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**REPAIR MATERIALS AND TECHNIQUES FOR CONCRETE
STRUCTURES IN NUCLEAR POWER PLANTS**

P. D. Krauss

**Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, Illinois 60062-2095**

March 1994

**ORNL Program Manager: D. J. Naus
Work Performed under ORNL Contract No. 90X-SJ024C**

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

**Prepared for the
U.S. Nuclear Regulatory Commission
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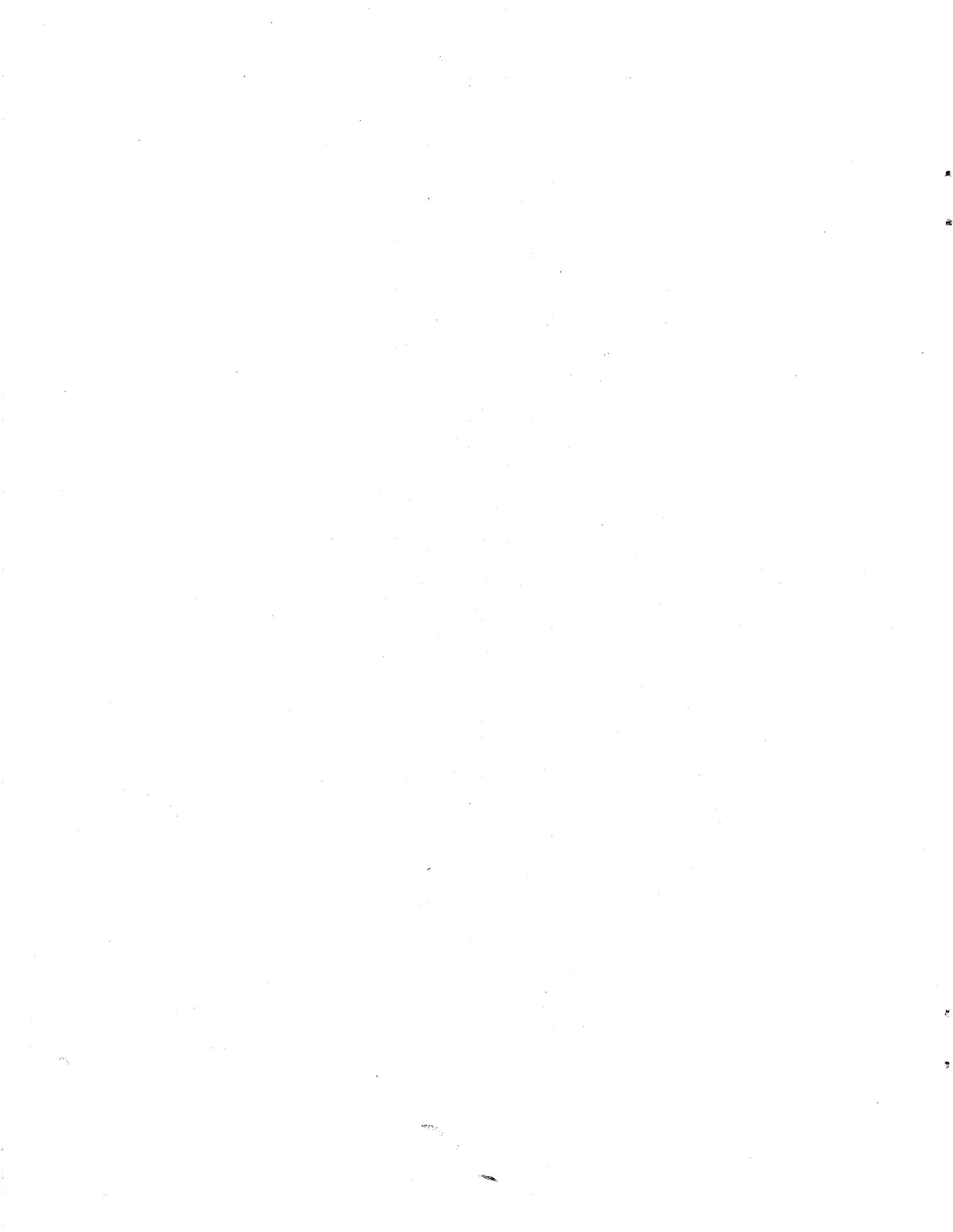


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REPAIR MATERIALS AND TECHNIQUES FOR CONCRETE STRUCTURES IN NUCLEAR POWER PLANTS

P. D. Krauss

EXECUTIVE SUMMARY

The Structural Aging Program, sponsored by the Nuclear Regulatory Commission (NRC) has the overall objective of developing an improved basis for evaluating nuclear power plant structures as they age. The work conducted and managed by the Oak Ridge National Laboratory includes the development of a structural materials property data base, establishment of structural component assessment and repair methodologies, and the formulation of a methodology for continued service determinations. This report addresses the assessment and repair of concrete structures in nuclear power plants.

This report is divided into six chapters. Chapter 1 is an introduction and discusses the type of concrete structures found in nuclear power plants. A brief discussion of safety related structures is included. Chapter 2 presents the results of a survey of the nuclear power plants in the United States. The survey was performed to identify the concrete components most susceptible to durability problems, to determine the rate of occurrence of deterioration, and to determine the durability of repairs. Cracking due to concrete shrinkage was cited as the most common type of distress. The causes of deterioration were similar to what is commonly experienced in most civil engineering structures except acid and chemical attack is much more common problem in nuclear facilities. Most plants reported repairing some concrete damage. The repairs have generally been minor and of little significance. Very few plants had information on the performance of the repairs or estimates of the service life of repair materials. Most plants do not perform routine inspections of the concrete structures. Procedures for performing condition surveys are discussed along with specific techniques to evaluate concrete structures with emphasis on nondestructive testing techniques.

Chapter 3 describes the various deterioration mechanisms that can effect concrete structures in nuclear facilities and discusses the effect of each deterioration mechanism. This section is separated into deterioration as a result of short-term and long-term exposures. Deterioration modes are further separated into exposure to physical or chemical phenomena. Emphasis is placed on mechanisms that affect the long-term durability of concrete, such as corrosion of embedded metals and alkali-reactive aggregates.

Repair techniques are described in Chapter 4. Practical experience is used to compare the various repair options as an aid in selection of the proper materials and techniques. Emphasis on the proper installation of repairs is important for durability. Concrete manuals and databases are referenced to provide the practicing engineer with sources for more information on concrete deterioration, repair techniques, and materials.

Chapter 5 discusses specific evaluation techniques and nondestructive test techniques. Many of the techniques require specialized equipment and data interpretation by experienced personnel. Case studies using various nondestructive techniques on structures are described.

The final chapter, Chapter 6, is a brief summary of the report. Most concrete structures associated with nuclear power plants have had good durability and few problems. This may be due to the young age of the structures and stringent quality control during construction. As these structures age it is important that deterioration is accurately identified and that corrective action is taken for continued service. Assumptions and myths associated with concrete durability should be carefully avoided. Routine inspection procedures and programs should be developed to assess the condition of the concrete structures and to acquire data to predict deterioration rates of concrete members.

ACKNOWLEDGEMENTS

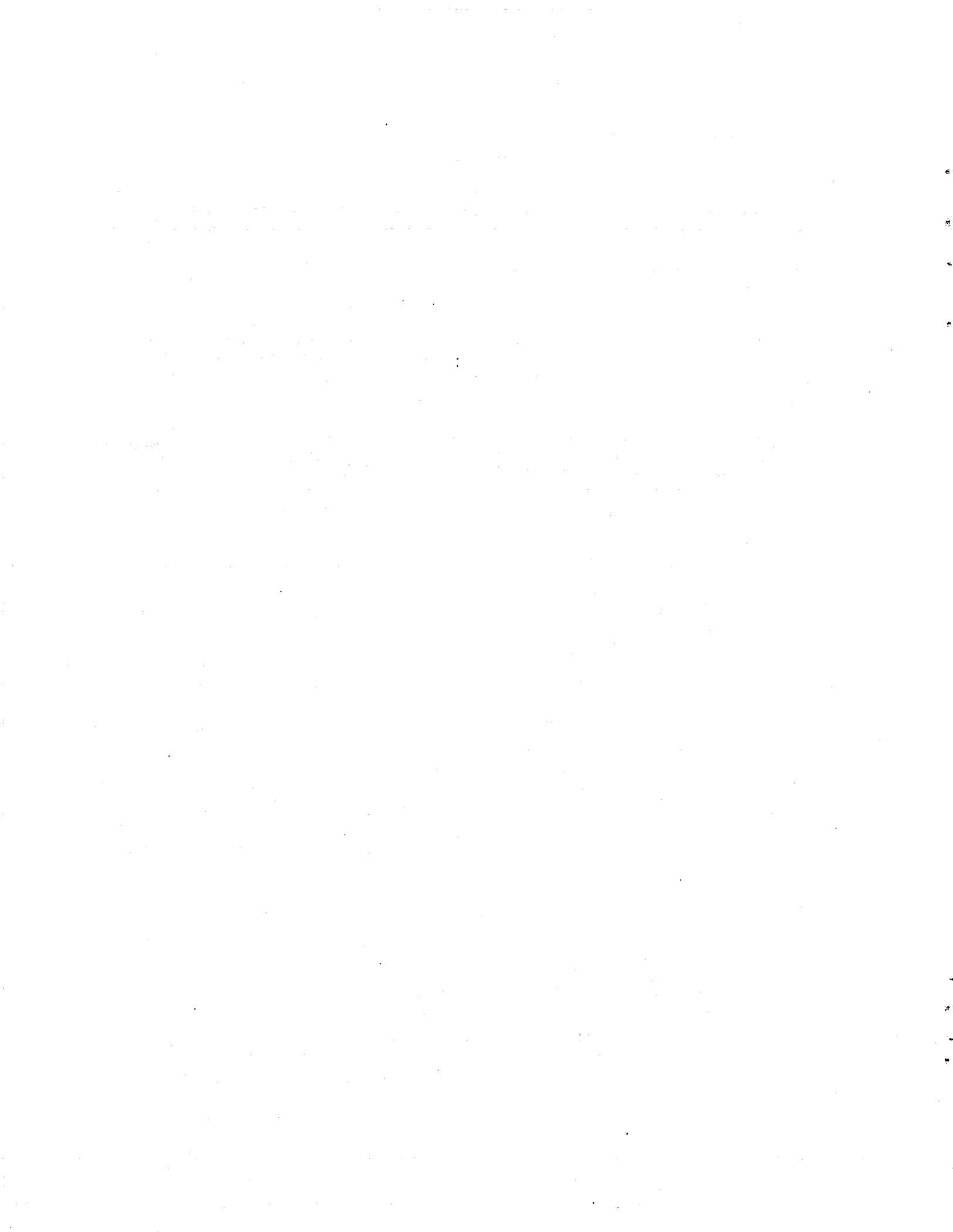
Technical and editorial support was received from many Wiss, Janney, Elstner Associates, Inc. staff members, including significant efforts by John M. Scanlon, William F. Perenchio, David M. McDonald, Thomas J. Rowe, Hemant Lymaye, and Donald W. Pfeifer. Acknowledgment and thanks go to all nuclear plant personnel and consultants who helped complete and return the plant surveys.

ABSTRACT

This report discusses deterioration and repair of concrete structures in nuclear power facilities. A survey of nuclear power facilities was performed to identify types of deterioration, locations of deterioration, and repair procedures. It was found that most plants do not have a routine inspection program for concrete structures. The deterioration has generally been minor and is often associated with construction problems. Little information was available on the performance or durability of repairs.

Concrete deterioration is discussed with special emphasis on deterioration associated with aging effects. A methodology for performing condition surveys is also presented. Much of the discussion on deterioration is useful for engineers building new structures as an aid to avoiding future problems. Evaluation techniques with emphasis on nondestructive methods are presented with a discussion of several recent investigations of large concrete structures.

Repair methods and materials are discussed. Little information was available from the plant survey or literature review on the durability of the various repair materials or techniques. Conflicting information was sometimes reported. Based on the literature and experience, the durability of a repair is mainly related to material selection and workmanship during installation. Discussion of common errors and misconceptions related to repair of concrete is presented.



1. INTRODUCTION

Many of the nuclear structures in the United States are over 20 years of age. As these power plants continue to age, there is an increased potential need for sound, proven technical recommendations on the repair of their reinforced and prestressed concrete structures. Within a nuclear power plant there are many different types of structures, each of which may require repair procedures that are appropriate for its specific function.

1.1 Objective and Approach

The objective of this report is to identify, describe and quantify the effectiveness of various techniques and materials that are available for use in repair of aged, deteriorating concrete structures associated with nuclear power plants. The approach to this work includes:

1. Identification and description of the various types of distress that afflict concrete structures, as well as the cause of this distress are presented. Threshold values and performance levels have been included wherever possible.
2. Identification and description of available techniques and materials for the repair, replacement or retrofitting of concrete structures are discussed. A discussion of criteria for the application of a particular technique or material is included. Known constraints for each technique are presented. The life expectancy and, anticipated durability of repairs are discussed when credible information is available. Description of alternate techniques and historical performance data are presented when available. Techniques to reduce the aging effects of concrete are also discussed.
3. Discussion of techniques to evaluate the effectiveness of the repair as well as methods to monitor long-term performance is provided. Generally, the problems observed with deteriorated concrete in nuclear power plant structures are similar to problems associated with structures found in other industries. Since nuclear power plants were constructed to strict specifications and most are still relatively new structures, minimal repair has been necessary. Therefore, the repair and repair performance of other common concrete structures was also discussed.

This report is directed toward the state-of-the-art concerning repair of aged and deteriorated concrete structures in Light Water Reactor (LWR) nuclear power plants and includes:

- types of distress normally found in concrete structures
- cause of this distress
- typical deterioration observed due to the distress
- selection of repair techniques
- discussion of expected durability of repair techniques
- potential problems and benefits anticipated from particular repair techniques
- discussion of case studies and common problems
- evaluation of repairs using various test methods

The deterioration of concrete structures is particularly accelerated when heat and moisture are readily available. This has been repeatedly observed with most mechanisms including corrosion, alkali-silica reactivity, and sulfate attack. While nuclear plant structures contain many indoor structures and members which are protected from rain water, these same structures and members are generally thick and in many cases utilize steel liner plates which prevent drying from one face. As a result, many members maintain high water contents and relative humidities for years, even decades. These high internal humidities can supply the necessary moisture to sustain numerous deterioration

mechanisms. Another unique potential accelerating factor in these plants is the environmental condition where a concrete member is sustained at high temperatures above ambient conditions. These high concrete temperatures when combined with high internal humidities could accelerate deterioration. Therefore, condition surveys should certainly focus extra attention on large members exposed to elevated temperatures.

This report has been produced based upon in-house expertise available at Wiss Janney Elstner Associates, Inc. (WJE), telephone interviews with engineers and contractors having experience in repairing and maintaining nuclear plants, review of reports on repairs at nuclear plants, an extensive literature review, and a formal nuclear power plant survey.

While this report provides general guidelines on repair and durability, such a document cannot replace a skilled engineer or materials technologist with experience in the detection and repair of defects in concrete structures. This report contains sections on condition surveys and producing durable concrete as they relate to nuclear power plants. Determination of expected service life for repairs is extremely difficult, as the life-spans depend not only on the quality of materials, but on workmanship and the environmental exposure conditions.

1.2 Structures and Safety Ratings

The concrete structures may be conveniently grouped into four main categories for discussion: reactor containment buildings, containment base mats, biological shield walls and buildings, and auxiliary buildings.

Reactor containment building — The containment building is one of the most important structures since it serves as a final barrier against a release of radioactive products under postulated accident conditions. Its design is based upon pressure and temperature loadings associated with a loss of coolant accident resulting from a double-ended rupture of the largest pipe in the coolant system. It is also designed to tolerate low probability environmental loadings. Early plants used steel containments. As reactors increased in size, composite steel-lined reinforced concrete structures were used. Fully-prestressed concrete containments were first built in the late 1960s.

Containment base mats — Most base mats are constructed of conventionally reinforced concrete. The design of the base mats is complex, site specific, and based on particular concrete properties and loading stresses.

Biological shield walls and buildings — Biological shield walls are typically fabricated as normal weight reinforced concrete and surround the primary steel containment vessel. The shield walls are designed to shield parts of the reactor coolant system from accident conditions, and to protect the containment vessel from atmospheric conditions and external missiles. The shield building, or secondary containment, is a medium leakage reinforced concrete structure that is designed to provide biological shielding from accident conditions; biological shielding from parts of the reactor coolant system during operation; and protection of the containment vessel from low temperatures, adverse atmospheric conditions, and external missiles.

Auxiliary buildings — Auxiliary buildings house the control room, spent-fuel pit, fuel handling, radioactive waste or waste management and protect the units and piping from environmental stresses. These structures are commonly box-shaped shear-wall buildings constructed of reinforced concrete. They often contain exterior and interior walls, foundation slabs, roof slab, floor slabs and columns. Their design considers such unusual events as tornadoes and earthquakes. The walls are typically 0.3 to 1.2 m (1 to 4 ft) thick and are constructed of normal weight concrete or heavyweight concrete.

Details of a safety rating system for each element are outlined in a document entitled *Structural Aging Assessment Methodology For Concrete Structures In Nuclear Power Plants*, by Hookham.^(1.1) Appendix B of this document presents examples of application of the methodology to three light-water reactor plants. The primary structure, its function and its cumulative rank, are listed. The environmental exposure and safety significance are ranked and key degradation factors are listed.

Figures 1.1 and 1.2 show the design of typical reactor containment structures.^(1.1) Table 1.1 lists typical safety-related concrete structures in LWR plants.^(1.1) Each of these structures is subjected to different environments and loadings; repair techniques must be appropriate for each type. For this reason, the function of each part of a structure must be understood in order for repairs to be successful. Some elements may be designed purely on structural capacity, while others on structural capacity and exposure conditions.

Tables 1.2 and 1.3 show the sub-element division of Category I concrete structures and their relative safety importance factor.^(1.1) Table 1.2 contains information relating to boiling water reactors (BWR), while Table 1.3 shows information relating to pressurized water reactors (PWR). These tables rate the sub-elements, depending on their structural contribution to the primary structure, on a scale from 0 to 10. The safety significance defined by Hookham^(1.1) is shown on Table 1.4. A structure with a safety significance ranking of 10 is considered to be of greatest importance to the overall safety. Whereas a structure with a ranking of 0 is considered to be non-safety related.

The primary structure is divided into sub-elements that are given various factors, such as:

- the structural importance of an element
- environmental exposure surrounding an element
- safety significance to the overall structural performance of the plant

The effects of deterioration of structures are many, but they will fall into the categories of structural safety, containment (either fluid or radiation), personnel safety, or aesthetics. These four factors of deterioration can be combined and used in describing the acceptability limit for any element in the structure.

Severe deterioration to either concrete or reinforcing steel may eventually result in unacceptable problems, or even structural collapse of a member or structural system. The deterioration may be a result of several interrelated factors. The acceptability limit of distress, and the repair, must be based upon the effect of failure of each. It is much more critical to prevent a supporting column from failing; for instance, than it is to protect a non-load-bearing element.

A nuclear plant contains many structures that retain water, hazardous nuclear products, or other potentially deleterious liquids. In liquid-containing structures, inadequate joints and cracks in the concrete are typically responsible for performance concerns or problems. If liquid confinement is considered the major factor for the element's satisfactory performance, then acceptability must be based largely upon leakage.

Reduced personnel safety may be a direct result of the concrete deterioration. Employee injuries may be caused by falling material, leakage of hazardous materials, or tripping and falling over degraded materials. The acceptability limit for personnel injury must also be considered in any repair or maintenance program.

At commissioning, the condition of the structural elements is well above the unacceptable deteriorated condition level. As the structure ages, it may deteriorate and thus its level of service may approach an unacceptable level. As shown in Fig. 1.3, when subjected to aging, the period of

time when a structure with a high importance, Structure 1, should be assessed and classed as unacceptable should be far shorter than that for a structure with moderate importance, Structure 2, or low importance, Structure 3. Due to its importance, it is necessary to repair Structure 1 before it reaches a moderately deteriorated condition. The effects of three repair procedures, (A, B and C) on the life-span of Structure 1 are shown in Fig. 1.3. Repair A may be less expensive than repairs B or C, but it does not extend the life span of the structure for as long as the other repair methods.

At the time of de-commissioning, it would be cost effective if all the elements in the plant would be close to their limit of acceptability. Thus, repairs on Structure 3 may be either undertaken at a later age, or completed using less expensive repair methods (such as, Repairs A or B) than those chosen for Structure 1 or 2. If the age of de-commissioning is known, repair methods can be selected to ensure the structures are acceptable until that time. Experience shows that conservative estimates should always be used.

1.3 References

- 1.1 Hookham, C. J., *Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants*, U. S. Nuclear Regulatory Commission, NRC FIN No. B0845, March 1991.

