete placement in preparing joint at reactor cavity wall, not determined to be structurally detrimental
			1976	Fresh concrete placed in area of standing water, because concrete forced water ahead of placement it was not considered detrimental, excess water removed
			1976	Hardened concrete observed splattered on rebars, extent of occurrence considered minor with bond reduction insignificant
Byron 1	50-454	1984	1979[a]	Four anchor head failures occurred in first year after stressing, cause was use of vanadium grain refinement process in conjunction with temperatures not high enough

Reactor plant	Docket No.	Year of commercial operation	Year of of occurrence	Summary description
Clinton	50-461	UC[c]	1984	Embed plate on outside of drywell wall pulled from concrete because of failure of several Nelson studs occurring as result of weld shrinkage, concrete excavated along plate edges, embed plate redesigned and grout placed into area where concrete was removed
Wolf Creek	50-482	1985	1978	Low concrete strength in reactor building base mat with some 90-d values below 28-d values, in-situ strength tests indicate concrete exceeded design values and low 90-d strengths were due to testing conditions
			1978	Voids up to 1.8 m (6 ft) wide and through-wall thickness occurred under equipment and personnel hatches in reactor containment building, voids repaired and quality assurance program updated
Callaway 1	50-483	1985	1977	Nineteen randomly located areas of honeycombing extending to bottom layers of rebar in reactor building base mat in annular area of tendon access area, cause was use of low slump concrete in congested area, defective material removed from 33 of 172 tendon trumplates and voids repaired
South Texas 1/2	50-498/499	UC[c]	1977	Crack in fuel handling building wall due to shrinkage, no structural significance
			1977	Rebars improperly located in buttress region of unit 1 containment, detailed analysis of as-built condition determined that no safety hazard to public occurred
			1978	Unconsolidated areas occurred in bottom surface of concrete slab in south unit 1 fuel-handling building; material removed by chipping to expose rebars, surface was epoxy-sealed followed by epoxy injection and a combination of dry packing, shotcreting, and epoxy injection
			1978	Voids occurred behind liner plate of unit 1 reactor containment building exterior wall because of planning deficiencies, long pour time and several pump breakdowns; sounding and fiberoptic exam through holes drilled in liner plate were used to determine extent, areas were repaired by grout injection
			1979	Voids were detected in 12 areas behind liner plate of reactor containment building exterior wall with cause being attributed to temporary weldments, normal concrete settlement/shrinkage, and liner movement; construction and quality control procedures strengthened
			1983	Rust and pitting were observed on tendons for units 1 and 2 while in storage at fabricating plant, cause was delayed and improper corrosion inhibitor application and storage in a facility without temperature and humidity control; detailed exam of 14 tendons revealed pitting up to >0.4 mm (15 mils) but strength and ductility exceeded limits; damaged tendons were replaced and controlled storage conditions utilized with properly applied corrosion inhibitor
Palo Verde 2/3	50-529/530	UC[c]	1984	Honeycombing around vertical tendon sheath blockouts with most voids at buttress/shell interface above last dome hoop tendon, condition was localized so area repaired

Reactor plant	Docket No.	Year of commercial operation	Year of of occurrence	Summary description
Marble Hill	50-546	C[d]	1979	High concrete pour rate may have bowed liner
			1979	A 0.3-m-deep (12-in.) void extending 6.1 × 1.4 m (20 × 4.5 ft) in axial direction in base slab for auxiliary building, void repaired by shotcrete injection
			1979[a]	Numerous surface defects (~4000) and inadequate patching resulting from poor concrete compaction and improperly prepared construction joints; breakdown in quality control and construction management attributed as cause; internal concrete inspection revealed it to be of high quality with higher than required strength; patches removed and replaced using good construction practices; providing good workmanship is used in repair and procedures followed, consultants determined structural integrity and shielding requirements should be met

[a]Described in more detail in Sect. 3.3.2.

[b]Described in more detail in Sect. 3.2.2.

[c]Under construction.

[d]Cancelled or indefinitely deferred.

NUREG/CR-4652
ORNL/TM-10059
Dist. Category RV

Internal Distribution

1. J. M. Corum
2. W. G. Craddick
3. L. R. Dole
4-6. D. M. Eissenberg
7. J. A. Getsi
8. W. L. Greenstreet
9. H. D. Haynes
10. J. E. Jones Jr.
11. P. R. Kasten
12. R. C. Kryter
13. A. P. Malinauskas
14. G. C. Murphy
15. F. R. Mynatt
16-19. D. J. Naus
20. C. B. Oland
21. C. E. Pugh
22. G. C. Robinson
23. H. E. Trammell
24. D. B. Trauger
25. ORNL Patent Office
26. Central Research Library
27. Document Reference Section
28-29. Laboratory Records Department
30. Laboratory Records (RC)