Table 1.1 — Typical Safety-Related Concrete Structure in LWR Plants^(1.1)

Primary Containment

Containment dome/roof
Containment foundation/basemat
Slabs and walls

Containment Internal Structures

Slabs and walls
Reactor vessel support structure (or pedestal)
Crane support structures
Reactor shield wall (biological)
Ice condenser dividing wall (ice condenser plants)
NSSS equipment supports/vault structures
Weir and vent walls (Mark III)
Pool structures (Mark III)
Diaphragm floor (Mark II)
Drywall/wetwell slabs and walls (Mark III)

Secondary Containment/Reactor Buildings

Slabs, columns, and walls
Foundation
Sacrificial shield wall (metallic containments)

Fuel/Equipment Storage Pools

Walls, slabs, and canals

Auxiliary building
Fuel storage building
Control room (or building)
Diesel generator building
Piping or electrical cable ducts or tunnels
Radioactive waste storage building
Stacks
In-take structures (inc. concrete water in-take piping and canal embankments)
Pumping stations
Cooling towers
Plant discharge structures
Emergency cooling water structures
Dams
Water wells
Turbine building

**Table 1.2 — Sub-Element Division of Category I Concrete Structures:
Boiling Water Reactor^(1.1)**

		<u>Importance Factor</u>
A. <u>Primary Containment</u>		
<u>Concrete Containment</u>		
1.	Basemat Foundation	10
2.	Drywell Pedestal	10
3.	Vertical Walls (Mark I)	8
4.	Steel Liner	4
5.	Suppression Chamber (Mark I)	8
6.	Chamber Steel Liner (Mark I)	4
7.	Vertical Walls (Mark II)	8
8.	Vertical Walls (Truncated Cone - Mark II)	8
9.	Concrete Dome (Mark III)	8
10.	Polar Crane Support (Mark III)	10
 <u>Steel Containment</u>		
1.	Basemat Foundation	10
 B. <u>Containment Internal Structures</u>		
1.	Bottom Slab (Steel Mark I and Pre-Mark Containments)	10
2.	Reactor Pedestal/Support Structure	9
3.	Biological (Reactor) Shield Wall	6
4.	Floor Slabs	6
5.	Walls	7
6.	Columns	7
7.	Diaphragm Floor (Mark II)	7
8.	NSSS Equipment Pedestals/Support	5
9.	Upper and Fuel Pool Slabs (Mark III)	5
10.	Drywell Wall (Mark III)	6
11.	Weir/Vent Wall (Mark III)	7
12.	Crane Support Structure (Mark III)	5

Table 1.2 (Cont'd)

	<u>Importance Factor</u>
C. <u>Secondary Containments/Reactor Buildings</u>	
1. Basemat Foundation (if isolated from containment basemat)	10
2. Walls	8
3. Slabs	6
4. Columns	7
5. Equipment Supports/Pedestals	4
6. Sacrificial Shield Wall (Metal Containments)	6
7. Spent/New Fuel Pool Walls/Slabs	6
8. Drywell Foundation (Mark I)	9
D. <u>Other Structures (Category I)</u>	
1. Foundations*	10
2. Walls*	8
3. Slabs*	6
4. Cable Ducts	5
5. Pipe Tunnels	5
6. Stacks	5
7. Concrete In-take Piping	6
8. Cooling Tower Basins	5
9. Dams	6
10. In-take Crib Structures	6
11. Embankments	4
12. Tanks	5
13. Water Wells	4

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

**Table 1.3 — Sub-Element Division of Category I Concrete Structures:
Pressurized Water Reactor^(1.1)**

	<u>Importance Factor</u>
A. <u>Primary Containment</u>	
<u>Concrete Containment</u>	
1. Basemat Foundation	10
2. Tendon Access Galleries	3
3. Vertical Walls (and Buttresses)	8
4. Ring Girder (Prestressed Concrete Containment Vessel)	9
5. Dome	8
<u>Steel Containment</u>	
1. Basemat Foundation	10
B. <u>Containment Internal Structures</u>	
1. Bottom Floor (Metal Containments)	10
2. Floor Slabs	6
3. Walls	7
4. Columns	6
5. NSSS Equipment Pedestals/Supports	5
6. Primary Shield Wall (Reactor Cavity)	8
7. Reactor Coolant Vault Walls	7
8. Beams	5
9. Crane Support Structures	4
10. Ice Condenser Divider Wall and Slab	5
11. Refueling Pool and Canal Walls	6
C. <u>Secondary Containment Buildings (metal containments)</u>	
1. Foundation	10
2. Walls	8
3. Slabs	6

Table 1.3 (Cont'd)

		<u>Importance Factor</u>
D.	<u>Other Structures (Category I)</u>	
	1. Foundations*	10
	2. Walls*	8
	3. Slabs*	6
	4. Cable Ducts	5
	5. Pipe Tunnels	5
	6. Stacks	5
	7. Concrete In-take Piping	6
	8. Hyperbolic Cooling Towers	5
	9. Dams	6
	10. In-take Crib Structures	6
	11. Embankments	4
	12. Tanks	5
	13. Water Wells	4

* Components of other site buildings such as Auxiliary, Turbine, Control, and Diesel Generator Buildings

Table 1.4 — Safety Significance Ranking Criteria^(1.1)

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

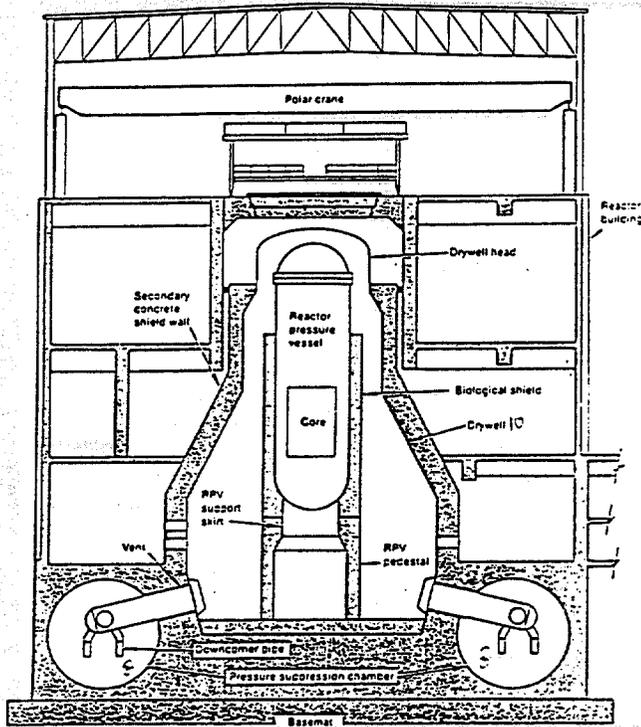


Fig. 2.1. BWR Mark I type metal containment enclosed in a reactor building. Source: *Residual Life Assessment of Major Light Water Reactor Components - Overview*, NUREG/CR-4731, Vol. 2, EG&G Idaho, Inc., Idaho National Engineering Laboratory, Idaho Falls, November 1989.

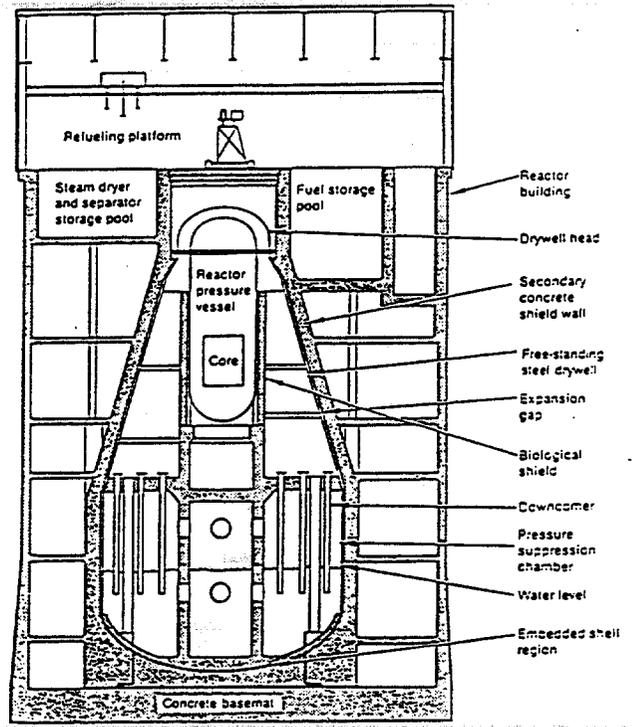


Fig. 2.2. BWR Mark II type metal containment enclosed in a reactor building. Source: *Residual Life Assessment of Major Light Water Reactor Components - Overview*, NUREG/CR-4731, Vol. 2, EG&G Idaho, Inc., Idaho National Engineering Laboratory, Idaho Falls, November 1989.

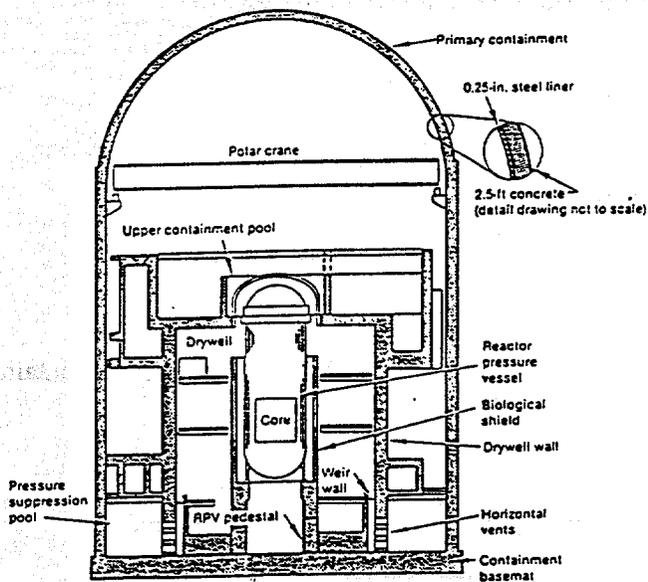


Fig. 2.3. BWR Mark III type reinforced concrete containment. Source: *Residual Life Assessment of Major Light Water Reactor Components - Overview*, NUREG/CR-4731, Vol. 2, EG&G Idaho, Inc., Idaho National Engineering Laboratory, Idaho Falls, November 1989.

Fig. 1.1 - Typical BWR containments^(1,1)

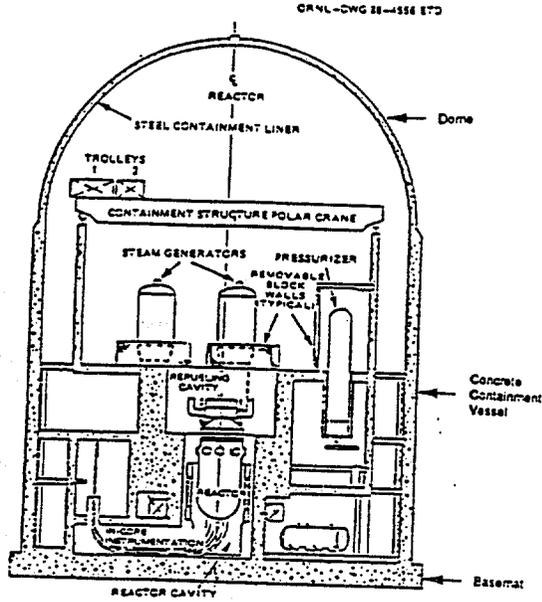


Fig. 2.4. PWR subatmospheric type reinforced concrete containment. Source: *Containment Performance Working Group Report, Draft Report for Comment, NUREG-1037, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.*

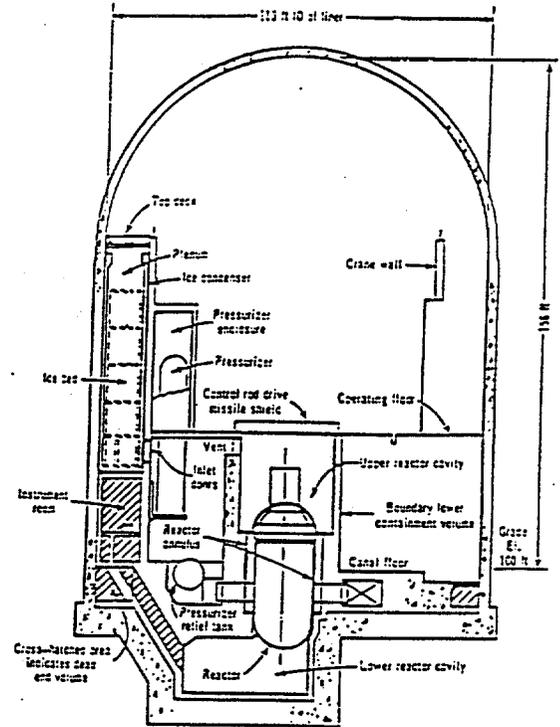


Fig. 2.5. PWR ice condenser type reinforced containment. Source: *"Structural Analysis and Design of Nuclear Plant Facilities," ASCE Manual and Report on Engineering Practice - No. 53, 1980.*

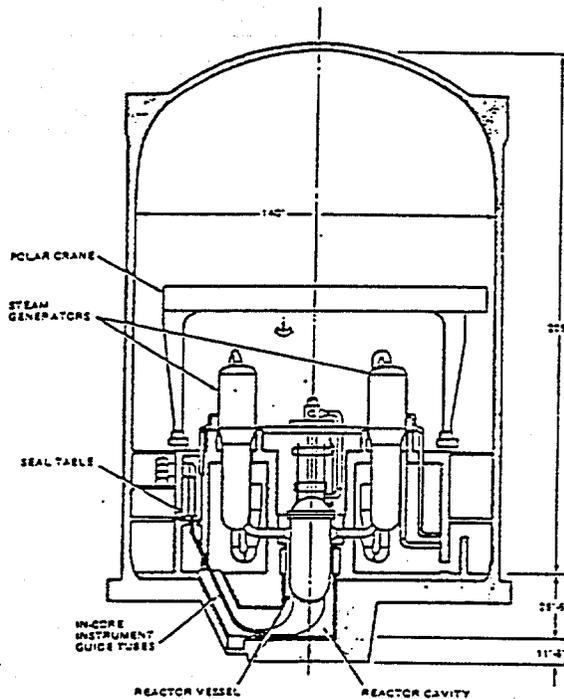


Fig. 2.6. PWR large dry type prestressed concrete containment. Source: *Containment Performance Working Group Report, Draft Report for Comment, NUREG-1037, U.S. Nuclear Regulatory Commission, Washington, D.C., May 1985.*

Fig. 1.2 - Typical PWR containments^(1.1)

DETERIORATION OF CONCRETE STRUCTURES

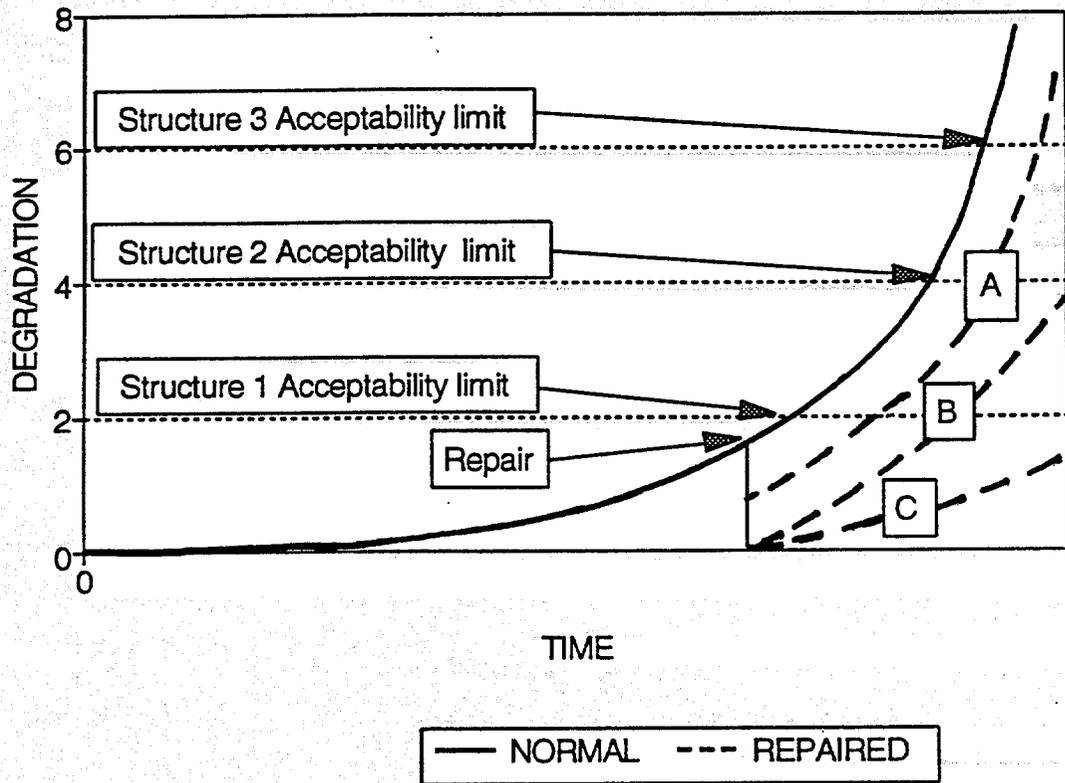


Fig. 1.3 - Deterioration time and acceptability limits for structures of different classifications

2. RESULTS OF NUCLEAR PLANT SURVEY

2.1 Survey Responses

A survey was developed by WJE and the U. S. Nuclear Regulatory Commission (NRC) to solicit information on the locations and types of concrete distress commonly found in nuclear power plants. The survey was distributed by the NRC to each of the plants in the United States. The survey included questions concerning the types of repairs performed and requested information or reports on the durability and service life of these repairs. A map of the United States showing the plants which responded to the survey is shown in Fig. 2.1. Twenty-nine plants returned surveys and the surveys represent 41 reactor units. A wide geographical distribution of the United States was represented by the completed surveys.

The dates of reactor commissioning ranged from 1962 to 1988. The types of containment varied as follows:

<u>Number of reactors</u>	<u>Containment type</u>
21	Concrete cylinder
8	Steel cylinder or sphere
5	Mark I
2	Mark II
2	Mark III
3	Other

The majority of the plants do not have a routine inspection program for the concrete components of the plant. NRC Information Notice No. 89-79^(2.1) requires that a general visual inspection be performed on the interior of the containment before each integrated leak rate test. The purpose of the inspection is to identify any evidence of structural deterioration that may affect the leak tight integrity of the containment. Other plant inspections of concrete structures in nuclear power plants are associated with post-tensioning system surveillance of the reactor containment buildings. Visual inspection of the anchorage concrete and a general review of the containment concrete are generally performed. Other than these limited inspections, regular inspections of concrete structures are not typically performed by the plants.

Twelve of the 29 plants conduct some inspections of concrete structures. The intervals of inspections vary from annual to 5-year intervals. Twelve plants reported using outside consultants for periodic inspection of concrete structures. Ten plants use outside consultants for the repair of deterioration. All plants conducting inspections rely mainly on visual inspection methods. Five plants reported that they periodically remove core samples, four plants reported performing chemical analyses on the concrete, and three plants reported making half-cell potential measurements. The use of other nondestructive techniques such as pulse velocity, pulse echo, impact echo or acoustic emission techniques was reported by two plants. Nondestructive tests such as linear polarization, AC impedance, strain instrumentation, X-ray analysis, rebound hammer and penetration resistance were mentioned as having been used by individual plants.

2.2 Deterioration Types

Twenty-six of the 29 plants reported that they experienced some type of damage or deterioration to the concrete in the plant. Cracking was the most common type of deterioration, with 25 plants reporting cracking situations. A summary of the reported deterioration is shown in the following table.

<u>Deterioration type</u>	<u>Number of plants</u>	<u>%</u>
Cracking	25	86
Spalling	19	65
Staining	16	55
Honeycombing	15	51
Reinforcing steel corrosion	10	34
Efflorescence	10	34
Popouts	8	28
Scaling	7	24
Delaminations	7	24

Most of the honeycombing problems were due to initial construction problems and have been repaired. Many of the delaminations reported were located in concrete domes. Other forms of deterioration reported by single plants were: damage due to ice buildup in the cooling tower fill, dusting, degradation of the prestressing steel, deterioration of grout, and staining of concrete due to leakage of tendon grease.

Drying shrinkage of the concrete and acid attack were the most common causes of concrete deterioration. A summary of the reported causes of deterioration is shown in the following table.

<u>Cause of deterioration</u>	<u>Number of plants</u>	<u>%</u>
Drying shrinkage	14	48
Acid attack	12	41
Thermal movement	10	34
Chemical attack	9	31
Impact	9	31
Cyclic freezing	8	28
Sea water exposure	7	24
Embedded steel corrosion	7	24
Plastic shrinkage	7	24
Abrasion	4	14
Temperature or fire	3	10
Sulfate attack	2	7
Structural overloads	2	7

Carbonation, corrosion of embedded metals other than reinforcing steel, vibration fatigue, joint spalling, and chemical reactions of aggregates were all reported by individual plants. None of the plants reported concrete damage due to radiation exposure. Deterioration of metallic aggregate in a proprietary grout was also reported.

2.3 Location of Deterioration

Of the respondents, 19 plants contained pressure water reactors, and 10 plants contained boiling water reactors. The survey requested information on the location of deterioration specific to the type of plant, and the results are discussed in the following sections. The importance factor (IF) assigned by Hookham^(1.1) is shown for each of the plant sub-elements, according to Table 1.3.

2.3.1 Pressurized Water Reactors

The most common locations of deterioration in the 19 PWR plants were the containment dome and in the walls and slabs of auxiliary structures. The following table shows the occurrence of deterioration in various sub-elements within PWR plants.

Pressure Water Reactors

<u>Location of deterioration</u>	<u>Number of plants</u>	<u>%</u>	<u>Importance factor (IF)</u>
Auxiliary structure walls	11	58	8
Auxiliary structure slabs	9	47	6
Containment dome	7	37	8
In-take crib structures	5	26	6
Auxiliary structure foundations	5	26	10
Concrete in-take piping	5	26	4
Basement foundation	4	21	10
Vertical walls and buttresses	4	21	8
Cooling towers	4	21	5
Containment floor	3	16	10
Containment walls	3	16	7
Pipe tunnels	3	16	5

Two plants or less reported problems in the following areas: tendon access galleries (2 plants, IF3), ring girder (2,IF9), liner (2,IF5), containment internal structures, such as, floor slabs (2,IF6), columns (1,IF6), NSSS equipment pedestals and supports (1,IF5), primary shield wall (2,IF8), beams (1,IF5), crane support structures (1,IF4), ice condenser divider wall and slab (1,IF5), refueling pool and canal walls (2,IF6), and auxiliary structures such as dams (2,IF6), embankments (2,IF4), and tanks (2,IF5). None of the plants reported damage to reactor coolant vault walls (IF7), cable ducts (IF5), stacks (IF5), or water wells (IF4).

2.3.2 Boiling Water Reactors

The most common areas of damaged concrete for the 10 BWR plants responding to the survey were the slabs, walls, and equipment supports of the secondary containment reactor buildings and auxiliary structures, as shown below. Sub-element importance factors were assigned according to Table 1.2.^(1.1)

Boiling Water Reactors

<u>Location of deterioration</u>	<u>Number of plants</u>	<u>%</u>	<u>Importance factor (IF)</u>
Secondary containment slabs	4	40	6
Auxiliary structure walls	4	40	8
Secondary containment walls	3	30	8
Secondary containment equipment supports	3	30	4
Auxiliary structure slabs	3	30	6
Pipe tunnels	3	30	5

Two plants reported damage in each of the following areas: steel liner (IF4), containment floor slabs (IF6), containment walls (IF7), spent or new fuel pool walls and slabs (IF6), cable ducts (IF5), concrete in-take piping (IF6) or in-take crib structures (IF6). Single plants reported damage in the containment base foundation (IF10), NSSS equipment supports (IF5), Mark III upper and fuel pool slabs (IF5), drywell wall (IF6), weir vent wall (IF7), auxiliary stacks (IF5), cooling towers (IF5), dams (IF6) or embankments (IF4). Deterioration of a spillway joint due to freezing and thawing and a spall of a precast concrete panel were also reported by individual plants. None of the plants reported damage to the drywell pedestal (IF10), Mark I vertical walls (IF8), Mark I suppression chamber (IF8), Mark I steel liner (IF4), Mark II and Mark III vertical walls (IF8), Mark III concrete dome (IF8), Mark III polar crane supports (IF10), containment bottom slab (IF10), reactor pedestal or support (IF9), biological shield wall (IF6), columns (IF7), Mark II diaphragm floor (IF7), Mark III crane supports (IF5), sacrificial shield wall (IF6), drywell foundations (IF9), tanks (IF5) or water wells (IF4).

2.4 Repair Procedures

Twenty-seven of the 29 plants reported that they have repaired damaged concrete at the plant. Fourteen plants reported using dry packing for crack repairs or spall repairs. Methods commonly used to repair cracks included epoxy injection (13 plants), grout injection (12 plants), flexible sealing (12 plants) and routing and sealing (8 plants). Proprietary epoxy materials were used for injection. Materials used for grout injection were similar to materials reportedly used for dry packing repairs. Flexible sealing of cracks was performed with polysulfide, polyurethane, and other unspecified materials. Plants also reported using polyurethanes, epoxies, and polymer-modified mortars for routing and sealing cracks. Drilling and plugging of cracks was reported by three plants. Three plants reported stitching cracks, using deformed reinforcing steel bars or steel plates.

Spall or delamination repairs were most commonly done by filling the cavity with normal concrete, grout, or dry pack. Fourteen plants reported using concrete replacement and 15 plants reported using dry pack. One plant reported using shotcrete, and none of the plants reported using preplaced aggregate techniques.

The use of sealers or coatings was reported by only five plants. The sealers or coatings used included silanes, epoxies, high molecular weight methacrylate, coal tar and unspecified materials.

Four plants utilized cathodic protection for conventionally reinforced structures and two plants reported having cathodic protection on prestressed concrete members.

2.5 General Comments from the Survey

Deterioration has generally been minor due to the high initial quality of the original construction and the relatively young age of most plants. Most repairs were associated with problems during initial construction. Repairs are typically performed on an as-needed basis. Little information is available on what was used for the repair, how the repair was installed or the durability of the repairs. Follow-up evaluations of these repairs are not commonly performed.

Only 5 plants reported having evaluated the performance of repairs; all use only visual methods for their repair evaluation. The following is a summary of some of the specific comments provided by the survey respondents.

Periodic shrinkage cracks were reported by plants but these cracks were generally tight and of only cosmetic concern. Repairs of cracks using epoxy injection have performed well with no evidence of lack of durability. Shrinkage cracks of an in-take structure in one plant allowed lake water to pass into a lower dry area. These cracks were repaired using epoxy injection, which is reportedly performing well. The specific information concerning the particular repairs and the durability of the repairs was not provided.

The neutralization/acid/caustic material containing structures have experienced surface deterioration due to exposure to sodium hydroxide and sulfuric acid. These basins were repaired by sandblasting, water washing, and application of either coal tar epoxy or other proprietary polymer coatings.

Water leakage and seepage problems were reported by several plants. Epoxy or chemical grouts were used for repairs. Occasional damage to pipe support embedments was caused by water hammer. This damage was commonly repaired with dry pack mortar or grout, or the embedments were relocated to sound concrete. Erosion of concrete pipe in the circulating water system was also reported by one plant.

Spalling of an expansion joint was caused by a wrench lodging in the joint. The resulting large shallow spall was not repaired due to location and shallowness of spall. In another plant, a small spall was reported at the junction of the corner of the shield wall and the side of the refueling canal due to a small piece of concrete bridging the joint during construction. A reinforcing bar was exposed due to the spall. The spall was repaired using grout to provide protective cover to the reinforcing bar.

Minor voids and honeycombed concrete were reported by several plants during construction. These areas were typically repaired with dry pack and have performed well. Repairs by preparing the surface, forming and pumping grout into the formed area from the base of the defect to the high point were also performed.

The Tennessee Valley Authority provided General Engineering Specification No.G-34, Rev. 5, *Requirements for the Repair of Concrete During Construction, Modification and Maintenance* for review. The specification covers the structural repair of concrete and for limiting water leakage due to cracks and wall penetrations. Portions of this specification are discussed in the repair section of this report.

2.6 Surveys of Civil Engineering Structures

Two surveys, one by ACI committee 348 and one by Hauser, that describe problems experienced with concrete materials in all types of civil engineering structures were summarized by Naus.^(2.2) The ACI survey concluded that about half of the errors causing distress were a result of design and half were a result of construction. The design errors were attributed to improper consideration of details or shrinkage and temperature effects. The survey identified those structures and service conditions that reveal problems in short periods of time so may not accurately represent problems associated with long-term durability. The European survey by Hauser reviewed nearly 800 failures. The results indicated that few structures fail in use and most failures could have been avoided if additional checking of data had been performed.

A literature review by Naus^(2.2) indicated that in general the durability of concrete components in nuclear-safety-applications has been good. The following list summarizes the results of the survey concerning the distribution and frequency of occurrence of component problems:

Number of Identified Occurrences			
voids/honeycomb	24	reinforcing steel	5
cracking/spalling	19	anchorage	5
post-tension system	17	dome delamination	2
defective material/low f_c	14	over temperature	2
improper placement/repair	6		

Most of the distress was not considered a structural or safety related problem and could easily be repaired. An annotated problem listing of each occurrence was presented by Naus.^(2.2) Five instances were identified which had potential for serious structural or safety concerns. These included two dome delaminations, voids under bearing plates, anchor head failures, and a breakdown in quality control and construction management. These incidences were not associated with aging and the problems and investigation of each occurrence are described in detail in the Naus report.^(2.2)

2.7 Condition Survey of Structures

In the initial stages of a site-related repair investigation, a detailed condition survey is necessary to gain an understanding of the scope of the particular problem and the environmental

exposure conditions. A number of documents have been produced regarding detection of concrete deterioration. One of the most authoritative documents is ACI 201.1R *Guide for Making a Condition Survey of Concrete In-Service*.^(2.3) A checklist for the inspection of concrete structures from this ACI document is in Subsection 2.9 of this chapter. Persons conducting condition surveys of concrete structures should be familiar with the ACI 201 Guide so that deterioration is accurately described, and universally understood.

Many other documents are available to aid in condition surveys. These include ACI 224.1R *Causes, Evaluation and Repair of Cracks in Concrete Structures*^(2.4) and ACI 207.3R *Practices for Evaluation of Concrete in Existing Massive Structures for Service Condition*.^(2.5) ASME has recently developed guidelines for inspection of plant components in the ASME Boiler and Pressure Vessel Code, Section XI, *Rules for In-service Inspection of Nuclear Power Plant Components*.^(2.6)

2.7.1 The Field Survey

Perenchio^(2.7) provided general direction on conducting condition surveys of concrete structures. The condition survey usually begins with the review of the as-built drawings, so that the position and orientation of embedded steel reinforcing and plates in the concrete is known prior to the site visit. Next is a detailed visual examination of the structure. This initial visual examination is used to document easily obtained information on instances that can result from or lead to structural distress such as cracking, spalling, leakage, and construction defects such as honeycombing and cold joints in the concrete. The instances of cracking, spalling, leakage, delamination, efflorescence, chemical attack, or structural distress are observed and documented.

Crack survey - Cracking can be due to shrinkage of the concrete, thermal effects, alkali-aggregate reaction, or structural loads. A crack survey is usually done by drawing the locations and widths of cracks on copies of project plans. Cracking patterns may appear that suggest weaknesses in the original design, construction deficiencies, unanticipated thermal movements, chemical reactivity, detrimental environmental exposure, restrained drying shrinkage, or overloading. Distress associated with cracks such as efflorescence, rust stains, spalling, or scaling are noted. Photographs are taken to record this information, and notes made on the survey sheets to indicate the area photographed.

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

If a more detailed study is then warranted, the results of the visual and delamination surveys are used to select portions of the structure that will be studied in greater depth. The detailed study on the suspect areas can include measuring half-cell potential, removing and testing of concrete and steel samples, removing concrete powder samples for chloride analyses, locating embedded steel, additional nondestructive testing evaluations and, in extreme cases, load testing. Detailed discussions of various nondestructive test procedures are included in Chapter 5 of this report.

Corrosion survey - Concrete normally provides a high degree of corrosion protection to embedded metals because of the high pH environment within the concrete. Steel in high pH concrete is polarized anodically by a thin protective film of iron oxide that forms on its surface. However, the passivity of this film can be disrupted by a reduction in the pH value of the moisture within the

concrete due to carbonation or by the ingress of sufficient quantities of chloride ions, or other ions to the steel surface.

This disruption of the protective film causes some steel areas to become more anodic, creating an electrochemical cell with the more cathodic unaffected steel, causing corrosion to occur. The electromotive force in these cells depends on the pH value, the concentration of chloride, and the moisture content adjacent to the anodic steel and on the transfer of oxygen through the concrete to the cathodic areas. To locate areas of corrosion activity within reinforced concrete, copper-copper sulfate half-cell studies can be performed in the field to determine the locations of active corrosion. The test indicates active areas, on a macro scale, where visible deterioration or cracking cannot be seen.

Copper-copper sulfate half-cell tests are performed in accordance with the American Society for Testing and Materials (ASTM) Standard Test Method C876, *Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete*.^(2.8) It is a rapid, inexpensive test procedure that is commonly performed, however, care must be exercised in the interpretation of the results. Specific details on the use of half cells and the interpretation of the data is discussed in Chapter 5 of this report. The method involves making one electrical connection between the reinforcing steel and the negative terminal of a high-impedance voltmeter, and another between the positive terminal of the voltmeter and the half cell.

By taking half-cell readings on the surface at multiple locations, an evaluation of the corrosion activity probability of embedded reinforcing steel (or other metals) can be made. If sufficient readings are taken on a grid pattern, a diagram can be prepared that resembles a contour map. On such a diagram, points of equal electrical potential are connected by isopotential lines, permitting areas of high potentials or high corrosion probability to be readily identified.

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

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

Concrete chloride content tests - Concrete powder samples or cores should be removed from the structure where significant chloride penetration is suspected. The samples are usually removed from several depths, extending to and beyond the embedded outer layer of reinforcing steel. If the concrete contains more acid soluble chloride than about 300 ppm by weight (0.7 kg/cu m [1.2 lbs/cu yd]), as measured by ASTM C1152,^(2.8) it is considered to contain sufficient chloride to support electrochemical corrosion of embedded steel when in a moist environment which has oxygen availability. Particularly susceptible to damage by chloride ion are dissimilar metals. Where aluminum or galvanized electrical conduit has been embedded in concrete, and is in contact with the reinforcing steel, the conduit will corrode rapidly, acting as an anode to the steel.

In elements exposed to an ocean marine environment, most of the concrete elements will eventually contain significant amounts of chloride. Water intake and discharge structures present unique and severe environmental conditions where water with dissolved chlorides, sulfates, and other minerals and salts is routinely in contact with the concrete. Some regions are always wet while other

regions may experience wet and dry periods. Experience has shown that regions experiencing wet and dry cycles exhibit the greatest distress.

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

2.7.2 Laboratory and Office Studies

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

Petrographic methods: thicknesses, distribution of cement, aggregate studies, estimation of water-cementitious materials ratios, air-void distribution, types of distress, recognition of unstable aggregates, deterioration mechanisms, and age at which cracking occurred.

Chemical techniques: chemical constituents of the cementitious materials, characteristics of the paste and aggregates, presence and quantity of chemical admixtures, quantification of chemical compounds within the cement paste, efflorescence, and carbonation effects are typically investigated.

Concrete strength testing: compressive strength, modulus elasticity, tensile strength, flexural strength and bond strength of patches or overlays.

Steel materials: Yield and ultimate strengths of reinforcing bars per ASTM A615, or prestressing tendons per ASTM A421.^(2.8)

Petrographic examination can determine the quality of the cement paste, whether or not the aggregates are unstable, if the concrete contains entrained air, or if it has been damaged by freezing and thawing, chemical attack, or other deterioration mechanisms. Petrographic examination of core samples is recommended for any concrete that may be showing distress or when the concrete components or durability of the concrete is not known. A trained petrographer experienced in concrete deterioration is needed for this evaluation. The petrographic examination is done on a section of concrete that is usually sawed vertically through the center of the core, then polished to a high luster. Examination is done both with the naked eye and through a low power microscope. A complete discussion of petrographic techniques can be found in ASTM C856, *Practice for Petrographic Examination of Hardened Concrete*.^(2.8)

Office studies — A crack survey map is prepared and studied for meaningful patterns. Half-cell data are studied and isopotential lines are drawn to assist in determining active corrosion sites. Chloride ion results are plotted versus depth, to determine the profile and the chloride content at the level of the steel. Any elements that appear to be structurally marginal, due either to non-conservative design or to the effects of deterioration, are identified and appropriate calculation checks made. If the calculations are inconclusive, suitable load testing may be indicated.

Design verification — Based on physical test results, chemical analysis of elements, and present condition of the structure, a redesign of various questionable elements should be accomplished to

verify compliance with present codes and design requirements. Based upon the initial survey results, additional testing such as, impact-echo, linear polarization, infrared thermography, X-ray or numerous other destructive or nondestructive tests may be appropriate.

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

2.7.3 Report

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

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

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

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

2.8 References

- 2.1 NRC Information Notice No. 89-79, U. S. Nuclear Regulatory Commission, Washington, DC.
- 2.2 Naus, D. J., *Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants*, ORNL/TM-10059, Martin Marietta Energy Systems, Inc., Oak Ridge National Lab., NUREG/CR-4652, Sept. 1986.
- 2.3 ACI Committee 201, *Guide for Making a Condition Survey of Concrete in Service*, ACI Manual of Concrete Practice, Section 201.1R, American Concrete Institute, Detroit, Michigan 1989.
- 2.4 ACI Committee 224, *Causes, Evaluation and Repair of Cracks in Concrete Structures*, ACI Manual of Concrete Practice, Section 224.1R, American Concrete Institute, Detroit, Michigan, 1992.
- 2.5 ACI Committee 207, *Practices for Evaluation of Concrete in Existing Massive Structures for Service Condition*, ACI Manual of Concrete Practice, Section 207.3R, American Concrete Institute, Detroit, Michigan, 1992.
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- 2.7 Perenchio, W. F., *The Condition Survey*, Concrete International, V. 11, No. 1, January 1989, pp. 59-62.
- 2.8 American Society for Testing and Materials, *Annual Book of Standards*, Philadelphia, PA, 1993.

ACI 201.1R-92

Guide for Making a Condition Survey of Concrete in Service

Reported by ACI Committee 201

Members of Committee 201 who voted on the 1992 revisions:

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This guide provides a system for reporting on the condition of concrete in service. It includes a check list of the many details that may be considered in making a report, and repeats the ACI 116 standard definitions of terms associated with the durability of concrete. Its purpose is to establish a uniform system for evaluating the condition of concrete.

The guide was revised by a task group chaired by K.R. Lauer. The other task group members are indicated by an asterisk.

Keywords: bridges (structures); buildings; concrete construction; concrete durability; concrete pavements; concrete corrosion; cracking (fracturing); deterioration; environments; freeze thaw durability; inspection; joints (junctions); popouts; quality control; scaling; serviceability; spalling; strength; surveys.

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CHAPTER 1—INTRODUCTION

This guide presents a system for making a condition survey of concrete in service. A condition survey is an examination

ACI Committee Reports, Guides, Standard Practices and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are intended to be part of the Project Documents, they should be phrased in mandatory language and incorporated into the Project Documents.

of concrete for the purpose of identifying and defining areas of distress. The system is designed to be used in recording the history of a project from inception through construction and subsequent life of the structure.

While it probably will be used most often in connection with the survey of concrete that is showing some degree of distress, its application is recommended for all concrete structures. In any case, records of the materials and construction practices used should be maintained because they are difficult to obtain at a later date.

The committee has attempted to include pertinent items that might have a bearing on the performance of the concrete. Those making the survey should, however, not limit their investigation to the items listed, thereby possibly overlooking other contributing factors. Following the guide does not eliminate the need for intelligent observations and the use of sound judgement.

Those performing the survey should be experienced and competent in this field. In addition to verbal descriptions, numerical data obtained by laboratory tests and field tests and measurements should be obtained wherever possible. Photographs, including a scale to indicate dimensions, are of great value in showing the condition of concrete.

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This report was approved by letter ballot of the committee and reported to ACI headquarters July, 1990.

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The check list is provided to facilitate a thorough survey. The definition of terms and associated photographs are an attempt to standardize the reporting of the condition of the concrete in a structure.

This guide should be used in conjunction with the following:

1. ACI Committee 116 "Cement and Concrete Terminology" (ACI 116R).
2. ACI Committee 311 "Recommended Practice for Concrete Inspection" (ACI 311.1R).
3. ACI Committee 201, "Guide to Durable Concrete" (ACI 201.2R).

CHAPTER 2—CHECK LIST

Personnel conducting the condition survey must select those items important to the specific concerns relating to the reasons for the survey. Other factors may be involved and should not be overlooked during the survey.

CHECK LIST

1. Description of structure or pavement
 - 1.1 Name, location, type, and size
 - 1.2 Owner, project engineer, contractor, when built
 - 1.3 Design
 - 1.3.1 Architect and/or engineer
 - 1.3.2 Intended use and history of use
 - 1.3.3 Special features
 - 1.4 Construction
 - 1.4.1 Contractor—general
 - 1.4.2 Subcontractors—concrete placement
 - 1.4.3 Concrete supplier
 - 1.4.4 Agency responsible for testing
 - 1.4.5 Other subcontractors
 - 1.5 Photographs
 - 1.5.1 General view
 - 1.5.2 Detailed close up of condition of area
 - 1.6 Sketch map—orientation showing sunny and shady and well and poorly drained regions
2. Present condition of structure
 - 2.1 Overall alignment of structure
 - 2.1.1 Settlement
 - 2.1.2 Deflection
 - 2.1.3 Expansion
 - 2.1.4 Contraction
 - 2.2 Portions showing distress (beams, columns, pavement, walls, etc., subjected to strains and pressures)
 - 2.3 Surface condition of concrete
 - 2.3.1 General (good, satisfactory, poor, dusting, chalking, blisters)
 - 2.3.2 Cracks
 - 2.3.2.1 Location and frequency
 - 2.3.2.2 Type and size (see definitions)
 - 2.3.2.3 Leaching, stalactites
 - 2.3.3 Scaling
 - 2.3.3.1 Area, depth
 - 2.3.3.2 Type (see definitions)
 - 2.3.4 Spalls and popouts
 - 2.3.4.1 Number, size, and depth
 - 2.3.4.2 Type (see definitions)
 - 2.3.5 Extent of corrosion or chemical attack, abrasion, impact, cavitation
 - 2.3.6 Stains, efflorescence
 - 2.3.7 Exposed reinforcement
 - 2.3.8 Curling and warping
 - 2.3.9 Previous patching or other repair
 - 2.3.10 Surface coatings
 - 2.3.10.1 Type and thickness
 - 2.3.10.2 Bond to concrete
 - 2.3.10.3 Condition
 - 2.3.11 Abrasion
 - 2.3.12 Penetrating sealers
 - 2.3.12.1 Type
 - 2.3.12.2 Effectiveness
 - 2.3.12.3 Discoloration
- 2.4 Interior condition of concrete (in situ and samples)
 - 2.4.1 Strength of cores
 - 2.4.2 Density of cores
 - 2.4.3 Moisture content
 - 2.4.4 Evidence of alkali-aggregate or other reaction
 - 2.4.5 Bond to aggregate, reinforcing steel, joints
 - 2.4.6 Pulse velocity
 - 2.4.7 Volume change
 - 2.4.8 Air content and distribution
 - 2.4.9 Chloride-ion content
 - 2.4.10 Cover over reinforcing steel
 - 2.4.11 Half-cell potential to reinforcing steel
 - 2.4.12 Evidence of reinforcement corrosion
 - 2.4.13 Evidence of corrosion of dissimilar metals
 - 2.4.14 Delaminations
 - 2.4.15 Depth of carbonation
 - 2.4.16 Freezing and thawing distress (frost damage)
 - 2.4.17 Extent of deterioration
 - 2.4.18 Aggregate proportioning and distribution
3. Nature of loading and detrimental elements
 - 3.1 Exposure
 - 3.1.1 Environment—arid, subtropical, marine, freshwater, industrial, etc.
 - 3.1.2 Weather—(July and January mean temperatures, mean annual rainfall and months in which 60 percent of it occurs)
 - 3.1.3 Freezing and thawing
 - 3.1.4 Wetting and drying

3.1.5	Drying under dry atmosphere	5.2.1.2.1	Percentage of deleterious material
3.1.6	Chemical attack—sulfates, acids, chloride	5.2.1.2.2	Percentage of potentially reactive materials
3.1.7	Abrasion, erosion, cavitation, impact	5.2.1.2.3	Coatings, texture, and particle shape
3.1.8	Electric currents	5.2.1.2.4	Grading, soundness, hardness
3.1.9	Deicing chemicals which contain chloride ions	5.2.1.2.5	Other properties as specified in ASTM Designation C 33 (C 330 for lightweight aggregate)
3.1.10	Heat from adjacent sources	5.2.1.2.6	Service record on other projects
3.2	Drainage		
3.2.1	Flashing		
3.2.2	Weepholes		
3.2.3	Contour		
3.2.4	Elevation of drains		
3.3	Loading		
3.3.1	Dead		
3.3.2	Live		
3.3.3	Impact	5.2.2	Fine
3.3.4	Vibration	5.2.2.1	Type, source, and mineral composition (representative sample available)
3.3.5	Traffic index	5.2.2.2	Quality characteristics
3.3.6	Other	5.2.2.2.1	Percentage of deleterious material
3.4	Soils (foundation conditions)	5.2.2.2.2	Percentage of potentially reactive materials
3.4.1	Compressibility	5.2.2.2.3	Coatings, texture, and particle shape
3.4.2	Expansive soil	5.2.2.2.4	Grading, soundness, and hardness
3.4.3	Settlement	5.2.2.2.5	Other properties as specified in ASTM Designation C33 (C 330 for lightweight aggregate)
3.4.4	Resistivity	5.2.2.2.6	Service record on other projects
3.4.5	Evidence of pumping		
3.4.6	Water table (level and fluctuations)		
4.	Original condition of structure		
4.1	Condition of formed and finished surfaces		
4.1.1	Smoothness		
4.1.2	Air pockets ("bugholes")		
4.1.3	Sand streaks		
4.1.4	Honeycomb		
4.1.5	Soft areas (retarded hydration)		
4.1.6	Cold joints		
4.1.7	Staining		
4.2	Defects		
4.2.1	Cracking	5.3	Mixing water
4.2.1.1	Plastic shrinkage	5.3.1	Source and quality
4.2.1.2	Thermal shrinkage	5.4	Admixtures
4.2.1.3	Drying shrinkage	5.4.1	Air entraining admixtures
4.2.2	Curling	5.4.1.1	Type and source
		5.4.1.2	Composition
		5.4.1.3	Dosage
		5.4.1.4	Manner of introduction
5.	Materials of construction	5.4.2	Mineral admixtures
5.1	Hydraulic cement	5.4.2.1	Class and source
5.1.1	Class or classes—(portland, blended, high alumina, ground granulated blast furnace slag)	5.4.2.2	Physical properties
5.1.2	Type or types, and source	5.4.2.3	Chemical properties
5.1.3	Chemical analysis (obtain certified test data if available)	5.4.3	Chemical admixtures
5.1.4	Physical properties	5.4.3.1	Type and source
5.2	Aggregates	5.4.3.2	Composition
5.2.1	Coarse	5.4.3.3	Dosage
5.2.1.1	Type, source and mineral composition (representative sample available)	5.4.3.4	Manner and time of introduction
5.2.1.2	Quality characteristics	5.5	Concrete
		5.5.1	Mixture proportions
		5.5.1.1	Cement content