External Distribution

31. Gunter Arndt, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, 5650 Nicholson Lane, Rockville, MD 20852
32. Hans Ashar, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, 5650 Nicholson Lane, Rockville, MD 20852
33. A. B. Johnson, Pacific Northwest Laboratory, P.O. Box 999, Richland, WA 99352
34. Don Lamken, Commonwealth Edison Company, One First National Plaza, P.O. Box 767 (Room 35FNW), Chicago, IL 60690
35. Alexander Marion, Rm. 920, Baltimore Gas and Electric Co., P.O. Box 1475, Gas and Electric Building, Baltimore, MD 21203
36. J. W. McElroy, Philadelphia Electric Co., P.O. Box 8699, Philadelphia, PA 19101
37. C. Michelson, ACRS, 20 Argonne Plaza, Suite 365, Oak Ridge, TN 37830
38. Jerry E. Rhoads, Washington Public Power System, 3000 George Washington Way, P.O. Box 968, Richland, WA 99352
39. R. L. Simard, Institute for Nuclear Power Operations, 1100 Circle 75 Parkway, Atlanta, GA 30339
40. George Sliter, Nuclear Engineering and Operations Department, Electric Power Research Institute, 3412 Hillview Avenue, P.O. Box 10412, Palo Alto, CA 94303
41. D. F. Sullivan, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, 5650 Nicholson Lane, Rockville, MD 20852
42. J. H. Taylor, Engineering and Risk Assessment Division, Department of Nuclear Energy, Brookhaven National Laboratory, Upton, NY 11973

43. Jim E. Thomas, Duke Power Company, P.O. Box 33189, 422 South Church Street, Charlotte, NC 28242
44. Charles Turk, Arkansas Power and Light Co., P.O. Box 551, Little Rock, AR 72203
45. Stanley Urbanowski, Yankee Atomic Electric Co., 1671 Worcester Road, Farmington, MA 01701
46. J. P. Vora, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, 5650 Nicholson Lane, Rockville, MD 20852
47. Don R. Woodlan, Licensing Engineer, Texas Utilities Generating Co., 400 North Olive Street, LB81, Dallas, TX 75201
48. Office of Assistant Manager for Energy Research and Development, Department of Energy, Oak Ridge Operations Office, Oak Ridge, TN 37831

49-50. Technical Information Center, Department of Energy, Oak Ridge, TN 37831

51-380. Given distribution as shown in NRC category RV (10 — NTIS)

NRC FORM 335
(2-84)
NRCM 1102,
3201, 3202

U.S. NUCLEAR REGULATORY COMMISSION

BIBLIOGRAPHIC DATA SHEET

SEE INSTRUCTIONS ON THE REVERSE

1 REPORT NUMBER (Assigned by TIDC, add Vol No., if any)
NUREG/CR-4652
ORNL/TM-10059

2 TITLE AND SUBTITLE
Aging of Concrete Components and Its Significance Relative to Life Extension of Nuclear Power Plants

3 LEAVE BLANK

4 DATE REPORT COMPLETED
MONTH: August YEAR: 1986

5 AUTHOR(S)
D. J. Naus

6 DATE REPORT ISSUED
MONTH: September YEAR: 1986

7. PERFORMING ORGANIZATION NAME AND MAILING ADDRESS (Include Zip Code)
Oak Ridge National Laboratory
P.O. Box X
Oak Ridge, Tennessee 37831

8. PROJECT/TASK/WORK UNIT NUMBER

9 FIN OR GRANT NUMBER
B0828

10. SPONSORING ORGANIZATION NAME AND MAILING ADDRESS (Include Zip Code)
U.S. Nuclear Regulatory Commission
Office of Nuclear Regulatory Research
Division of Engineering Technology
Washington, DC 20555

11a. TYPE OF REPORT
Topical

b. PERIOD COVERED (Inclusive dates)

12 SUPPLEMENTARY NOTES

13 ABSTRACT (200 words or less)

The objective of the study was to: (1) expand upon the work which was initiated in the first two EPRI studies relative to longevity and life extension considerations of safety-related concrete components in LWR facilities, and (2) develop background which will logically lead to subsequent development of a methodology for assessing and predicting the effects of aging on the performance of concrete-based materials and components. These objectives are consistent with NPAR Program Goals:† (1) to identify and characterize aging and service wear effects which, if unchecked, could cause degradation of structures, components, and systems and thereby impair plant safety; (2) to identify methods of inspection, surveillance and monitoring, or of evaluating residual life of structures, components, and systems, which will assure timely detection of significant aging effects prior to loss of safety function; and (3) to evaluate the effectiveness of storage, maintenance, repair and replacement practices in mitigating the rate and extent of degradation caused by aging and service wear.

†B. M. Morris and J. P. Vora, "Nuclear Plant Aging Research (NPAR) Program Plan," NUREG-1144, Division of Engineering Technology, Office of Nuclear Regulatory Research, U.S. Nuclear Regulatory Commission, Washington, DC (July 1985).

14 DOCUMENT ANALYSIS – a KEYWORDS/DESCRIPTORS
Aging
Concrete
Evaluation Methodology
In-service Inspection
Repair

b IDENTIFIERS/OPEN ENDED TERMS

15 AVAILABILITY STATEMENT
Unlimited

16 SECURITY CLASSIFICATION
(This page) Unclassified
(This report) Unclassified

17 NUMBER OF PAGES

18 PRICE

☆U.S. GOVERNMENT PRINTING OFFICE 1986—631-056/40043