- 5.5.1.2 Proportions of each size aggregate
 - 5.5.1.3 Water-cementitious materials ratio
 - 5.5.1.4 Water content
 - 5.5.1.5 Chemical admixture(s)
 - 5.5.1.6 Mineral admixture(s)
 - 5.5.1.7 Air entraining admixture
 - 5.5.2 Properties of fresh concrete
 - 5.5.2.1 Slump or other workability measure
 - 5.5.2.2 Bleeding
 - 5.5.2.3 Air content
 - 5.5.2.4 Unit weight
 - 5.5.2.5 Temperature
 - 5.5.3 Type
 - 5.5.3.1 Cast-in-place
 - 5.5.3.2 Precast
 - 5.5.3.3 Prestressed (pre-tensioned or post-tensioned)
 - 5.5.4 Reinforcement
 - 5.5.4.1 Type (bar, mesh or fibers)
 - 5.5.4.2 Yield strength
 - 5.5.4.3 Thickness and quality of cover
 - 5.5.4.4 Field or shop fabricated
 - 5.5.4.5 Use of welding
 - 5.5.4.6 Presence of coating
 - 5.5.4.6.1 Type
 - 5.5.4.6.2 Condition
 - 5.5.5 Initial physical properties of hardened concrete
 - 5.5.5.1 Strength—compressive, flexural
 - 5.5.5.2 Modulus of elasticity
 - 5.5.5.3 Density and homogeneity of microstructure
 - 5.5.5.4 Percentage and distribution of air
 - 5.5.5.5 Volume change potential
 - 5.5.5.5.1 Shrinkage or contraction
 - 5.5.5.5.2 Expansion or swelling
 - 5.5.5.5.3 Creep
 - 5.5.5.6 Thermal properties
 - 5.5.6 Field test results
 - 5.5.6.1 Description of tests and frequency
 - 5.5.6.2 Actual results for full project
 - 5.5.6.3 Actual results for concrete under survey
 - 5.5.6.4 Evaluation of strength results per ACI 214
6. Construction practices
- 6.1 Storage and processing of materials
 - 6.1.1 Aggregates
 - 6.1.1.1 Grading
 - 6.1.1.2 Washing
 - 6.1.1.3 Storage
 - 6.1.1.3.1 Stockpiling
 - 6.1.1.3.2 Bins
 - 6.1.1.3.3 Moisture control/prewetting
 - 6.1.1.3.4 Cooling
 - 6.1.1.3.5 Heating
 - 6.1.2 Cement and admixtures
 - 6.1.2.1 Storage
 - 6.1.2.2 Handling
 - 6.1.3 Reinforcing steel and inserts
 - 6.1.3.1 Storage
 - 6.1.3.2 Placement
 - 6.2 Forming
 - 6.2.1 Type
 - 6.2.2 Bracing
 - 6.2.3 Coating-type and time of application
 - 6.2.4 Insulation
 - 6.3 Concreting operation
 - 6.3.1 Batching plant
 - 6.3.1.1 Type—automatic, manual, etc.
 - 6.3.1.2 Condition of equipment
 - 6.3.1.3 Batching sequence
 - 6.3.1.4 Availability of computer printouts
 - 6.3.2 Mixing
 - 6.3.2.1 Type—central mix, truck mix, job mix, shrink mix, etc.
 - 6.3.2.2 Condition of equipment
 - 6.3.2.3 Mixing time
 - 6.3.3 Transporting—trucks, buckets, chutes, pumps, etc.
 - 6.3.4 Placing
 - 6.3.4.1 Methods—conventional, underwater, slipform, etc.
 - 6.3.4.2 Equipment—buckets, elephant trunks, vibrators, etc.
 - 6.3.4.3 Weather conditions—time of year, rain, snow, dry wind, temperature, humidity, etc.
 - 6.3.4.4 Site conditions—cut, fill, presence of water, etc.
 - 6.3.4.5 Construction joints
 - 6.3.4.6 Contraction and isolation joints
 - 6.3.5 Finishing
 - 6.3.5.1 Type—slabs, floors, pavements, appurtenances
 - 6.3.5.2 Method—manual or machine
 - 6.3.5.3 Equipment—screeds, floats, trowels, straightedge, belt, etc.
 - 6.3.5.4 Hardeners, water, dust coat, coloring, etc.
 - 6.3.6 Curing

- 6.3.6.1 Type (water, covering, curing membrane, forms in place)
- 6.3.6.2 Application
- 6.3.6.3 Duration
- 6.3.6.4 Efficiency
- 6.3.7 Form removal (time of removal)
 - 6.3.7.1 Vertical
 - 6.3.7.2 Shoring

CHAPTER 3—DEFINITIONS AND ASSOCIATED PHOTOGRAPHS

Distress manifestations have been categorized and illustrated by photographs. Their severity and extent of occurrence have been quantified where possible. Their purpose is to attempt to standardize the reporting of the condition of the concrete in a structure. Those performing the survey should be thoroughly familiar with the various types of distress and the rating scheme before starting the survey.

A.1 Crack—A complete or incomplete separation, of either concrete or masonry, into two or more parts produced by breaking or fracturing.

A.1.1 Checking—Development of shallow cracks at closely spaced but irregular intervals on the surface of plaster, cement paste, mortar, or concrete.

A.1.2 Craze cracks—Fine random cracks or fissures in a surface of plaster, cement paste, mortar, or concrete.

Crazing—The development of craze cracks; the pattern of craze cracks existing in a surface.

A.1.3 D-cracking—A series of cracks in concrete near and roughly parallel to joints, edges, and structural cracks.

A.1.4 Diagonal crack—In a flexural member, an inclined crack caused by shear stress, usually at about 45 deg to the axis; or a crack in a slab, not parallel to either the lateral or longitudinal directions.

A.1.5 Hairline cracks—Cracks in an exposed concrete surface having widths so small as to be barely perceptible.

A.1.6 Pattern cracking—Fine openings on concrete surfaces in the form of a pattern; resulting from a decrease in volume of the material near the surface, or increase in volume of the material below the surface, or both.

A.1.7 Plastic cracking—Cracking that occurs in the surface of fresh concrete soon after it is placed and while it is still plastic.

A.1.8 Shrinkage cracking—Cracking of a structure or member due to failure in tension caused by external or internal restraints as reduction in moisture content develops, or as carbonation occurs, or both.

A.1.9 Temperature cracking—Cracking due to tensile failure, caused by temperature gradient in members subjected to external restraints or by temperature differential in members subjected to internal restraints.

A.1.10 Transverse cracks—Cracks that develop at right angles to the long direction of the member.

A.2 Deterioration— 1) Physical manifestation of failure of a material (e.g., cracking, delamination, flaking, pitting,

scaling, spalling, straining) caused by environmental or internal autogenous influences on hardened concrete as well as other materials; 2) Decomposition of material during either testing or exposure to service.

Disintegration—Reduction into small fragments and subsequently into particles.

A.2.1 Abrasion damage—Wearing away of a surface by rubbing and friction.

A.2.2 Blistering—The irregular raising of a thin layer, frequently 25 to 300 mm in diameter, at the surface of placed mortar or concrete during or soon after completion of the finishing operation; blistering is usually attributed to early closing of the surface and may be aggravated by cool temperatures. Blisters also occur in pipe after spinning or in a finish plastic coat in plastering as it separates and draws away from the base coat.

A.2.3 Cavitation damage—Pitting of concrete caused by implosion, i.e., the collapse of vapor bubbles in flowing water which form in areas of low pressure and collapse as they enter areas of higher pressure.

A.2.4 Chalking—Formation of a loose powder resulting from the disintegration of the surface of concrete or of applied coating, such as cement paint.

A.2.5 Corrosion—destruction of metal by chemical, electrochemical, or electrolytic reaction with its environment.

A.2.6 Curling—The distortion of an originally essentially linear or planar member into a curved shape such as the warping of a slab due to creep or to differences in temperature or moisture content in the zones adjacent to its opposite faces.

A.2.7 Deflection—Movement of a point on a structure or structural element, usually measured as a linear displacement transverse to a reference line or axis.

A.2.8 Deformation—A change in dimension or shape.

A.2.9 Delamination—A separation along a plane parallel to a surface as in the separation of a coating from a substrate or the layers of a coating from each other, or in the case of a concrete slab, a horizontal splitting, cracking or separation of a slab in a plane roughly parallel to, and generally near, the upper surface; found frequently in bridge decks and other types of elevated reinforced-concrete slabs and may be caused by the corrosion of reinforcing steel; also found in slabs on grade caused by development, during the finishing operation, of a plane of weakness below the densified surface; or caused by freezing and thawing, similar to spalling, scaling, or peeling except that delamination affects large areas and can often be detected by tapping.

A.2.10 Distortion—See *Deformation*.

A.2.11 Dusting—The development of a powdered material at the surface of hardened concrete.

A.2.12 Efflorescence—A deposit of salts, usually white, formed on a surface, the substance having emerged in solution from within either concrete or masonry and subsequently been precipitated by evaporation.

A.2.13 Erosion—Progressive disintegration of a solid by the abrasive or cavitation action of gases, fluids, or solids in motion.

A.2.14 Exfoliation—Disintegration occurring by peeling

off in successive layers; swelling up and opening into leaves or plates like a partly opened book.

A.2.15 Exudation—A liquid or viscous gel-like material discharged through a pore, crack, or opening in the surface of concrete.

A.2.16 Joint spall—A spall adjacent to a joint.

A.2.17 Pitting—Development of relatively small cavities in a surface; in concrete, localized disintegration, such as a popout; in steel, localized corrosion evident as minute cavities on the surface.

A.2.18 Peeling—A process in which thin flakes of mortar are broken away from a concrete surface, such as by deterioration or by adherence of surface mortar to forms as forms are removed.

A.2.19 Popout—The breaking away of small portions of a concrete surface due to localized internal pressure which leaves a shallow, typical conical, depression.

A.2.19.1 Popouts, small—Popouts leaving holes up to 10 mm in diameter, or the equivalent.

A.2.19.2 Popouts, medium—Popouts leaving holes between 10 and 50 mm in diameter, or the equivalent.

A.2.19.3 Popouts, large—Popouts leaving holes greater than 50 mm in diameter, or the equivalent.

A.2.20 Scaling—Local flaking or peeling away of the near-surface portion of hardened concrete or mortar; also of a layer from metal.

A.2.20.1 Scaling, light—Loss of surface mortar without exposure of coarse aggregate.

A.2.20.2 Scaling, medium—Loss of surface mortar 5 to 10 mm in depth and exposure of coarse aggregate.

A.2.20.3 Scaling, severe—Loss of surface mortar 5 to 10 mm in depth with some loss of mortar surrounding aggregate particles 10 to 20 mm in depth.

A.2.20.4 Scaling, very severe—Loss of coarse aggregate particles as well as mortar, generally to a depth greater than 20 mm.

A.2.21 Spall—A fragment, usually in the shape of a flake, detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion within the large mass.

A.2.21.1 Small spall—A roughly circular depression not greater than 20 mm in depth nor 50 mm in any dimension.

A.2.21.2 Large spall—May be roughly circular or oval or in some cases elongated, more than 20 mm in depth and 150 mm in greatest dimension.

A.2.22 Warping—A deviation of a slab or wall surface from its original shape, usually caused by either temperature or moisture differentials or both within the slab or wall.

A.3 Textural features and phenomena relative to their development.

A.3.1 Air void—A space in cement paste, mortar, or concrete filled with air; an entrapped air void is characteristically 1 mm or more in size and irregular in shape; an entrained air void is typically between 10 μ m and 1 mm in diameter and spherical or nearly so.

A.3.2 Bleeding—The autogenous flow of mixing water within, or its emergence from, newly placed concrete or mor-

tar; caused by the settlement of the solid materials within the mass; also called water gain.

A.3.3 Bugholes—Small regular or irregular cavities, usually not exceeding 25 mm in diameter, resulting from entrapment of air bubbles in the surface of formed concrete during placement and consolidation.

A.3.4 Cold joint—A joint or discontinuity resulting from a delay in placement of sufficient time to preclude a union of the material in two successive lifts.

A.3.5 Cold-joint lines—Visible lines on the surfaces of formed concrete indicating the presence of joints where one layer of concrete had hardened before subsequent concrete was placed.

A.3.6 Discoloration—departure of color from that which is normal or desired.

A.3.7 Honeycomb—Voids left in concrete due to failure of the mortar to effectively fill the spaces among coarse aggregate particles.

A.3.8 Incrustation—A crust or coating, generally hard, formed on the surface of concrete or masonry construction or on aggregate particles.

A.3.9 Joint—A physical separation in concrete, whether precast or cast-in-place, including cracks if intentionally made to occur at specified locations; also the region where structural members intersect such as a beam-column joint.

A.3.10 Laitance—A layer of weak and nondurable material containing cement and fines from aggregates, brought by bleeding water to the top of overwet concrete; the amount is generally increased by overworking or over-manipulating concrete at the surface by improper finishing or by job traffic.

A.3.11 Sand pocket—A zone in concrete or mortar containing fine aggregate with little or no cement.

A.3.12 Sand streak—A streak of exposed fine aggregate in the surface of formed concrete, caused by bleeding.

A.3.13 Segregation—The differential concentration of the components of mixed concrete, aggregate, or the like, resulting in nonuniform proportions in the mass.

A.3.14 Stalactite—A downward-pointing deposit formed as an accretion of mineral matter produced by evaporation of dripping water from the surface of concrete, commonly shaped like an icicle.

A.3.15 Stalagmite—An upward-pointing deposit formed as an accretion of mineral matter produced by evaporation of dripping water, projecting from the surface of concrete, commonly conical in shape.

A.3.16 Stratification—The separation of overwet or overvibrated concrete into horizontal layers with increasingly lighter material toward the top; water, laitance, mortar, and coarse aggregate tend to occupy successively lower positions in that order; a layered structure in concrete resulting from placing of successive batches that differ in appearance; occurrence in aggregate stockpiles of layers of differing grading or composition; a layered structure in a rock foundation.

A.3.17 Water void—Void along the underside of an aggregate particle or reinforcing steel which formed during the bleeding period; initially filled with bleed water.

3. CONCRETE DETERIORATION MODES

The comprehensive report by Naus entitled *Concrete Component Aging and Its Significance Relative to Life Extension of Nuclear Power Plants*^(3.1) describes subjects that are of special significance to the longevity and life extension of concrete nuclear power plant components. The majority of the concrete problems described by Naus were identified as minor. They were related to concrete cracking, concrete voids, and low concrete strengths at early ages. Five instances of distress considered to be major problems were described in detail. These isolated incidents were related to design, construction, or human error. They involved dome delaminations, voids under tendon-bearing plates, anchor head failures, and a breakdown in quality control and construction management.

Many factors can cause deterioration of concrete structures. Those that may affect Category I concrete structures are shown in Table 3.1^(3.27) in terms of potential mechanisms that can affect the concrete, mild steel reinforcement, prestressing, and liner structural steel systems. Table 3.2^(3.27) presents degradation factors for the concrete, mild steel reinforcement and prestressing, and potential areas where the degradation factors can impact the performance of these material systems.

NUMARC Report 90-01, *Pressurized Water Reactor Containment Structures License Renewal Industry Report*,^(3.2) presents information on age-related degradation mechanisms and their significance in PWR power plants. The degradation mechanisms considered non-significant for applicable PWR containment structures included: freeze-thaw, leaching of calcium hydroxide, reactions of aggregates, elevated temperature, irradiation, concrete interaction with aluminum, and strain aging. Mechanisms that are considered potentially significant include attack due to aggressive chemicals, corrosion of mild steel and prestressing systems, prestress loss, corrosion of concrete containment liners, corrosion of free-standing steel containment shells, settlement, and fatigue of hot penetrations. The NUMARC report^(3.2) conclusions are very general. They should not be referenced without careful review and evaluation of specific plant features and conditions. The conclusions were based on significant assumptions and need to be used with caution. Experience from WJE field investigations indicates that elevated temperatures can dramatically increase the rate of deterioration due to corrosion and sulfate attack and that reactions of aggregates can cause long-term durability problems.

The mechanisms of deterioration and their effects on concrete are described below. They are separated into those that result from either short-term exposures or long-term exposures involving physical or chemical changes.

3.1 Short-Term Exposures

Short-term exposures that result in deterioration include fire damage, thermal shock, early-age freezing, impact, early-age cracking, and seismic loadings.

3.1.1 Thermal Shock and Fire

The ability of concrete structures to perform exceptionally well both during and after exposure to elevated temperatures has been well documented. Principally due to its high capacity for heat absorption and relatively low thermal conductivity, concrete can sustain its strength and integrity during fire events, as well as other high temperature excursions. Three of the 29 plants responding to the survey reported deterioration due to temperature or fire.

Concrete undergoes a variety of physical and chemical changes during heating and subsequent cooling. It is important to understand these material changes, as well as the post-fire material

properties and evaluation techniques to identify the extent and nature of possible degradation of concrete in order to effectively evaluate options for reuse and restoration of a damaged structure.

A concrete structure can be exposed to a variety of thermal events during its lifetime that may result in serious degradation requiring repair. Elevated temperature exposures include:

- Fire — high intensity, high temperature exposure for a relatively short duration
- Heat soaking — operating conditions of relatively low elevated temperatures for long durations
- Thermal cycling — alternating heating and cooling at moderate temperatures for short to moderate durations over an extended period of time

Standard fires — Fire resistance ratings of building elements are traditionally specified by codes and are determined by the methods and criteria outlined in ASTM E119, *Standard Methods of Fire Tests of Building Construction and Materials*.^(3.3) The fire exposure condition defined by this procedure is known as the standard fire, and has been used since the 1920s to measure the relative fire performance of concrete and other structural systems.

The specific time-temperature relationship for the standard fire is shown in Fig. 3.1 and provides for temperatures of 540°C (1000°F) at 5 minutes, 1010°C (1850°F) at 1 hour, 1050°C (1925°F) at 2 hours, and 1095°C (2000°F) after a 3-hour duration. The fire represents the combustion of approximately 4.5 kg (10 lbs) of wood with an equivalent heat potential of 8,000 BTU per pound for each square foot of exposed area per hour of test. (1BTU/ft²/hr/°F = 5.678 W/m²/°C)

The ASTM E119 test is used to determine the length of time that an element will contain a fire, retain its structural integrity, or retard the transmission of heat. While these properties are principally used to define compliance with code-specified criteria (2-hour rated assembly), they are also used to judge the relative severity of a fire damage.

Real fires — Real fires, as opposed to standard laboratory tests fires are rarely, if ever, as severe in nature. Accordingly, it is generally agreed that the safety factors in structural fire design are actually contained in the E119 fire curve and the fire resistance (hourly) ratings prescribed by code authorities.

As shown in Fig. 3.2, real fires are comprised of three distinct phases: ignition, flame, and cooling. According to substantiated theory, the basic factors that effect these phases and the corresponding time-temperature relationship during fire development in an enclosed space include:

- Characteristics of the fuel load
- Size of the compartment
- Ventilation of the enclosed space
- Characteristics of the bounding structure enclosing the space

A comparison of time-temperature relationships for low, moderate, and high ventilation conditions of a real fire is presented in Fig. 3.3. A detailed explanation of all influencing parameters is beyond the scope of this paper, but the relationships shown in Fig. 3.3 clearly show the effect of ventilation on maximum gas temperature achieved during a fire and the duration of the flame phase before cooling occurs. As shown in the figure, higher ventilated fires can result in much higher initial gas temperatures, compared to the E119-type fire, but typically exhaust the fuel source sooner. These fires are often referred to as short duration, high intensity fires (SDHI). When ventilation is restricted, the resulting fire is lower in intensity, but typically longer in duration. The amount of fuel available to a fire also determines the flame phase duration. Note the unrealistic duration of the E119 fire, which, by definition, is based on an inexhaustible fuel source.

Pool fires — A unique type of fire exposure condition occurs as a result of a so-called hydrocarbon pool fire. These conditions result from large, free-burning, fluid hydrocarbon-fueled fires that may occur at processing facilities or other areas that store significant quantities of volatile combustibles. Fires of this nature are significantly different than standard building fires, principally due to the very rapid development of high temperatures and heat fluxes that can be sustained for great periods of time, depending on the quantity of fuel that is available. Pool fires can induce greater thermal shock and thermal loading to concrete systems than real or laboratory standard fires.

The time-temperature relationship of a representative hydrocarbon pool fire is shown in Fig. 3.4.^(3.12) The fire is represented by a constant heat flux of 17.33 W/sq cm (55,000 BTU sq ft/hour) after 5 minutes of exposure. For the example shown in Fig. 3.4, equivalent gas temperatures of approximately 1025°C (1875°F) are required to produce the desired heat flux, significantly greater than those of the standard E119 fires, or most real fires.

Heat soaking — Heat soaking refers to the long-term exposure of concrete elements to elevated temperatures associated with normal operating conditions within a facility. For nuclear power plants, current practice outlined in ANSI/ASME BPV-III-2 limits most in-service design concrete temperatures to 65.5°C (150°F) under normal operating conditions. Many industrial facilities contain processing equipment or boilers that subject supporting structural components to near constant temperatures from 38°C (100°F) to 94°C (200°F). Specialty furnaces, foundry floors, and support bases often encounter sustained temperatures of up to 260°C (500°F) to 315°C (600°F).

The duration of elevated temperature exposures is generally regarded as the design service life of the facility, which may range from 30 to 50 years. Premature failures often occur due to the inability of the concrete to sustain imposed loads as material properties degrade over time. Designs of unique concrete storage casks for burial of nuclear waste and other specialty civil structures are based on much higher long-term temperature exposures.

Heat soaking results in greater degradation of mechanical properties of concrete compared to equal temperature conditions encountered in a fire. While many of the in-service temperatures are regarded as moderate (less than 150°C (300°F)), the corresponding changes of compressive and flexural strength, modulus, and creep, are sufficient to affect the performance of a structure.

Thermal cycling — Thermal cycling conditions involve the alternating heating and cooling at moderate temperatures for extended periods of time. These temperature excursions are similar to heat soaking in that they are often encountered in normal service of nuclear power plants, industrial facilities, and other processing environments.

Cycling can result in degradation of mechanical properties, oftentimes to a greater degree than constant temperature heat soaking.^(3.15) The influence of moisture content, external moisture availability, and differential thermal properties of constituent components affect resulting strength, modulus, and creep properties of concrete exposed to repetitive thermal cycling.

General concrete properties at elevated temperatures — Elevated temperatures cause alterations of the physical and chemical composition of portland cement concrete that significantly affect the mechanical properties. References 3.5 to 3.16 concern the evaluation of a variety of concretes at elevated temperatures (hot), as well as residual properties after cooling.

At temperatures up to 100°C (212°F), concrete loses evaporable water from the paste matrix. At greater temperatures the chemically combined water is gradually driven off from the paste, resulting in a dehydration and decomposition of the cement gel and calcium hydroxide. Bond between paste and aggregate constituents may be broken and lost as a result of the

dehydration, thus decreasing strength and stiffness properties at increasing temperatures. Dehydration of the calcium hydroxide and gel is completed at 595°C (1100°F) and 845°C (1550°F), respectively.^(3.8)

Siliceous or calcareous aggregates will also go through physio-chemical changes during heating. Quartz-bearing siliceous aggregates exhibit a prominent alteration upon heating due to crystalline transformation from the α to β phases. This change occurs at approximately 570°C (1060°F), and is accompanied by a significant increase in volume of up to 5.7 percent.^(3.8) This internal expansion can lead to surface popouts, spalling, and micro-cracking at the paste-aggregate interface.

At temperatures of approximately 230-290°C (450-550°F), sedimentary or metamorphic aggregates may change coloration and display a slight pink hue. Further heating to temperatures up to 595°C (1100°F) may produce a darker pink or reddish color change. Calcareous aggregates will dissociate completely at about 815-870°C (1500-1600°F), resulting in expansion of the aggregate and disruption of surrounding matrix. Some concrete components may melt at temperatures greater than 1300°C (2400°F), though these conditions are rarely experienced in normal, building fires.

Concrete made with normal cements and aggregates has been used successfully over many years in locations such as soaking pits, foundry floors, and furnace and compressor supports at relatively high, sustained and repetitive temperatures up to 370°C (700°F). Refractory concretes, made with high alumina cements and heat-resistant aggregates, however, are capable of resisting sustained temperatures up to 1800°C (3270°F). Heat-resistant aggregates include crushed firebrick, expanded fire clay, flint fire clay, calcined kaolin, and corundum.

Concrete containing carbonates and sanded lightweight aggregates retains more than 75 percent of its original compressive strength up to 650°C (1200°F) when heated unstressed and tested hot; for siliceous aggregate concrete the corresponding temperature is 800°F (427°C).^(3.55) When hot concrete (500°C [930°F]) is quenched with water, as may happen during a fire, there is a greater reduction in residual compressive strength than when the concrete is slowly cooled. This is attributed to thermal shock.

The decline of flexural strength is much greater than that of compressive strength. The flexural strength of siliceous aggregate concrete at 400°C (750°F) varies from 25 to 0 percent of the original flexural strength. Similar tests on limestone aggregates found a 50 percent reduction at the same maximum temperature. Again, quenching results in greater strength losses.

With thermal cycling, most of the ultimate strength loss occurs during the first few cycles, with gradual losses during continued cycling, when heated up to 300°C (575°F). The inter-relationship of thermal cycling, moisture content, and thermal volume change is important in nuclear applications of concrete. Tests on sealed concrete exposed over durations up to 14 days of thermal cycling up to 150°C (300°F), resulted in the following conclusions:

For a constant moisture condition (sealed), the thermal coefficient of expansion decreased.

After the first thermal cycle (20 to 150 to 20°C [70 to 300 to 70°F]) of the concrete in a moist condition, the concrete undergoes a significant residual expansion, and this expansion successively increases with additional cycles. A larger permanent expansion will occur in concrete of high moisture content than for concretes of lower moisture contents.

Compressive strength: Many detailed studies of compressive strength loss as a function of thermal exposure have been performed. A comprehensive listing of these works is presented in References 3.8 and 3.10.

Figure 3.5 presents a compilation of the effects of temperature on the hot strength of concrete as summarized from Ref. 3.10. This figure shows the reduction in strength for a wide variety of concretes tested in several different manners. Factors which influence strength performance as reported in Reference 3.10 include:

- Concrete made with siliceous aggregate loses greater strength than calcareous aggregate concrete at temperatures greater than 540°C (1000°F)
- Greater strength loss is experienced when moisture does not escape while heating
- Concrete that is loaded during heating loses less strength than unloaded material
- Low cement-to-aggregate ratio mixes lose less strength than rich mixes
- Initial strength of concrete has little effect on percentage of strength reduction resulting from exposure to elevated temperatures

For conventional concretes made with siliceous and calcareous aggregates and loaded while heated, little strength reduction is experienced up to temperatures of approximately 425°C (800°F), as shown in Fig. 3.6. These results were obtained by loading specimens to 40 percent of the design strength during heating and subsequently testing specimens to failure while hot. At 540°C (1000°F), siliceous aggregate concrete experiences loss of hot strength to approximately 65 percent of room temperature strength, with little to no reduction for calcareous concrete. Strengths of calcareous and siliceous aggregate concrete have been reduced to approximately 80 and 45 percent of original strengths, respectively, at exposure temperatures of 760°C (1400°F).

Residual strength, the strength of concrete after cooling of the material from the elevated temperature condition, is typically lower than values measured while hot. This reduction is principally associated with the additional microcracking and matrix disruption associated with the cooling cycle. Residual strengths reported for a variety of concretes from Ref. 3.13 are presented in Fig. 3.7. For siliceous aggregate concrete, residual strengths of 75, 50, and 20 percent of original may be anticipated after heating and subsequent cooling from temperatures of 175, 370, and 565°C (350, 700, and 1050°F), respectively.^(3.55)

Long-term and cyclic exposure: Studies reported in Ref. 3.8 and Ref. 3.9 indicate that longer durations of heating result in greater reduction of strength, though most strength is lost during the first 30 to 60 days of exposure. Figure 3.8 presents the results of a study^(3.9) showing increasing reduction of strength for increasing durations of exposure of up to 4 months and increasing exposure temperatures up to 450°C (850°F). The results shown in the figure are average results for both dolostone and limestone aggregate concretes, having w/c ratios of either 0.33, 0.45, or 0.60, except at 150°F where only the limestone aggregate concrete results are presented. Figure 3.9^(3.8) presents a range of reductions reported by several researchers for several temperature ranges and heat soak periods up to 900 days. It is apparent that limited data exist to fully describe the long-term effects of high temperature heat soaking for durations greater than 3 to 4 years.

Due to nonuniform volume changes, thermal gradients, and repetitive interfacial movements, strength loss associated to thermal cycling is generally greater than concrete exposed to a single excursion or heat soaked.

Modulus of elasticity: Figures 3.10 and 3.11 show the relationship of reduction in modulus of elasticity at various temperature exposures. The trends in the noted relationships are similar, but slightly greater, to that experienced for compressive strength, with reductions of approximately 50 percent at temperatures of 260 to 315°C (500 to 600°F). Limited results in Fig. 3.11 indicates that

the reduction in residual modulus is greater when the duration of heating is longer, due to the effects of microstructure alteration and bond degradation.

Thermal expansion: Volume change of concrete due to thermal expansion plays an integral role in cracking, disruption, and overall structural behavior during a fire or heating event. Aggregates in concrete always expand upon heating, while the cement paste undergoes substantial shrinkage, as shown in Fig. 3.12.^(3,8) Values for coefficient of thermal expansion, when drying is allowed, can range from 2.2 to $4.4 \times 10^{-6}/^{\circ}\text{C}$ (4 to $8 \times 10^{-6}/^{\circ}\text{F}$).

Expansion of concrete due to heating is not fully reversible after cooling, and the residual expansion can vary from 10 to 50 percent of the total thermal expansion.^(3,8) This permanent expansion is attributable to dehydration of the matrix and disruption of the microstructure.

Spalling: Many types of exposures involving rapid heating, direct flame impingement, or spillage of high temperature liquids can produce spalling of surfaces of concrete. Spalling results in the fracturing of the concrete surface to variable depth and is oftentimes explosive in nature. The loss of concrete due to spalling exposes "new" material at greater depths to temperature extremes. While general disruption and deterioration of concrete surfaces can occur due to exposure to fire, the action of explosive spalling is a somewhat random occurrence. Several theories exist as to the mechanism of spalling.

Thermal gradient: A rapidly heated surface expands at a significantly greater rate than a cooler underlying material, resulting in the buildup of stresses near the interfaces. When the shear/tensile capacity of the concrete has been reached, a failure plane develops and the outer concrete is disengaged. This process can reoccur when the "new" layer of concrete is exposed and heated at a rate greater than the next interior layer.

Restraint: Localized heating and expansion of a surface or portion of a structure is resisted by remaining portions of the structure that are cooler, stiffer, and undergoing less dimensional change. A region of very high compressive stresses can develop parallel to the surface of the member and may ultimately fail the material by crushing.

Moisture plug: Under rapid heating, free water in pores of the concrete may be transformed to steam. If the steam is unable to migrate through the pore structure and "vent" to the exterior, significant pressures may develop internally. If the internal pressure is unable to be resisted by the tensile strength of the restraining concrete, failure and disruption of the surface material will occur.

Aggregate properties: Individual aggregate particles or the entire aggregate population may possess thermal expansion properties radically greater than the composite concrete material and, upon heating, cause localized spalling or popouts.

Calcination: Limestone aggregate concrete can go through a physio-chemical change when exposed to prolonged temperatures above approximately 650 to 700°C (1200 to 1300°F). The effect of this is most apparent when cooling of the material occurs and the free lime (CaO) resulting from the calcining (dissociation of the CaCO_3 to CaO and CO_2) combines with available moisture. The reaction is expansive in nature and will disrupt the surface of the material. This results in the surfaces taking on an abraded, or scaled, appearance. As "fresh" surfaces are exposed to the atmosphere as outer layers are shed, this material will also react with moisture to continue the process. This shedding, or progressive scaling-like phenomenon, will continue to a depth where material has not been subjected to elevated temperatures sufficient to dissociate the CaCO_3 . Surface dusting and on-going scaling has been reported to continue for several weeks after a fire.

Areas of a fire-damaged structure exhibiting calcinated surfaces provide strong clues as to the temperature distribution resulting from the fire. Calcinated concrete surfaces do not represent a severe structural or strength related concern, since many times the depth of affected concrete is relatively shallow. However, these affected surfaces, if not appropriately repaired, can result in a potential durability concern for certain environmental exposure conditions, such as freeze/thaw, wetting/drying, or chemical attack.

Cracking of fire damaged concrete: Concrete members may exhibit several different types of cracking upon exposure to fire, or during subsequent cooling phases. Some cracking is a direct result of restraint to volumetric growth during heating associated with geometry of individual members. A classic example of this behavior is the heating of a concrete column that is rectangular or square in cross section. Upon uniform heating, the column expands in all directions, effectively attempting to move the concrete at the corners outward. This region of the column is restrained by the cooler, underlying concrete. Upon further heating, the tensile capacity of the concrete at the corner is reached, and failure occurs. Vertical cracking, typically 7 to 10 cm (3 to 4 in.) from the corners of the column, will occur as the thermal growing square column "sheds" its corners. If heating of a member is rather slow and uniform, but cooling is rapid and sudden, the above described phenomenon can also occur as outer layers of concrete cool, contract, and are restrained by slower cooling interior concrete.

Transverse and random cracking can occur due to differential dehydration of concrete surfaces. This so-called thermal cracking is most noted at temperatures above 540°C (1000°F) and may be localized in nature.

Cracking associated with overall length changes of structural members can also occur. Beams, columns, and slabs that are heated for long periods of time will expand, resulting in significant dimensional changes that require accommodation by other connected portions of the structure. Shear failures of concrete columns due to dimensional growth of integrally cast supported slabs have been reported during long duration fires.

Reinforcing steel: Hot rolled mild steel complying with ASTM A615 (Grade 60) is generally used as reinforcing bars for concrete structures. This material exhibits a reduction in strength and modulus when measured hot for increasing exposure temperatures. Figure 3.13^(3.11) shows the influence of temperature on the hot yield strength of certain steels and indicates that at temperatures of 425 and 595°C (850, and 1100°F), the strength of mild reinforcing bars is reduced to nominally 75 and 50 percent of room temperature strength, respectively. Similar reductions in modulus of elasticity are experienced at these temperatures.

Residual strength of mild steel is generally unaffected if exposed to temperatures less than 595 to 650°C (1100 to 1200°F). At temperatures greater than 650°C (1200°F), some metallurgical realignments of the steel can occur, resulting in slight reductions in yield strengths. Steel that has been rapidly cooled (quenched) may also experience slight post-exposure embrittlement, characterized by reduction in elongation.

Recent studies^(3.16) have suggested that some degradation of the yield strength may occur in Grade 60 bars that have been exposed to temperatures of 480 to 540°C (900 to 1000°F), then cooled and tested at room temperature. Figure 3.14 shows residual strength results from testing of bars exposed to temperatures up to 750°C (1380°F).

The tensile strength of cold-drawn prestressing steel is affected to a greater extent when exposed to temperatures above 260°C (50°F) than is the yield strength of mild steel reinforcement or the tensile strength of high-strength steel alloy bars, as shown in Fig. 3.13. When tested at temperatures of 260, 425, and 595°C (500, 800, and 1100°F), the tensile strength of the cold-

drawn prestressing steel is reduced 15, 50, and 85 percent, respectively, relative to the room temperature strength.

Evaluation of fire damaged concrete: Evaluation of a fire damaged structure involves a systematic process similar to that employed when investigating more "conventional" damage, such as that associated with environmental deterioration, corrosion damage, or structural overload. A key difference is the need to gain an accurate estimate of the maximum temperature that various parts of the affected structure were subjected to, so that engineering judgements can be made to determine overall residual strength. This information is often obtained by insightful observations from the fire damaged area, performance of in-place nondestructive testing of suspect concrete and steel, and use of detailed petrographic examinations, supplemented by controlled heating tests of undamaged concrete. Following a fire, after it has been determined that a structure is safe to enter, the first step in evaluating damage is to obtain an estimate of the maximum temperature reached and its duration. An estimate of the temperature can be generally determined by the color of the concrete during exposure: the color for siliceous or limestone aggregate concrete changes as the temperature increases. It changes from a pink or red to gray at about 600°C (1100°F), then to a buff color at 900°C (1650°F). Maximum temperature exposure and its remaining tensile strength can be determined by examining the micro-structure and microhardness of prestressing steel. The effects of high temperatures on normal reinforcing steel are not as critical, but similar metallographic methods may be used.

Once the maximum temperature and duration have been established, the available engineering data on the concrete properties can be incorporated in an appropriate structural analysis to determine the acceptability of the structure.

The following is a generalized outline for the assessment and repair process.

1. Identify the Extent and Severity of the Fire/Thermal Damage

Perform a condition survey of the affected areas

Review fire department records

Determine type and nature of combustibles

Inspect condition of nonstructural materials

Map visual damage

– locations of combustibles, smoke, soot, char

– areas of spalling

– sound for delaminations

– measure deformations, plumbness, deflection, lateral sweep, twist

– depth of spalling

– depth of reinforcement

– depth of char

Perform nondestructive testing to assess extent of damage

– pulse velocity tests

– in-place hardness testing

– sample concrete and steel material samples

– petrographic examinations

– companion thermal exposure testing

– strength testing

Assess structural considerations

– structural analyses

– in-situ load testing

2. Develop Repair Methodology

Defer repair work; repairs not needed

Restore concrete by patching

Restore concrete by patching and supplement with protection schemes
Replace affected elements with new

3. Evaluate and Select Repair Materials
 - Conventional concrete mixes
 - Specialty concrete mixes
 - Shotcrete
 - Supplemental reinforcement schemes
 - Crack repair materials
 - Assess post-repair durability considerations

4. Implementation and Inspection of Repair

A number of nondestructive tests (NDT) can be performed on the concrete to determine its residual engineering properties. These procedures include rebound hammer, ultrasonic pulse velocity, and impact-echo.

Nondestructive testing of fire damaged concrete: One of the most versatile and effective tools for evaluating the extent and magnitude of reduction of overall concrete in a fire damaged structure is the use of through-transmission ultrasonics.^(3.14) The use of ultrasonic test methods to evaluate material properties and defects in concrete has been employed for a number of years. These techniques are based on detecting changes in amplitude, phase, and direction of mechanical waves as they propagate through a concrete member. Changes in wave characteristics generally indicate a corresponding change in the internal structural composition of the concrete.

Through transmission ultrasonics, as well as impact echo methods, enable measurements of the pulse velocity of a suspect member to be evaluated. Under non-fire conditions, the relationship between pulse velocity and compressive strength can vary significantly for different mixes, due to differences in aggregate type, aggregate content, water-cement ratio (w/c), air content, moisture content, and density. A generalized relationship for a variety of concretes is presented in Fig. 3.15. However, when the technique is applied to a single mix, such as normally would be encountered when assessing a fire damaged concrete, relative differences can be interpreted as to areas that may have been significantly affected by a fire. Pulse velocities will decrease due to corresponding changes in density and modulus. Accordingly, due to the paste dehydration, matrix restructuring, and micro-cracking resulting from the fire, a corresponding decrease in measured pulse velocity will occur for a given strength concrete. Figure 3.16^(3.9) presents the relationship between change in pulse velocity and compressive strength for a specific concrete exposed to temperatures up to 590°C (1100°F). Mapping of measured velocities over the entire structure provides evidence of those regions that may have been more severely affected than others. Further information on nondestructive techniques is provided in Chapter 5 of this report.

3.1.2 Plastic Shrinkage Cracking

Plastic shrinkage cracks are those that occur while the concrete is relatively fresh and has not started to harden. They usually appear on exposed horizontal surfaces, and can occur anytime that the ambient conditions (temperature, humidity, wind velocity) are conducive to rapid evaporation. Certain types of concrete are more susceptible to this phenomenon than others. Seven of the 29 plants reported plastic shrinkage cracking in the survey.

Plastic shrinkage cracking generally can occur when the rate of evaporation exceeds the rate of bleeding of the concrete mix. The width of the crack at the location of the start of separation may be as much as 6.3 mm (¼ in.). However, the cracks are usually no more than 0.6 or 0.9 m (2 or 3 ft) long and are rarely more than 50 to 75 mm (2 to 3 in.) deep. Such cracks are seldom

significant structurally, and once the crack starts, the stresses rapidly dissipate.

This type of cracking has become a significant problem in recent years due to the wide usage of latex modifiers, superplasticizers (HRWRs), air entrainment, and silica fume. Latex and HRWRs greatly reduce the water content and therefore the bleeding capacity of the concretes. The rate of evaporation can therefore more easily exceed the rate of bleeding. Silica fume intensifies the problem because HRWRs must be used to compensate for the extreme fineness of the silica fume material. The HRWR reduces the amount of bleed water available while the high fineness of the silica fume reduces the rate at which the water can move through the concrete. Air entrainment also reduces the bleeding rate.

The obvious solutions to the problem are to reduce the evaporation rate or increase the bleeding capacity of the concrete. The former can be accomplished to various degrees by sunscreens, windbreaks, fog mist, or monomolecular curing films. The most effective means of avoiding the loss of bleed water (reducing evaporation) is with impermeable curing covers such as polyethylene sheeting. However, this method can be cumbersome. The most cost-effective and potent method to date has been the use of a fog mist applied to the concrete surface from the upwind side of the work. This must be done with a commercial grade fog nozzle, which produces a mist so fine that it is nearly impossible to damage the concrete surface and can provide broad coverage. Great care must be exercised so that none of this fogged water becomes part of the concrete during finishing. Wetting down the subgrade and the forms before the concrete is placed is also helpful. Increasing the bleeding capacity of the concrete is usually not practical; however, the use of a water reducing admixture containing hydroxylated carboxylic acid tends to increase the bleeding rate.

Plastic shrinkage cracks sometimes form on the top surface of precast concrete when concrete is cast outside in winter conditions in heated forms and covered with a tarpaulin. The forms and the warm concrete heat the air between the concrete and the tarpaulin, greatly reducing its relative humidity. This warm, moist air leaks out and is replaced by cold, dry air that quickly warms up and absorbs moisture. Normally, plastic shrinkage cracks are roughly parallel to each other and generally perpendicular to the wind direction. In the case of the heated and covered precast concrete, little wind is involved and the cracks have random orientations. They can be very severe and aesthetically unacceptable. Plastic shrinkage cracks occur very early after casting and are not considered a problem associated with long-term aging. However, these early-age cracks can allow the penetration of liquids and gases into the concrete.

3.1.3 Craze Cracking

Craze cracking on concrete surfaces is a network of fine random cracks caused by shrinkage of the mortar on the top surface of the horizontal slab concrete. The cracks rarely penetrate more than 3.2 mm ($\frac{1}{8}$ in.) into the concrete and are most obvious on steel troweled surfaces. The cracks occur early and are usually noticeable by the first week after placement. Often they are not readily visible until the surface of the concrete is wetted and allowed to dry. Craze cracking does not affect the structural integrity of the concrete and rarely affects the durability or wear. However, they can be unsightly. Craze cracking occurs due to poor construction practices such as inadequate curing, a high water content, excessive floating or jitterbugging the concrete to depress the coarse aggregate, or finishing while bleed water is present on the surface. Any process to increase the water or paste content along the surface, such as sprinkling cement on the surface to absorb bleed water, may result in crazing.

Crazing can be prevented by starting proper wet curing as soon as possible. The concrete mix should be designed so that segregation does not occur. Keeping the slump less than 13 cm (5 in.) should reduce the tendency for segregation. Cement should not be sprinkled on the surface to absorb the bleed water and the concrete should not be finished while bleed water is present.

3.1.4 Early Freezing

Freezing of concrete at an early age should be prevented at all costs. If concrete freezes before it has gained about 2 MPa (300 psi) in compressive strength, it can sustain permanent damage. The value of 2 MPa (300 psi) is an indicator that the strength is high enough to withstand one freezing cycle, and also that sufficient free water has combined with the cement to reduce the freezable water to a manageable amount. However, to withstand multiple cycles of freeze-thaw, the concrete must have a compressive strength of at least 24 MPa (3500 psi) and an adequate air void system, as discussed in the following long-term exposure section on freezing and thawing.

3.1.5 Early-Age Heat of Hydration

Temperature changes induced internally by the hydration of portland cement and other cementitious materials significantly affect relatively young concrete. When large volumes of concrete are being placed in areas that do not permit adequate dissipation of hydration heat, the concrete may increase in temperature by as much as 56°C (100°F). A rule of thumb is that a temperature rise of 7 to 8°C (13 to 15°F) occurs per 45 kg (100 lbs) of portland cement per 0.76 cu m (1 cu yd) of concrete, starting at moderate construction temperatures. Such high temperatures, due to the heat of hydration, can create conditions that result in early-age cracks during the cooling.

3.2 Long-Term Exposures (Physical)

3.2.1 Concrete Drying Shrinkage

The loss of mixing water from newly cast concrete during exposure to air at less than 100 percent relative humidity (RH) causes drying shrinkage. Starting from the saturated condition, the relation between the amount of water lost and unrestrained drying shrinkage of new concrete is roughly linear through two distinct phases. In the first phase, the water that is lost consists primarily of free water. It is accompanied by a relatively small amount of shrinkage. During the second phase, the water loss consists primarily of adsorbed water. It is accompanied by a large amount of shrinkage. The adsorbed water is contained in capillary and gel pores. Drying shrinkage of unreinforced, unrestrained newly cast concrete in a 23°C (73°F), 50 percent RH environment can range from about 500 to 1000 microstrain (500 to 1000 $\times 10^{-6}$ in./in.). Concrete shrinkage in air having higher or lower RH can be significantly lower or greater than the typical range stated above.

Drying shrinkage was the most common cause of cracking in the nuclear plants surveyed, with 14 of 29 plants reporting problems. The majority of the drying shrinkage will occur within the first several years after construction, except for massive concrete members and those members partially covered with steel liner plates which may take decades. Continued cracking due to long-term drying effects is usually unanticipated, unless changes in the environmental exposure conditions occur or the element is massive.

The rate of water loss from concrete depends on the evaporation rate and the surface-to-volume ratio of the concrete members. Within the concrete, the rate of moisture loss varies inversely as the square of the distance from the nearest drying surface. The higher the surface to volume ratio, the faster the concrete members will dry and shrink. With a low surface to volume ratio, the concrete will dry more slowly. With very large members, the rate will be very slow. With rapid drying rates combined with very slow diffusion of mix water toward surfaces, due either to low permeability or large dimensions, a large degree of strain differential will develop in a large member. This is a result of the surface drying and shrinking, while the interior portions of the member remain at a high moisture content and therefore shrink much less. This can produce compressive stresses in the interior and tensile stress in the exterior portions. The tensile stresses

on the surface can result in cracking. The same phenomenon occurs in mass concrete due to cooling effects after concrete heat of hydration temperatures have reached their peak.

The shrinkage of reinforced concrete members is less than that of unreinforced concrete because the reinforcement restrains the shrinkage. Reinforcement tends to increase the number of cracks but greatly restricts their individual widths. Of course, more reinforcement means less overall shrinkage. The same is true with concrete aggregates; however, aggregate restraint is not as uniform or as powerful as that of steel. Some aggregates restrain shrinkage better than others. Thus, quartz, calcite and dolomite restrain the shrinkage better than do sandstones or the pure minerals horn-blend or pyroxene.

Once concrete has reached equilibrium with a given temperature and RH, its volume will remain stable until the humidity (or the temperature) changes. If the humidity decreases, the concrete will shrink. If the RH increases, the concrete will swell. These cyclic effects will continue to act whenever a change in humidity or moisture occurs; however, the magnitudes of change will diminish with time due to the irreversible shrinkage caused by atmospheric carbonation.

The amount of drying shrinkage a given concrete will experience depends on a large number of factors. The following list includes some of the factors that result in increased drying shrinkage:

high water content	aggregates that adhere to entrained air bubbles
high air content	aggregates with high absorption
high cement content	lightweight aggregates
aggregates with low moduli of elasticity	clay contaminants in aggregates
small maximum aggregate size	calcium chloride admixtures
high sand content	triethanolamine admixtures
fine cement	intermediate (3-7 days) length of initial moist curing
non-optimum SO_3 in cement	

The effects of shrinkage are often overlooked during design since the main emphasis is placed on compressive strength. The selection of concrete mixture materials can have a pronounced effect on the overall drying shrinkage of concrete slabs, as shown in Tables 3.3,^(3.17) 3.4,^(3.18) and 3.5.^(3.18) The construction practices can also affect drying shrinkage as shown in Table 3.6.^(3.19) These tables also show the cumulative effect of the various factors.

Careful selection of concrete materials can minimize drying shrinkage. Aggregate type and grading, subgrade preparation, w/c, concrete placement temperature, and curing all affect performance. To reduce the overall shrinkage, the largest practical maximum aggregate size should be specified, the aggregates and cement should have low shrinkage characteristics, fines and clay material passing the 200 mesh should be a minimum, and the least amount of cement to achieve the required compressive strength should be used.

The amount of shrinkage that occurs as concrete dries is dependent upon a number of other factors. These include amount and type of aggregate, amount of mixing water, properties and quantities of admixtures used, ambient temperature, RH and the method used for curing, and the time of drying. All types of cement cause concrete shrinkage, but some cause more than others, for example, if a cement is deficient in gypsum it will result in greater shrinkage than a cement that contains the optimum gypsum content. ASTM C596^(3.3) can be used to determine the effect of portland cement on the drying shrinkage of a graded Ottawa sand mortar subjected to standard temperature, RH, and rate of evaporation conditions. However, the drying shrinkage of paste or mortar may not be good predictors of the drying shrinkage of concrete.

Shrinkage of plain concrete (cement, aggregates, and water) is typically in the range of 0.04 to 0.08 percent, when the concrete is exposed to air at 50 percent RH. This means that concrete dried from an initially wet condition to a state of equilibrium with 23°C (73°F) air at 50 percent RH can shrink approximately 18 mm (0.72 in.) for each 30.5 m (100 ft) in length, on average. This same amount of shrinkage occurs when the temperature of concrete is reduced by 56°C (100°F). Therefore, the volume changes due to increased temperatures and decreased moisture contents can offset each other.

A means of combating the effects of drying shrinkage is the use of shrinkage-compensating cement. The precautions applicable to portland cement concrete apply as well to concrete made with shrinkage-compensating cement. Additional quality control testing must be done when using shrinkage compensating cement, including the ASTM C878^(3.3) concrete expansion test, prior to construction.

An area of concern often overlooked by designers is the curling of slabs-on-grade. Concrete slabs exposed to wet subgrades and dry atmospheres curl or warp due to the wet base and the dry top surface of the slab. Curling is usually upward, at the corners and edges. Heavy loads on the slab edges can cause cracks in these unsupported curled edges. Therefore, when casting a floor, the subgrade should be dry to minimize the moisture gradient and allow moisture to migrate out of the bottom of the slab as well as the top. The slabs should not be cast directly on an impervious clay base or plastic sheeting. If upward moisture movement is a potential problem, a minimum 50 mil thick impermeable vapor barrier, covered with 150 mm (3 in.) of gravel or crushed stone and blinded with at least a 13-mm (½-in.) thick layer of sand, is recommended.

3.2.2 Exposure to Seawater

Seven of the 29 plants responding to the survey noted deterioration of concrete due to seawater exposure, which causes both chemical and physical attack. The wave action causes abrasion and cavitation. Ingress of saltwater causes corrosion of the embedded reinforcing steel and other metals. The concrete may also be subjected to cyclic freezing damage, leaching, and damaging wet and dry cycles. Dissolution of the lime components can also occur.

3.2.3 Wetting and Drying

Disintegration by weathering is caused mainly by the disruptive action of freezing and thawing and by the expansion and contraction under restraint that results from temperature variations and alternate wetting and drying. Water is necessary for cement hydration; however, after the cement has hydrated, any remaining water becomes part of the mechanisms by which concrete disintegrates. Without water, freezing and thawing, corrosion of reinforcing steel, alkali-silica reactivity, sulfate or acid attack, ingress of soluble salts, and many other deteriorating mechanisms would not occur in hardened concrete.

The effects of wetting and drying can be very detrimental to hardened concrete. Wetting causes concrete to expand, and drying causes shrinkage. Expansion of moist concrete begins after initial set and, if the concrete is kept continuously moist, will continue slowly for years. This expansion is normally very small, seldom exceeding 0.025 percent, unless the cement content is very high or if the cement contains a high concentration of sulfur trioxide (SO₃) or free lime or magnesia. Contraction due to cooling or drying can result in tensile stresses that cannot be withstood by the concrete. Cracking then occurs, especially if the concrete is being restrained by an embedded substrate, attached steel, or a rigid external boundary condition.

Certain materials combine with cement to form relatively insoluble compounds that disrupt the concrete because their volume is greater than the volume of solids from which they were

formed. This disintegration involves a combination of chemical and physical forces. Where concrete is subjected to wetting and drying, certain salts, such as sodium sulfate, may cause surface disintegration by exerting crystallization pressure in the pores of the concrete. Such action appears to be purely physical, due to an in-situ increase in water content of the molecules. In highly impervious concretes this type of attack would be largely superficial. On the other hand, porous or highly permeable concrete may be affected throughout the mass. The most common of such agents include the sulfates of sodium, potassium, magnesium, and calcium. Sulfate reacts chemically with the calcium hydroxide and hydrated calcium aluminate in cement paste to form calcium sulfate and calcium sulfoaluminate, respectively. These reactions are accompanied by expansion and disruption of the paste, which, in turn, can disintegrate concrete. Deterioration due to sulfate-containing water is discussed in detail in the section on sulfate attack in this chapter.

In desalination plants, where concrete may be exposed to condensing water vapors or the dripping of distilled water, the concrete is rapidly attacked by the mineral-free liquid. Such pure water rapidly dissolves available lime and other soluble compounds of the cement matrix, resulting in deterioration and decomposition of the concrete. Concrete exposed to distilled or mineral-free liquids needs to be protected by coatings or lining materials that are not affected by the liquid.

Concrete in power plants needs to be protected from sea water and brine. In such environments, high quality concrete can normally protect itself from brine water at less than 93°C (200°F). Normal high quality concrete may be suitable even exposed to brine at temperatures as high as 121°C (250°F) if enough sacrificial concrete is in place to protect the cross section of the designed concrete member.

Prevention — Disintegration of concrete normally needs water either to initiate the chemical reaction or, in the case of freezing conditions, to saturate the concrete, before major deterioration occurs. After the concrete has matured, the drier it is maintained the more durable it will be.

Wetting and drying of exterior concrete in service is normal. In order to prevent damage to concrete due to wetting and drying, precautions need to be taken during construction. Cracks allow easy access of water into the concrete. As discussed, to reduce cracking due to drying shrinkage, use less water in the concrete, use large aggregates (38 mm [1½ in.]) to minimize cement paste content, place the concrete at as low a temperature as practical, dampen the subgrade and the forms, dampen aggregates if they are dry and sorptive, and provide an adequate amount of reinforcement to distribute and reduce the size of cracks if they occur. To reduce overall restraint, provide an adequate number of contraction joints. In severe service conditions, sealers and membranes should be considered.

3.2.4 Freezing and Thawing

Freeze-thaw damage to concrete is caused by the pressures resulting from the increase in volume during the phase change from water to ice. It can result in severe deterioration of non-air-entrained or marginally air-entrained concrete. Certain types of aggregates are also susceptible to damage from freezing and thawing. In the survey, 8 plants reported deterioration due to freeze-thaw.

Cooling towers in northern locations are in an especially aggressive environment and many have had less than satisfactory freeze-thaw performance. Any concrete in a freezing environment that may become saturated with water, or nearly so, is susceptible to freezing and thawing damage. Water in-take structures as well as any areas that may pond water, such as the flat surfaces of the containment dome or the ring girder of post-tensioned containments, are examples.

The phase change from water to ice is accompanied by a volume change of approximately 9 percent. If the pressures that can result from this volumetric change are not prevented, cracking and

deterioration of the concrete may occur. Air entrainment of concrete reduces or eliminates such damage by providing tiny pressure-relief sites, distributed uniformly throughout the concrete.

Concrete can be damaged if it is cast in cold weather and allowed to freeze. This results in early scaling of the surface, and the damage would soon be observed. Therefore, early freezing is not considered a long-term durability problem if identified and repaired early.

Freezing and thawing, while in a nearly saturated condition, is one of the most common deterioration mechanisms for hardened portland cement concrete. To explain this mechanism in more detail, it is necessary to review the accepted theories relating to the freeze-thaw deterioration of concrete. Assuming the use of durable aggregates (discussed below), the ability of concrete to resist damage by freezing and thawing is a function of the pore structure of the cement-paste phase and the moisture condition of the concrete when it is exposed to freezing.

A paper by T. C. Powers entitled, *Studies of the Physical Properties of Hardened Portland Cement Paste*^(3.20) describes the basic nature of the pore structure and its contribution to the physical properties of concrete. The following is a paraphrase of this Powers study from Philleo.^(3.21)

The principal finding of the Powers study was that when portland cement hydrates, the hydration product is a colloidal solid occupying about 2.2 unit volumes greater than the original volume. The hydrated mass is a "gel", about a quarter of which consists of very small pores. Based on this solid occupying 2.2 unit volumes, of which one was cement, then the water needed to hydrate the one unit of cement was 1.2 unit volumes or a 1.2 water-cement ratio by volume, which converts to a 0.38 water-cement ratio by mass. But cement will not hydrate unless in a relative humidity above about 85 percent; therefore, unless water is furnished from the outside, only 90 percent of the cement will be hydrated at a water-cement ratio of 0.38. Consequently, an additional 10 percent of water, resulting in a water-cement ratio of 0.42, would be needed for 100 percent cement hydration.

At any practical w/c, the gel formed contains pores so small that water within them will not freeze at normal temperatures. But between these gel formations are "capillary pores." These are much larger, so water in them will freeze at only a few degrees below the normal freezing temperature of water. Therefore, any concrete having a cement paste containing capillary pores can contain freezable water. If the volume of water in the capillary pores exceeds 91 percent of their volume, the expansion during freezing cannot be accommodated. If the capillary pores are fully saturated, the 9 percent expansion will exert an internal pressure of 100 MPa (15,000 psi) to 200 MPa (30,000 psi) on the concrete. Only a few cycles of freezing and thawing of such "critically saturated" concrete will destroy it.

The rate of freezing will affect both the velocity of water, moving within the capillary pores, and the advancement of the ice front. If air bubbles can be uniformly dispersed within the cement paste so as to produce a spacing factor of 0.2 mm (0.008 in.) or less, the concrete will accommodate the excess volume and little pressure will develop. The spacing factor, symbolized as L , represents the statistical maximum distance freezable water must travel to encounter an entrained air bubble. Thus, concrete containing an adequate percentage of small air bubbles has the ability to withstand the deteriorating effects of freezing and thawing.

Based upon ACI 301^(3.22) and our experience, concrete containing at least the following total air percentages, as tested in fresh concrete by either ASTM C231^(3.3) or ASTM C173,^(3.3) will be frost resistant.

Max. size of coarse aggregate, mm (in.)	10 (3/8)	13 (1/2)	19 (3/4)	25 (1)	38 (1 1/2)	51 (2)	76 (3)	152 (6)
Air content, %	8	7	6	5	4 1/2	4	3 1/2	3

Symptoms — Visual examination of concrete damaged by freezing and thawing may reveal surface scaling and delaminations parallel to the freezing surface. It can normally be detected early by viewing the corners or joints. The deterioration can range from light scaling, exposing only the surface sand grains, to heavy scaling where the coarse aggregate is exposed.

Reactors in the northern part of the United States are obviously most susceptible to freezing and thawing deterioration. Category I structures associated with the cooling water system are most susceptible to damage since the concrete is usually wet. Other locations of potential damage include areas where the concrete is intermittently wet, such as, in-take and discharge structures, on-grade structures, where water or rain ponds, or at leaking cracks. The deterioration usually is visually evident before significant loss of integrity of the member.

Prevention — ACI 201.2R^(3.23) describes techniques for preventing deterioration caused by freezing and thawing of the concrete in wet environments. It recommends:

- Designing the structure to minimize exposure of the concrete to high moisture contents
- Using low w/c concrete
- Using air entrainment
- Using suitable materials and frost resistant aggregates
- Providing adequate curing of the concrete
- Paying special attention to proper construction practices

It is also good construction practice to slope exposed surfaces to prevent water ponding. ACI 306, *Recommended Practice for Cold Weather Concreting*,^(3.24) discusses the required concrete maturity to resist damage to freezing and thawing. ASTM C33, *Standard Specification for Concrete Aggregates*,^(3.3) divides the country into severe, moderate and negligible weathering regions, as shown in Fig. 3.17.

Non-air-entrained concrete suffers most from freezing and thawing deterioration. Concretes used in reactor containment structures typically contain air-entraining admixtures that meet ASTM C260^(3.3) requirements. Scaling of non-air-entrained concrete can occur rapidly due to the application of sodium or calcium deicing salts. It can also occur if the finishing operation was performed while bleed water was on the surface. If bleed water is worked back into the surface of a slab, it results in a very high w/c, low strength top surface. Insufficient curing can also result in a weak surface layer susceptible to scaling.

To prevent scaling, the concrete should be adequately air-entrained and properly finished and cured. Do not use deicing salts such as calcium or sodium chloride on recently placed concrete. Never use ammonium sulfate or ammonium nitrate as a deicer because these attack the concrete chemically and result in deterioration.

The service life of non-air-entrained or moderately air-entrained concrete can be extended by drying the concrete and preventing additional moisture ingress. This can be done by applying sealers, membranes or coatings, so long as the concrete is not in contact with the ground or has other sources of moisture. Any method to extend the life of non-air-entrained concrete that is wetted in a freezing environment should be considered beneficial but not curative.

3.2.5 Erosion

ACI 116R^(3.25) defines erosion as a "progressive disintegration of a solid by the abrasion or cavitation action of gases, fluids, or solids in motion." Soil erosion usually occurs during a short period of time during which the erosion can be observed. Concrete, on the other hand, suffers from erosion over a much longer period. Usually months or years elapse before the erosion is evident. There are two primary causes of concrete erosion: abrasion and cavitation.

Abrasion erosion — Abrasion erosion is caused by repeated rubbing and grinding of particles on the concrete surface. Abrasion erosion damage might be found on slab or wall elements exposed to heavy traffic or moving water. The damage normally results in a gradual wearing away of the surface, exposing aggregates. In extreme cases, the aggregates are undercut and removed from the concrete matrix. In hydraulic structures, erosion results from the abrasive effects of waterborne gravel, rock, or other debris moving over a concrete surface during construction or operation of the structure. The condition is readily recognized by the well-worn appearance of the concrete. Abrasion erosion normally occurs when water is the transporting medium, carrying various abrasive materials that roll over, grind, or impact the concrete surface, but the abrasion can occur without water. Wind or gravity alone may be the force causing abrading material to rub against or impact the concrete.

Cavitation erosion — The second type of erosion is referred to as cavitation erosion. This type is due to repeated impact forces caused by the collapse of vapor bubbles in rapidly flowing water or other fluids. The damage caused by the collapse of the vapor bubbles is very different from the damage resulting from abrasion erosion. Instead of smooth, worn surfaces, the concrete will be very rough and deeply pitted. In severe cases, cavitation erosion may remove large quantities of concrete, endangering the integrity of the structure. Such damage, once started, accelerates because the damaged rough surfaces are more susceptible to the cavitation forces. Generally, fluid velocities exceeding 40 ft/sec (12.2 m/sec) can produce cavitation erosion in concrete.

Erosion or abrasion damage was reported by 4 of the 29 plants surveyed. Erosion of concrete piping was mentioned by 2 plants. The Category I structures possibly affected by erosion or cavitation are associated with water intake or discharge and cooling systems.

Symptoms — Damage from abrasion erosion results in worn and polished concrete surfaces. Such erosion does not normally in itself destroy concrete, but may contribute to more rapid deterioration due to reduced cover for reinforcing steel and to greater environmental exposure.

Structures damaged by cavitation erosion appear rough and severely pitted. The damage normally begins on the downstream edge of concrete joints; the surfaces are actually fractured, leaving severely pitted and extremely rough surfaces. Such damage normally requires immediate corrective action. It can rapidly destroy structures.

Prevention — A number of methods are available for preventing or at least reducing abrasion-erosion damage. To increase abrasion resistance, it is necessary to consider the hydraulic exposure and the use of the structure. Unless the adverse conditions that cause abrasion erosion damage are minimized or eliminated, it will be extremely difficult for any economical material currently available to prevent continued abrasion loss. The following methods may be helpful in preventing deterioration due to abrasion.

- a. For abrasion resistance, concrete should contain the maximum amount of the hardest available coarse aggregate and have the lowest practical water-cementitious materials ratio. The following table^(3.26) reflects the typical abrasion erosion loss attained from concretes containing various aggregates when tested in

accordance with the requirements of ASTM C1138, *Test Method for Abrasion Resistance of Concrete (Underwater Method)*. The test procedure involves subjecting the concrete specimens to abrasion erosion due to the wear and impact of steel grinding balls on the concrete surface. The steel grinding balls are propelled by water in the test container. The water is propelled by a submerged mixer paddle. Water velocity on the surface of the specimen is approximately 1.8 m/sec. (6 ft/sec). The abrasion erosion loss of concrete with a w/c between 0.4 and 0.7 containing various aggregates resulted in the ranking from best to worse: chert, trap rock, quartzite, and limestone.

If an increase in abrasion erosion resistance is desired, it may be beneficial to use silica fume as a mineral admixture, and a high range water reducer as a chemical admixture, in order to increase the strength of the concrete.

Typical Abrasion Erosion Data For Various Concretes (3.26)					
Material				Properties	
Aggregate source	Cement, kg/cu m (lbs/cu yd)	Silica fume, % of cement by mass	W/C	Compressive strength MPa (psi)	Abrasion erosion loss, % by mass
Limestone	317 (534)	0	0.45	39.4 (5,710)	6.9
Limestone	269 (454)	17.6	0.53	49.5 (7,180)	5.0
Limestone	351 (592)	42.9	0.21	95.5 (13,850)	2.2
Diabase (NY)	317 (534)	0	0.45	40.7 (5,910)	7.7
Diabase (VA)	317 (534)	0	0.45	39.1 (5,670)	6.1
Diabase (VA)	269 (454)	17.6	0.53	58.5 (8,480)	4.3
Diabase (VA)	351 (592)	42.9	0.21	95.2 (13,810)	2.3
Chert (MS)	346 (584)	0	0.45	32.7 (4,740)	4.1

- b. Steel fibers do not increase the abrasion erosion resistance of concrete; on the contrary, their use reduces the resistance slightly. It is theorized that the exposed fiber being vibrated by the water puts additional surface stress on the concrete causing a greater loss of the concrete mass. Loose fibers may also become an abrasive.
- c. The abrasion erosion resistance of vacuum-treated, polymer, polymer-impregnated, and polymer-portland cement concretes is significantly superior to that of comparable conventional concrete, but at an increased cost associated with materials, production, and placing.
- d. Surface coatings of polyurethanes, epoxy resin mortar, furan resin mortar, acrylic mortar, and iron aggregate toppings exhibit good abrasion resistance, but special problems may exist with surface preparation and thermal incompatibility, as well as increased cost.

Cavitation erosion damage cannot be prevented as easily as abrasion erosion damage. The high water velocity and surface irregularities create a vacuum on the concrete surface, which results in a tensile fracture. Consequently, prevention must be primarily concerned with reducing the water velocity, increasing the smoothness and streamlining of the surface and the concrete strength, and maintaining the front edge of joints approximately 1.5 mm (1/16 in.) above the downstream edge.

While proper material selection can increase the cavitation resistance of concrete, the only totally effective solution is to reduce or eliminate the causes of the cavitation. If this is not an option, the next best solution is to replace the damaged concrete with more cavitation-resistant materials. Like abrasion resistance, cavitation resistance can be increased by use of a properly designed, low w/c, high-strength concrete. Unlike abrasion erosion, cavitation resistance can be improved by the use of steel-fiber concrete. The steel fibers help resist the tensile forces associated with cavitation. They have been shown to be effective in reducing concrete loss due to cavitation in large hydraulic structures. Cavitation-resistant concrete needs hard, dense aggregate with less than 50 mm (2 in.) maximum size and good paste-to-aggregate bond. From a construction standpoint, tolerances have to be closely controlled, and the concrete needs to be finished to a smooth surface. Since proper curing primarily affects the quality of the top surface of the concrete, it is very important. Water curing is best. Curing membranes can be helpful, but they should be applied as soon as the water sheen leaves the surface, before the surface dries.

3.2.6 Irradiation

Irradiation of concrete by fast or thermal neutrons emitted by the reactor core or by gamma rays produced by the capture of neutrons by adjacent members can cause deterioration.^(3.27) Fast neutrons cause atomic displacements in certain types of aggregates such as flint. Gamma rays produce radiolysis of water within the concrete, which can affect the concrete's creep and shrinkage properties. Damage to concrete by prolonged exposure to radiation typically occurs as cracks and spalls of the surface. However, the shielding properties of the concrete are not significantly affected by such exposure.

Steel can be degraded by neutron irradiation caused by the displacement of atoms within the steel. Irradiation increases the yield strength, but reduces the ultimate tensile ductility and increases the ductile to brittle transition temperature.^(3.1,3.2) NUMARC^(3.2) concluded that damage to the embedded reinforcing steel due to radiation is not expected, since the radiation fluence and flux levels anticipated during normal operations is 10^{14} neutrons/cm², well below the threshold for degradation of 10^{19} neutrons/cm². Further research on the long-term effects of radiation on embedded steel has been suggested by Hookham.^(3.27)

None of the plants surveyed reported damage to concrete as a result of radiation. A threshold level for neutron fluence damage to concrete of 1×10^{19} neutrons/sq cm and 10^{10} rads, for gamma radiation, has been reported.^(3.1) NUMARC^(3.2) reported that the levels of neutron fluence in PWR at the containment wall would be less than this threshold and no larger than 10^{17} neutrons/sq cm. NUMARC also reported that gamma radiation on containment concrete is mitigated due to distance and shielding, making its effect insignificant. Hookham^(3.27) reports that radiation levels approaching these thresholds may occur in the Category I primary shield wall after 40 years or more of operation. Heat caused by radiation effects may also cause reduction in the mechanical properties of concrete, loss of moisture and volume changes. Research showed that the tensile strength may be decreased more significantly than the compressive strength when subjected to neutron radiation.^(3.28) The neutron radiation resistance depends on the mix proportions, type of cement, and type of aggregate in the concrete although specific recommendations could not be made. The radiation dose decreases rapidly with increasing distance from the exposed surface and normally has limited effect at distances more than 0.5 m (20 in.) from the exposed face. Change in concrete resistance to irradiation due to exposure magnitude and duration may be of importance for biological shield and prestressed concrete reactor vessel applications.^(3.28)

3.2.7 Temperature Changes

Category I structures are generally limited to design and operating temperatures of 65°C (150°F).^(3.27) Local areas at pipe penetrations may approach the temperature of the NSSS coolant, 345°C (655°F). The structures most likely to be affected by higher temperatures include the containment structures near NSSS equipment and containment wall hot piping penetrations. The containment and shield structures are also subject to diurnal and seasonal temperature effects.

The volume of concrete increases or decreases due to temperature changes. As in wetting and drying, the changes in volume due to temperature changes cause no problems unless the concrete is restrained. Three basic temperature change phenomena must be considered. First, there are the temperature changes that are generated internally by the early-age heat of hydration of the portland cement during curing in relatively large concrete placements. Second, there are the temperature changes that occur due to ambient conditions. Third, there are special cases of short-term temperature changes as discussed previously, such as fire damage.

The concrete compressive strength, tensile strength and modulus of elasticity may be reduced after prolonged exposure to marginally elevated temperatures. It has been suggested that strength reductions in excess of 10 percent can occur after prolonged exposure in the range of 82-93°C (180 to 200°F).^(3.29) The strength loss was minimized when the concrete is heated while in a loaded condition. In general, lean concrete mixes lost less strength than mixes with high cement contents. A greater strength loss occurred when the concrete was exposed to cyclic temperature changes than when it was exposed to a single temperature rise to the same temperature level. More mature or older concrete suffered less strength loss than younger concrete when exposed to elevated temperatures.

As mentioned in the Fire Damage section, mature concrete will withstand exposure to relatively high temperatures, but when quenched (cooled rapidly), can fail due to large superficial tensile stresses. Temperature rises in fresh concrete caused by the hydration of cement do not in themselves harm the concrete, but rapid cooling of the outer surfaces of the new concrete while the central mass is still hot can cause excessive tensile stresses and cracking. When the temperature drop exceeds about 15°C (25°F) per day, cracking can be expected.

Externally generated temperature changes can also disrupt the concrete. One of the best examples of this phenomenon is the high temperature of the surface of a pavement slab during an extremely hot summer day, while the underside remains relatively cool. Pavements are normally designed and constructed with joints to permit expansion, especially the expansion resulting from the heat of hydration of cement, and with contraction joints to control cracking during cooling. If these joints become filled with incompressible debris and the necessary space is no longer available to permit expansions due to temperature increases and the slab expands, it can lift itself off of the subbase, resulting in a blowup.

For example, a problem due to daily temperature changes arose when opposing elevated prestressed concrete girders for crane rails, oriented in a north-south direction, were exposed to sunlight and expanded and contracted at different times during daily temperature changes, causing the crane rails to separate. Such conditions can result in disasters. Therefore, such members should be either designed to allow for such movement or insulated to prevent extreme temperature variations.

In general, internal temperature changes during construction need to be controlled by lowering the placing temperatures and by insulating the concrete after the temperature has peaked to reduce the rate of cooling. Concrete elements exposed to external temperature changes need to be inspected and maintained by verifying that the expansion and contraction joints are operable and

do not contain rigid materials that will prevent the concrete from expanding. Extreme differential temperatures between opposite surfaces of concrete members must also be avoided to prevent warping.

3.2.8 Fatigue and Vibration

Fatigue and vibration may rapidly deteriorate concrete. Cyclic or fatigue loads can occur at locations such as NSSS equipment supports, pump and turbine supports, and local areas in the containment such as liner anchors. These fatigue loads may cause concrete to fail at a substantially lower load than that required to cause damage during a single load application. For example, the load required to cause a failure of the concrete at 10,000,000 cycles is approximately 55 percent of that required for failure in a single loading. Concrete can fatigue at any load level, with the main influence being the load range between the minimum and maximum load to which the member is subjected. As the load range increases, the number of cycles required for failure decreases. This loss of strength under repeated loading is sensitive to many influences including, but not limited to: load range, load rate, material properties, and environmental conditions.

Fatigue loads damage the concrete by causing microcracks to develop in the cement paste. The fatigue damage then continues indefinitely with age, as the microcracking grows and spreads throughout the concrete with each repeated loading. This microcracking also causes an overall loss of stiffness in the concrete. Fatigue of the mild reinforcing steel is not considered a problem, as the normally low levels of stress in the reinforcement are below the 138 MPa (20 ksi) fatigue threshold of the steel.

On a scale visible to the unaided eye, the fatigue damage manifests itself as large cracks that widen with age. These cracks and the loss in overall stiffness due to the microcracking can cause large deflections over time. In addition to the structural problems due to the overall weakening of the concrete, other problems can develop as the widening cracks expose the interior of the concrete to aggressive solutions. The cracks also expose the reinforcing steel to possible corrosion.

Fatigue problems typically become very obvious before any type of failure occurs, with large deflections and very wide cracks becoming noticeable. Also, fatigue problems are generally slow developing, allowing the damage to be detected before problems can occur.

As the design and endurance levels used by the nuclear industry are generally conservative, no deterioration due to overloading or fatigue was reported in the plant survey.

3.2.9 Creep

Creep is the slow, continued deformation in concrete subjected to continuous loads. The effect of creep is to reduce stress in the concrete under restraint. Creep can be advantageous, reducing the tendency for concrete to crack due to early thermal and shrinkage stresses. Creep is not usually a cause of distress, unless the design of the member is inadequate. Improper design may result in excessive deflections that may impose unintended loads on nonstructural elements, resulting in cracking or damage.

Creep is defined as the time dependent deformation due to sustained load. The magnitude of creep deformation may be 2 to 5 times the elastic deformation that occurs when the specimen is initially loaded. Creep affects long-term deformations and may reduce induced stresses in the concrete. It is usually advantageous in structures because it will enable stress concentrations to dissipate, reducing the tendency of a concrete to crack.

Creep of concrete cannot be measured directly. It is determined by subtracting shrinkage and instantaneous strains from the total deformation of loaded concrete specimens. In the determination

of creep, both loaded and non-loaded specimens are tested. The non-loaded specimens are used to determine shrinkage (or expansion). Instantaneous (or elastic) strain is determined by measuring the specimen deformation immediately after the load is applied. Creep is a time-dependent process; as the concrete under load ages, the rate of creep reduces. The relationship between shrinkage, elastic, and creep strain is shown in Fig. 3.18.

Several environments may be considered when measuring creep. Basic creep is the creep deformation that occurs without moisture movement from or to the specimen. This is usually obtained by sealing the specimens in copper sheeting prior to loading. Basic creep is observed in massive structures such as dams and very thick foundations and is independent of the specimen size.

The creep of specimens under drying conditions is called drying creep, the amount of which is dependent on the ambient environment, the size and the shape of the concrete member. Thinner specimens exhibit greater drying creep than thicker specimens. The amount of drying creep may be 2 to 3 times the basic creep obtained from sealed specimens that are prevented from drying.

Creep may also be divided into the quantity that is recoverable after the load is released from the structure and the amount that is not recoverable. Typical creep recovery is shown in Fig. 3.19.^(3.31,3.32)

Creep is dependent on many factors, including: mixture proportions, aggregate type, cement type, storage humidity and temperature, stress level, and size of specimen. It is generally assumed that the creep of concrete in compression is proportional to the applied load, up to 0.4 times the ultimate strength of the concrete. This stress level is typically greater than that used in design. As the concrete age at loading is increased, the amount of creep reduces. Creep is greater at lower humidities. Unfortunately, testing is rarely conducted at humidity levels less than 50 percent, which may be more typical of a nuclear plant.

Creep of concrete at higher temperatures is greatly increased. The rate of creep at 70°C (160°F) may be 3.5 times greater than the creep measured at 21°C (70°F). The amount of creep for specimens at 21, 46, 71 and 96°C (70, 115, 160, 205°F) is shown in Fig. 3.20.^(3.33) The influence of temperature over a wide range of values is shown in Fig. 3.21.^(3.34)

Typical problems observed involving creep of concrete are related to serviceability rather than strength. Unacceptable sagging of beams is common. It is usual to add an upward camber to prestressed concrete beams to accommodate future creep; however, inaccurate creep prediction may result in unacceptable camber of concrete beams. In normally reinforced concrete beams, use of top and bottom steel is necessary to reduce additional deformations due to creep. Unless openings or fixtures allow for creep deformations, cracking or load sharing may occur. Where loaded concrete is attached to stiffer structures (such as tanks, guide rails, and towers), problems due to creep of the concrete have been observed. Building cladding has sometimes been damaged due to shortening of reinforced concrete columns, and decorative tiles or concrete walls and columns have failed in a brittle and explosive manner due to creep-induced stresses. Non-load-bearing walls placed under concrete slabs should have sufficient clearance to prevent load being placed in the wall as the slab creeps.

Many different prediction methods for shrinkage and creep are available. ACI committee 209^(3.30) presents relatively simple formula for the prediction of shrinkage and creep. More complex formula are presented by Comité Européen du Béton (CEB)^(3.35) and Bazant, et al.^(3.36) Despite the number of prediction methods presented, the accuracy of prediction is poor. Typical coefficients of variation of ± 30 percent may be expected from any prediction method. If accurate creep prediction is required, the materials to be used on a particular project must be tested under expected service conditions.

3.3 Long-Term Exposures (Chemical)

3.3.1 Alkali-Reactive Aggregate

Alkali-aggregate reactions occur when susceptible aggregates in concrete react with alkalis (sodium and potassium hydroxides), which may come from the cement or outside sources. Alkali reactivity can be divided into two main categories: alkali-silica and alkali-carbonate reactions. The process usually is very slow, requiring 15 years or more to develop to a deleterious degree, and can be serious. Only one plant surveyed reported problems with aggregate reactions. However, WJE has done extensive work evaluating non-weather-exposed power plant structures with large-sized pedestals that have deteriorated due to alkali-silica reactions (ASR). Special attention should be given to these reactions due to their significance related to aging and long-term durability. Repairs of structures damaged by alkali-aggregate reactions typically have not been successful in stopping the deterioration. However, certain methods, such as sealing the concrete, have been used to reduce the reaction rates and thus extend the service life of the structure. Although alkali-aggregate reaction can be very detrimental to a concrete structure's capability to function as designed, it seldom destroys the structure. Under many circumstances, the reactive component of the aggregate is depleted before destruction occurs. Before replacing moderately deteriorated ASR-affected structures, the concrete should be petrographically evaluated and laboratory tested to determine if the reactivity has ceased.

Alkali-silica reactivity — Alkali-silica reactivity is more common than alkali-carbonate reactivity. The reactive forms of silica in aggregates are opal; chalcedony; micro-crystalline quartz; crystalline tridymite; and rhyolitic, dacitic, latitic, or other glass-like aggregates. These aggregates are highly siliceous, containing large amounts of silicon dioxide (SiO_2). Some of the slower reacting aggregates include granite gneisses, metamorphosed sub graywackes, and fractured and strained quartz. These reactive siliceous aggregates combine with alkalis and calcium hydroxide from the cement to form solid alkali-silica or calcium-alkali-silica gel complexes. The calcium-alkali-silica gel may be non-expansive, but the alkali-silica complex shrinks and swells with changes in moisture. Which of these compounds is formed depends upon the relative concentrations and surface areas of the reactive constituents.

Category I structures that are exposed to rain, groundwater, cooling water or high humidities are most susceptible to deterioration by ASR. These areas may include unprotected areas of the basemat and shell, any area in contact with groundwater, and areas of the dome and ring girder concrete. ASR is an important consideration for assessing the long-term durability of a structure, because the deterioration may not be evident for 20 to 30 years after construction, and since the deterioration is difficult to stop or mitigate. Map cracking, popouts and spalls are typical signs of ASR. Such indications should be taken seriously. Concrete susceptible to ASR should be identified early so that mitigation techniques can be applied as soon as possible. Repairs may be substantial if deterioration occurs and is allowed to continue, especially in Category I structures.

Distress due to aggregates reactive to the alkalis in portland cement was first identified by Stanton^(3.37) in 1940. He identified aggregates that were responsible for the deterioration of numerous structures in California. The problem has now been identified throughout the United States and in many other countries.

The magnitude of the problem is increasing, due to EPA restrictions on cement plant disposal of alkalis, and due to the depletion of good aggregate supplies. The states known to have structures affected by reactive aggregate prior to 1966 are shown in Fig. 3.22.^(3.38) In addition to those shown, most states west of the Mississippi River are now known to have structures affected by ASR. Although the problem is not new, more research is needed to understand the complex reactions responsible for the deterioration, and to provide rapid and accurate methods to identify potentially reactive aggregate sources.

The internal expansion caused by ASR usually results in "pattern" or "map" cracking, and in popouts of the exterior surface. However, the typical pattern cracking may not be present due to directional restraint, such as prestressing, which results in multiple cracking parallel to the prestressing. The cracks may be very wide at the surface but they generally narrow with depth. Dark reaction rims around the aggregates are often, but not always, found. The cracks can sometimes be traced to particular reactive coarse aggregate particles that have loosened from the cement paste. Reaction products form in the microcracks and may extrude to the surface in severe cases. However, no single manifestation can be taken as confirmation of alkali-aggregate reaction and examination by an experienced petrographer should be done, as well as laboratory tests.

The expansive alkali-silica material has some cementing properties while it is dry. However, it will imbibe water when available and expand, exerting high hydrostatic pressures within the cement paste, cracking the concrete and eventually increasing the dimensions of the structure. The gel may migrate through the pores and accumulate in cracks and voids. It can dry to a white powder, then return to a expansive gel when it becomes wet again (the white powder should not be confused with common efflorescence, caused by leaching and carbonation).

Marginally reactive aggregates often show damage after 20 or more years of service. Even when expansive reaction products have formed early in the structure's life, damage may not occur for many years due to the ability of the concrete to accommodate the expansion and movement of the gel within the paste. Entrained air can help to a minor degree by providing well-distributed voids.

Most nuclear plants have been constructed with materials selected using state-of-the-art methods at the time of construction. NUMARC^(3,2) reported that ASR was not a concern since petrographic examinations of aggregate were performed to eliminate reactive aggregate sources and when reactive aggregates were used, mitigation techniques such as using low alkali cement were incorporated. However, petrographic examination and mitigation measures are now known not to be completely reliable or effective.

Recently, additional rock types have been discovered to be reactive. Higher alkali cements have now been used in areas of the country that had not previously experienced, but now have, alkali-aggregate reactivity. The cement used in these areas today may be high alkali, not only because of the aforementioned EPA restriction but also because many cement companies import foreign cement for distribution in United States markets.

Prediction of the extent of deterioration resulting from using certain deleterious aggregates is difficult. The amount of expansion depends upon the nature of the reactive aggregate, the amounts of reactive material, the amount of alkali and moisture available, and the particle size of the reactive materials. In laboratory tests, maximum expansion occurred when the aggregate contained 5 to 10 percent opal with particle sizes between 0.25 to 0.05 mm (0.010 to 0.002 in.). Outside these limits the expansion decreased. The expansion is also a function of the alkali content, and detrimental expansion usually does not occur when the total available alkali by weight in the cement is less than 0.4 percent.

ASR is accelerated by alternate wetting and drying and by high temperatures. Seawater and other supplies of soluble alkalies are thought to aggravate the alkali-silica reaction by supplying chloride ions. Other types of deterioration, such as freezing and thawing, may occur at the same time as ASR. The combination of these factors can greatly increase the rate of damage. The alkali-silica reaction mechanisms are very complex, and the material percentages and particle sizes necessary to result in detrimental expansion vary with each type of aggregate.

Test methods (ASR): Identifying deleterious aggregates is difficult. The best way to avoid problems with alkali-silica reactive aggregates is to use only aggregates with good long-term field performance. This is not always practical, and often service records are not available.

The most reliable test method for potential reactivity is by petrographic examination, using ASTM C295.^(3.3) However, a trained petrographer experienced in reactive aggregate and concrete technology is required for this work. X-ray diffraction analyses can also be helpful. The grain size, microstructure, and surface area of the reactive particles will significantly affect the reaction rate and the resulting deterioration. Unfortunately, there is no agreement on the specific aggregate characteristics, so conflicting petrographic results may be obtained on marginally reactive sources. Further investigations are then required.

Fabricating mortar specimens with the proposed aggregate, then measuring the expansion over a 6-month period of time can provide good correlation to field performance of most (but not all) aggregates. This test procedure is referred to as the mortar bar test and is described in ASTM C227.^(3.3) However, the mortar bar test may underestimate the expansion of many types of aggregate and overestimate others. Work by Stark, et al.,^(3.39) has shown that the maximum allowable expansion of 0.1 percent at 6 months prescribed as the failure criteria may not correlate with the expansions at a later time. Many aggregates have been determined to be innocuous at 6 months, only to exceed the allowable expansion within 8 months and continue to expand over time. Also, coarse aggregates cannot be tested accurately since they must be crushed prior to fabricating the test specimens. This crushing exposes new surfaces and changes the particle size and, thus, no longer represents the field conditions. However, the mortar bar test and recent improvements to the test are still one of the best methods to assess the potential reactivity of an aggregate source, although it requires time that may not be available.

A "quick chemical" test, ASTM C289,^(3.3) is also available to determine the potential reactivity of aggregates. The aggregate is crushed and exposed to a solution of sodium hydroxide to determine the amount of silica and the reduction in alkalinity of the solution after a short reaction period. While the test procedure is rapid and useful, the results are often unreliable. The aggregates are crushed, several minerals will interfere with the test and the interpretation of the test results is difficult. The best advice may be to perform all three laboratory test and to base decisions on the composite results.

Mitigation techniques in new concrete: Current concrete specifications throughout the United States generally require the use of low-alkali cement if reactive aggregates are suspected. Typically, an upper limit of 0.60 percent is placed on the alkali in the cement. The expansions can be reduced by using a low alkali cement, but that alone is often not totally effective. Alkali can be present in the aggregates, or mix water, or can infiltrate the concrete from the environment. Also, alkali may concentrate in specific areas of the structure due to migration of alkali-laden moisture within the concrete. Low-alkali cement may not be locally available because cost and environmental concerns restrict its manufacture. Other approaches must then be taken.

An effective method of preventing detrimental expansion of potentially reactive aggregate is to add a suitable pozzolan, such as fly ash or silica fume, to the concrete. These admixtures increase the amount of available silica so the ratio of calcium to silica in the calcium silicate hydrates (CSH) formed during hydration is reduced. This provides sites within the CSH for sodium and potassium ions. Once these ions are removed from solution, the pH drops and the reaction product that forms is the less expansive calcium-alkali-silica complex. Typical quantities of pozzolans or fly ash used for this purpose are between 15 and 30 percent by weight of cement. Silica fume contains higher silica contents and can be effective at lower dosage rates. Generally, the higher the addition rate the greater the reduction in expansion. Care must be exercised when selecting a mineral admixture and its addition rate, because improper choices may result in increased expansion. Some fly ashes contain high amounts of alkalis, that will limit their effectiveness or even aggravate the ASR. Strength and

workability requirements also limit the practical dosage rates of the mineral admixtures. Silica fume is the most effective mineral admixture for preventing concrete expansion, followed by Type N natural pozzolans, and Type F fly ash; Type C (high calcium) fly ash is the least effective.

Alkali-carbonate reactivity (ACR) — Many carbonate rocks are reactive but few are expansive. The detrimental carbonate rocks are limited to dolomitic limestones that generally contain clay. The dolomite reacts with the alkali, forming calcite, brucite and alkali carbonate. The detrimental aggregates generally have an extremely fine-grained matrix. Dark rims that form around the reactive carbonate aggregate particles are more common than for ASR-affected particles.

Petrographic examinations and mortar bar tests are useful to identify potentially reactive carbonate aggregates, but chemical test procedures are meaningless. ASTM C586, *Potential Alkali Reactivity of Carbonate Rocks for Concrete (Rock Cylinder Method)*^(3.3) can be used to determine the potential reactivity of carbonate aggregates. The mortar bar method, ASTM C227, has been used, but the results are not always indicative of field performance.

Reducing the alkali content of the cement and incorporating mineral admixtures are often ineffective in preventing the expansion of alkali-carbonate reactive aggregates. Blending the reactive aggregate with non-reactive aggregates and reducing the maximum size of the aggregates has been found to reduce the expansion. Accurate identification of alkali-carbonate reactive aggregates has not always been attained, and mitigation measures have not been totally effective. Fortunately, alkali-carbonate reactive aggregates are not widespread.

Repair of concrete damaged by alkali-aggregate reactions — The repair of ASR damaged structures is not straightforward. Most repair methods must be considered as attempts to extend the service life of the structure. Practicality may dictate that the only solution is to remove and replace the structure. The current expansion rate can be tested by monitoring the concrete using strain gage points or by testing core samples in accelerated laboratory tests. The development of an effective repair strategy is dependent upon determining if the expansion and cracking will continue and if moisture can be reduced.

The most effective way to reduce the deterioration rate is to keep concrete dry. Without moisture, the ASR gel will desiccate and the expansion will cease. Sealers appear to be somewhat effective in slowing the deterioration, but the results have been variable. Drying a concrete structure can be difficult, especially for cooling towers or portions of a structure in contact with the ground.

Wetting and drying of the concrete accelerates the ASR deterioration. As water evaporates from the surface, additional water carrying soluble alkali is drawn through the concrete, concentrating the alkalis near the surface. The alkali content may become higher along surfaces, promoting increased alkali-silica expansion and deterioration. Core samples can be analyzed for alkalis to determine if such leaching has occurred, and to help predict future expansion.

Preventing the transfer of moisture from the ground through the concrete, such as by sealing the concrete surface, may reduce the overall expansion by eliminating the drying. This will reduce the alternate swelling and shrinking of the gel and stop the redistribution of soluble alkali due to moisture migration. Our experience with cooling towers has shown that when continuously wet, concrete containing silica gel does not always deteriorate. Those that are exposed to wetting and drying generally deteriorate quickly.

Water vapor pressure and residual ASR or ACR expansion will tend to blister and debond coatings or membranes applied to the surface. Even after the surface is sealed, time will be needed for the reaction rate to slow and, thus, some additional expansion, deterioration and repair to the coating must be expected.

Blight^(3.40) tested silicone, silane, resin-modified cement slurry, and a synthetic resin emulsion in laboratory and field tests to determine their effectiveness in slowing deterioration of concrete with ASR. Laboratory tests showed that the silane sealer was the most effective water-proofer. Silanes allow the concrete to dry since it is not a complete barrier to vapor. However, field tests indicated that in a semi-arid environment, such as that of South Africa, the untreated concrete dried so quickly sealing was not considered beneficial.

There is considerable risk in repairing concrete with ASR. Future cracking due to the expansion of the existing concrete must be expected, and maintenance will be required. However, if accelerated long-term testing indicates that the expansion has slowed, repairing the concrete may provide good long-term performance. Experimental and specialized rehabilitation techniques such as by vacuum impregnation with a monomer may be effective for limited areas. However, removing and replacing the reactive concrete is usually the best solution.

3.3.2 Leaching and Efflorescence

Ten of the 29 plants returning surveys reported problems of efflorescence. ACI 116^(3.25) defines efflorescence as "a deposit of salts, usually white, formed on a surface, the substance having emerged in solution from within either concrete or masonry and subsequently been precipitated by evaporation." It is unsightly but harmless unless it accumulates within the pores of the concrete. Such accumulation can disrupt the surface. The calcium hydroxide in the concrete, masonry or mortar can leach to the surface. Then, when exposed to the carbon dioxide in the atmosphere it becomes calcium carbonate, which appears as a whitish deposit. This is water-insoluble and must be removed with acid or by abrasion. Sodium and potassium hydroxides from the cement may also be deposited on the concrete surface. They quickly carbonate but, being water-soluble, they wash off easily.

The incidence of efflorescence is most prevalent where cracks are present in concrete water retention structures. Efflorescence may be reduced by reducing the amount of water passing through the concrete by thoroughly sealing the concrete surface or the cracks. Dense concrete that is well cured and has low permeability is less prone to leaching.

Leaching of concrete may occur when large volumes of water pass over a concrete surface, particularly water with high purity. Continued leaching causes surface etching and eventual disintegration. Saltwater dissolves the calcium hydroxide and some alumina compounds from the cement paste, eventually leaving only silica, iron oxide, and alumina. This process is usually very slow, unless water is continuously migrating through the concrete.

Dissolution of the lime compounds in concrete can occur by interaction with magnesium chloride, magnesium sulfate and carbon dioxide in seawater. The magnesium salts react with the concrete to form magnesium hydroxides and causing production of ettringite or its chloride isomorph. The ettringite formation may cause expansive pressures and cracking. Seawater (or other waters) with high contents of carbon dioxide tend to have low pH, which is aggressive to concrete and can soften the exposed surface.

Salts in soil that are in contact with a concrete or masonry structure that is unprotected by a moisture barrier can migrate upward through the concrete as much as 0.9 m (3 ft) and become deposited on the surface. Since leaching is controlled mostly by temperature, humidity and wind, the efflorescence forms more frequently in winter than summer, as the faster rate of evaporation indoors in winter permits greater migration of salts to the surface. Leaching over long periods of time can result in a concrete having a higher porosity making it more susceptible to attack by aggressive chemicals.

The following information on methods for removing efflorescence has been paraphrased from Technical Note CS-MR-4.3 in the U. S. Army Corps of Engineers REMR Notebook,^(3.41) and other references.^(3.42)

Under normal conditions, efflorescence can be removed by scrubbing with a dry brush or with water and a brush, light sandblasting, or light sandblasting and hosing with clean water. If the surface of the structure requires tender care for aesthetic reasons, it may be advisable to finish blast with such things as crushed nutshells, wood shavings, salt, or sugar, followed by a careful cleanup.

The efflorescence from water-insoluble salts can normally be removed with a dilute solution of acid. Suggested solutions include:

- 1 part hydrochloric acid in 9 to 19 parts water (muriatic acid)
- 1 part phosphoric acid in 9 parts water
- 1 part phosphoric acid plus 1 part acetic acid in 18 parts water

Prior to using these solutions on the surface of the structure, they should be tested on a small, inconspicuous area to determine their effectiveness and to refine the cleaning technique. Further, hydrochloric acid should be avoided on reinforced concrete since it may lead to future corrosion of the embedded steel.

Before applying these dilute acid solutions, the surface should first be saturated with clean water to prevent absorption of the acid into the concrete or masonry. Apply the dilute acid solutions to small areas, not more than about 0.4 sq m (4 sq ft) at a time. Wait 5 minutes, then remove the efflorescence with a stiff brush. Repeat if necessary, then wash the area with clean water to remove any unreacted acid.

Concrete that is heavily laden with efflorescence can be treated with a poultice of paper pulp and water. When the poultice dries, remove it. Reapply several times, if necessary.

From a safety standpoint, workmen using the acid solution should wear rubber gloves, safety glasses, and other protective clothing. All precautions on labels and MSDS sheets should be observed. Acids can be harmful to eyes, skin, and respiratory system.

To prevent recurrence of efflorescence on concrete or masonry surfaces, it is necessary to keep water absorption low and to restrict moisture movement through the concrete. The following recommendations may be helpful.

1. Use a moisture barrier—sealer, membrane, impermeable overlay—on exterior walls. (Caution: water-repellent surface treatments can reduce the amount of efflorescence; however, such treatments can cause the dissolved salts to be deposited beneath the surface, causing crypto florescence. If the concrete treated with a water-repellent contains large amounts of salt, and if these salts accumulate and crystallize beneath the membrane surface, flaking and spalling can occur).
2. Use wide, overhanging roofs to protect walls from rainfall.
3. Locate lawn sprinklers or other water sources so that walls are not subject to unnecessary wetting.
4. Divert moisture flow away from the structure.
5. Install drains to reduce high ground water table.

6. Use epoxy injection or chemical grouting at cracks, or flexible sealing materials in joints subject to efflorescence.

3.3.3 Salt Crystallization

Deterioration of concrete through salt crystallization, although occasionally reported in the literature, does not occur under ordinary circumstances. The proposed mechanism for deterioration is the formation of crystals that occupy a greater volume than the pores or capillaries in which they were deposited. This requires in-situ conversion of one hydrate state of a sulfate to another, which usually requires temperature or humidity changes that do not occur in ordinary concrete structures. WJE does not regard salt crystallization to be a major deterioration mechanism for concrete. "Salt crystallization" deterioration is really chemical attack, the conversion of the crystal type into another of larger volume, without going into solution. Repeated evaporation can cause salts to build up and increase the potential for cracking. Structures in contact with ground water or having fluctuating water levels are susceptible to the buildup of salts.

3.3.4 Sulfate Attack

Sulfate attack of portland cement concrete occurs when certain phases of the cement paste within hardened concrete react with sulfates, usually supplied from external sources, such as ground water. This reaction results in the formation of ettringite, which occupies a much greater volume than the original solid components. Depending mainly on the concrete w/c, the reaction may severely damage concrete within a few years.

Below-grade portions of concrete exposed to sulfate-containing soil or water are susceptible to attack. Sulfate levels of 1500 ppm or higher are considered aggressive to concrete by the U. S. Bureau of Reclamation. Deterioration has been seen in structures exposed to high sulfate ground waters and also in evaporative cooling towers, where sulfate and other ions concentrate as the water is continuously recycled or treated with sulfuric acid. The rate and degree of attack by sulfate-containing water or soil is dependent on the concentration of sulfate ions, the cation present (sodium, potassium, ammonium, or magnesium), the C_3A content of the cement, and the permeability and quality of the concrete. The deterioration can occur due to several reactions. The calcium hydroxide in the hardened concrete can be converted to calcium sulfate. The calcium aluminates can be converted to calcium sulfoaluminates or sulfoferrite. Finally, the calcium silicates in the hydrated paste can be decomposed. What reaction occurs depends mainly on what cation is present with the sulfate. The reactions can more than double the solid volume of a component, causing disruption and deterioration of the mortar or concrete. Attack by calcium or sodium sulfate solutions results in cracking and a soft and powdery surface. Attack by magnesium sulfate solutions initially results in a hard, dense surface skin, and subsequent deterioration of the surface.^(3.43)

Concrete can be manufactured to be resistant to sulfate attack by using an adequate cement content, a low C_3A cement, a mineral admixture such as fly ash, low water-to-cement ratio, good consolidation, and proper curing. Any action to reduce the permeability of the concrete will increase the sulfate resistance.

Repair of concrete deteriorated by sulfate attack requires removing all of the soft and deteriorated concrete by mechanical means or high pressure water jetting. The spalled areas can be built up with dry pack, patching, or overlay materials. The entire exposed area should be prepared and treated with a quality sealer or coating to prevent further exposure to the high sulfate waters or soils. Restoration of structures that have deteriorated through sulfate reactions has not been totally successful. Some rapid-setting repair materials contain both portland cement and plaster and self destruct due to internal sulfate attack. Such proprietary repair materials, which have been used at nuclear plants, are discussed in the repair section of this report.

3.3.5 Acid and Chemical Attack

Acidic deterioration of concrete is a common problem, although it is generally recommended that concrete not be exposed to acids. However, nuclear plants and spent fuel processing plants, must utilize acidic solutions. The containment of these solutions is critical. Acids rapidly react with the calcium hydroxides and carbonates at the surface of concrete, causing surface deterioration. Twelve of the 29 plants surveyed reported problems with concrete deterioration due to acid attack, commonly by boric acid.

Hookham^(3.27) presents a list of chemicals commonly found in nuclear power plants and their reactivity with concrete and embedded steel or liner plates, as shown in Table 3.7. Acidic groundwater exposure of below-grade Category I structures was also mentioned as a primary concern.

Both organic and mineral acids can aggressively attack concrete. It is very vulnerable to strong acids such as sulfuric, nitric, sulfurous, hydrochloric and hydrofluoric. Concrete is attacked, to a lesser degree, by weaker acids such as acetic, carbonic, and tannic, and many organic oils. Also, some substances that contain or produce acids will attack concrete. Examples include certain industrial wastes, silage, milk, and fruits and vegetables. Organic acids are often produced during the microbial fermentation of organics in water. Milk decomposes into lactic acid. Vegetables decompose into lactic and acetic acids. Also, hydrogen sulfide produced in sewage by fermentation is itself an acid and can also oxidize to form highly aggressive sulfuric acid. Acids such as sulfuric and carbonic acid can also be present in groundwater.

Attack of concrete by several species of bacteria and fungi can occur. This is most common in sewage structures. Most often, microorganisms oxidize sulfur-containing materials or, under anaerobic conditions, reduce them to hydrogen sulfide. The hydrogen sulfide can then be oxidized to sulfuric acid. Normally, deterioration occurs on the walls above the water level, due to condensation of the hydrogen sulfide gas.

In one instance of acid attack, tank walls and pipes in a new sewage treatment plant were being etched even though the effluent of all industries served by the treatment plant were required to be adjusted to a pH of at least 7.0. Sampling of the plant influent showed pH values dropping as low as 3.2 at various times during the year. These low pH periods coincided with the times when each of the major food crops grown in the area were harvested. Although the pH of the effluent from the canneries was above 7, anaerobic bacteria were decomposing the organic wastes on the way to the treatment plant, producing large amounts of aggressive organic acids.

A few acids may not aggressively attack concrete because they produce insoluble calcium salts that hinder the penetration of the acid. An example is oxalic acid, which can actually protect concrete from other acids, such as acetic. It is sometimes used to remove rust stains from concrete. However, it is poisonous and should be handled carefully.

Water containing ammonium or magnesium can ion exchange with the calcium in hydrated cement paste, resulting in a reduction of the concrete strength. Bases (chemicals of pH above 7) typically do not attack concrete, but may if their concentration reaches a sufficiently high level. Potassium hydroxide, for example, has been found to attack concrete at concentrations above 25 percent (pH above 14). Sulfate and nitrate salts of moderate to high concentrations cause slow disintegration by causing conversion of the aluminate phases to ettringite in the portland cement hydrate phase, or by producing very soluble substances. In general, good concrete is not adversely affected by petroleum oils or coal tar distillates.

The reaction between most acids and concrete results in soluble salts that can be leached from the concrete. This results in increased permeability, which intensifies the attack of acids or other aggressive solutions.

Acid cannot penetrate far into concrete before being neutralized by its high pH. Therefore, the interior of concrete cannot become deteriorated without the external surface of the concrete being severely deteriorated or destroyed. The rate of deterioration is based upon the type and concentration of the acid and the length of exposure. The rate of penetration is also affected by the permeability and the constituents of the concrete, and the quantity of neutralizing materials in the concrete such as calcium hydroxide and limestone aggregate. Acids that contain sulfate and sulfuric acid convert the calcium hydroxide to calcium sulfate, then cause further detrimental expansion due to the formation of ettringite. Other acids may not result in expansion and swelling, but may slowly dissolve the cement paste.

Acid resistant concretes — Acid resistance of portland cement concrete can be increased by reducing the permeability of the concrete and by selecting acid-resistant or sacrificial aggregates. High resistance to penetrating gases and liquids is achieved by having as few capillaries as possible. This means that concrete having the lowest water content and best compaction possible is best in resisting any type of chemical attack.

If strong acid resistance is desired, portland cement concrete should not be used, or if it is, it should be coated with a barrier material. The ACI 515LR, *A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete*,^(3,44) contains an extensive list of materials and their effect on concrete. The document also provides excellent advice on surface preparation, application procedures, and barrier system selection where such barriers are required.

3.3.6 Carbonation

Carbonation occurs when CO₂ present in the atmosphere reacts with moisture and hydration products. The main reaction is with calcium hydroxide Ca(OH)₂, forming calcium carbonate. Carbonation of portland cement paste has two distinct effects, one chemical and one physical. The chemical effect is to drop the pH of the pore liquid from approximately 13 to somewhat less than 9. The physical effects are irreversible shrinkage that can be equal to that due to drying and a moderate increase in strength. Calcium hydroxide crystals in noncarbonated concrete restrain the shrinkage. Dissolution of the crystals by carbonation removes this restraint, allowing more shrinkage to occur.

Carbonation was not considered a significant cause of deterioration in the nuclear power plants that were surveyed with only 1 plant reporting problems. However, carbonation is slow, especially in good quality concrete, and may not be evident for many years. Therefore, it may be a factor concerning the long-term durability and aging of concrete structures.

The carbonation process is slow because of the relatively low levels of carbon dioxide in the air (in non-urban areas, about 0.03 percent) and the normally low permeability of concrete. For example, a piece of concrete discovered in England in 1983 was traced to the son of Joseph Aspdin, the man who first patented portland cement and gave it its name. The concrete was 136 years old. The piece was sawed in half and immediately sprayed with the pH indicator phenolphthalein. Carbonation was seen to have progressed to a depth of about 5 mm (½ in.). This was very dense concrete, and the cement was coarsely ground. Modern concretes will range greatly in rate of carbonation; however, a good average estimate may be 6 mm (¼ in.) per decade. Carbonation rates are very dependent upon atmospheric moisture, being nearly zero near 0 or 100 percent RH, with much higher rates near 50 percent RH.

A common practice that causes severe carbonation is the use of salamander heaters for concrete placement indoors in cold weather. The high concentration of carbon dioxide caused by the open flame heater can attack fresh concrete flatwork and cause rapid carbonation. Carbon dioxide is heavier than air, so it concentrates near the floor. Carbonation in this case is more serious because the CO_2 precipitates the calcium hydroxide from solution, leaving only silica and alumina gels in place of the normal calcium aluminate and calcium silicate hydrates. The gels have no strength; therefore, a soft surface results. Carbon dioxide producing heaters are still being used to heat enclosures for casting concrete, but the practice is slowly diminishing as specification writers become aware of the problem. Combustion products should always be vented to the outside, through heat exchangers.

The chemical effect of concrete carbonation is important in that the drop of pH from 13 to less than 9 changes any embedded steel system from an anodically controlled, passive condition, to one in which the corrosion can become active. This is not the rapid, chloride-instigated electrochemical type of corrosion that produces such quick destruction in bridge decks, parking structures, and marine buildings. But, given sufficient time, the same sort of damage can be done by corrosion of the steel in carbonated concrete. In chloride-free concrete, if the pH decreases below about 10, the passive surface oxide layer can be destroyed and corrosion of the steel may occur. This type of corrosion is usually termed atmospheric corrosion versus chloride-induced corrosion.

3.3.7 Corrosion of Embedded Steel

One of the major mechanisms of deterioration in concrete structures is corrosion of embedded reinforcing steel. Ten of the 29 plants surveyed reported such damage. Structures in contact with salt or sea water or chloride-containing ground water are most susceptible to corrosion. Structures subjected to airborne saltwater spray are also susceptible to corrosion, but to a much lower degree. Cracks in structures will increase the potential for corrosion.

Corrosion is the attempt of the metal to return to its natural state, usually the oxide. The corrosion products of steel occupy a greater volume than the original steel, typically from 3 to 7 times greater. This increased volume strains the surrounding concrete and may result in cracks, delamination, and spalling of the concrete cover over the corroding steel.

Corrosion of embedded steel can be caused by the galvanic action of two dissimilar metals in contact, or by concentration cells of chlorides or oxygen along the length of a bar. When two different metals are attached, a corrosion cell can develop. Each type of metal or alloy has a certain energy associated with, as shown on the electromotive or galvanic series, the order of how noble the metal is. If two metals are attached in an electrolyte, such as wet concrete, the most noble metal becomes cathodic, the least noble becomes anodic and corrodes. Aluminum, for example, is less noble than iron. The aluminum will corrode rapidly if connected to iron in concrete, especially if chloride is present.

Corrosion in structures is most commonly caused by ingress of deicing chemicals or sea water, or by the addition of chlorides into fresh concrete. Calcium chloride was commonly used as an accelerator for concrete before its detrimental effects on steel were identified. Chlorides can also be present in the aggregates or mix water, especially if coastal aggregates or seawater are used in the concrete. Chloride in the concrete also tends to cause the concrete to hold more moisture, reducing the resistivity of the concrete and aggravating corrosion.

If chloride ions are present in sufficient amounts in concrete, the normal passivated film on the steel surface can be disrupted. Deterioration of the embedded steel may then occur, even at high pH levels. The threshold for corrosion of steel is considered to be between 0.02 and 0.03 per cent acid soluble chloride ion by weight of concrete.

The useful service life of a concrete structure may be a function of the corrosion rate of the reinforcement. The total service life is the sum of the initiation period and the deterioration period, where the initiation process is the carbonation or the chloride penetration of the concrete cover, while the deterioration period is the time required for debilitating corrosion of the reinforcement.

Initiation process – Carbonation is a result of the diffusion of CO_2 through air-filled pores. While CO_2 needs air-filled pores for penetration, chloride spreads by diffusion through totally or partially water-filled pores. Wetting and drying of the concrete surface with water containing chloride will cause a rapid ingress of chloride due to absorption. As a result of diffusion, the chloride concentration will decrease towards the interior of the concrete. The chloride profile in the concrete can be established in structures by standard chloride titration tests in the laboratory, on core or powder samples removed from the structure.

Both carbon dioxide and chlorides penetrate faster toward the reinforcing through cracks. Narrow cracks, <0.1 mm (<0.004 in.), might be self-healing due to deposition of hydration products. However, wider cracks that do not heal often have a high chloride concentration at the root of the crack and near the reinforcing. Cracks that are transverse to the reinforcing are sometimes less harmful than cracks along the length of the steel bar, since they only expose a small area of the steel to deterioration and the steel is in a position to restrict their widths.

Deterioration process – Corrosion of reinforcing steel in concrete can be divided into two processes, the anodic and the cathodic. The anodic process is the dissolution of metallic iron atoms to ferrous ions after the effectiveness of the protective layer at the surface of the reinforcement has been destroyed. The cathodic process is the reduction of oxygen by reaction with water and free electrons to form hydroxyl (OH^-) ions. The anode and cathode are separated by a distance that can be a micron or several feet. Oxygen is required only at the cathode; it removes electrons from the steel that were liberated during the oxidation of the iron at the anode. The anodic and cathodic areas may alternate along a continuous reinforcing steel bar, or represent the top and bottom layers of steel in a slab.

The typical chloride induced corrosion reactions are as follows.^(3,45) If steel is placed in fresh concrete, the initial reaction will be to stabilize the ferrous ion concentration. This will occur because concrete (or mortar) quickly saturates with hydroxyl ion, and the following reaction occurs:



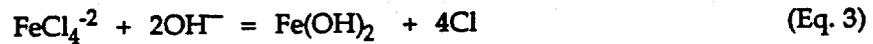
The reaction produces a thin layer of ferrous hydroxide at the steel surface. Passivation follows, possibly after reaction of the ferrous hydroxide to form other products such as gamma iron oxide. The hydroxyl ion concentration then becomes stable due to the abundant supply from the hydrated cement. Introduction of sufficient chloride ion at the steel surface can destroy this passivation, probably by complexing the ferrous ion as follows:



The reaction with chloride removes the ferrous ion from the system, causing the steel to corrode further to produce more ferrous ion to satisfy the chemical equilibrium in Eq. 1 and that in Eq. 2.

Corrosion would stop as soon as sufficient chloride is complexed to satisfy equilibrium, but two reactions work against such cessation. First, additional chloride ions may migrate to the steel surface, either because a continuous external supply of chloride is introduced into the concrete (for example, by deicer salts), or because chloride ions, which have a negative charge, migrate from other areas of the concrete due to the electrical attraction of the positive ferrous ions. Second, the ferrous

chloride complex ion can migrate away from the steel and exchange with the enormous concentration of hydroxyl ions in the cement paste as follows:



This reaction precipitates out rust (ferrous hydroxide is one form) and releases the chloride ions to migrate back to the steel to complex more ferrous ions. The consequences of these reactions are numerous. Among them are:

Corrosion will continue as long as the ferrous chloride complex can move away from the steel surface.

The half-cell potential will become more negative because the ferrous ion concentration is lowered from that of Eq. 1 to that necessary to satisfy both Eq. 1 and Eq. 2, as determined by simultaneous solution of the equations.

Corrosion rate and corrosion potential will, for a certain period, be a function of chloride ion concentration and therefore time of exposure to chloride salts.

Cracks will lead to decreased ferrous ion concentration, and therefore a more negative potential, due to introduction of air and therefore oxygen. Cracks also allow direct access of chlorides and carbon dioxide to the steel.

The rate of corrosion is controlled by the conductivity of the concrete, the difference in potential or voltage between the anodic and cathodic areas, and the rate at which oxygen reaches the cathode. Concrete with low permeability contains less water under a given exposure and, therefore, should have a lower electrical conductivity and provide better resistance to corrosion.

Corrosion is optimum at a humidity of 70 to 80 percent and will generally cease if the humidity within the concrete drops below 40 percent. At higher humidities or in submerged concrete, corrosion is slowed due to the lack of oxygen at the cathode. However, when anodes are in high humidity concrete and cathodes in lower humidity, such as in a tidal area, severe corrosion may occur.

Cracks in concrete subjected to external chlorides or carbonation should be repaired to prevent corrosion of the embedded steel. Increased cover over the steel significantly improves resistance to chloride ion ingress. A minimum of 50 mm (2 in.) cover should be used for concrete exposed to marine environments. Epoxy-coated or stainless reinforcing steel provide excellent protection from chloride-induced corrosion. It is important that the epoxy coating be properly applied and that care is taken to prevent or repair damage to the coating. The best protection against corrosion of embedded steel in new construction is to utilize epoxy-coated steel, increase the cover, and install crack-free concrete utilizing prestressed concrete. Admixtures to reduce the permeability of the concrete also help, but to a lesser degree. Existing structures can be protected by encasing the member in additional concrete or by coating the structure with a penetrating sealer, coating, or membrane.

Discussion of chloride ion thresholds — Acid-soluble chloride analysis is most commonly used to determine the chloride content of concrete. Investigations of the chloride content of concrete adjacent to corroding reinforcing steel indicate that acid-soluble chloride threshold concentration to be approximately 0.59 to 0.83 kg/cu m (1.0 to 1.4 lbs) of chloride per cu yd of concrete. Water-soluble chloride content may be more meaningful, since only the water soluble chloride is available to cause corrosion; however, difficulties in testing for water-soluble chloride leads to problems and a lack of reproducibility.

It is common to analyze for chlorides in concrete at the level of the rebar to determine whether the "threshold" for active corrosion has been reached. The concentration of chloride required may be related to the pH of the porewater in the hardened cement paste. The threshold appears to be about 0.15 percent water-soluble chloride ion by weight of cement (porewater pH = 12.6). The Federal Highway Administration recommends a similar value.

A practical value, based on experience with buildings in a temperate climate, is 0.25 percent by weight of cement or 0.83 kg/cu m (1.4 lbs/cu yd) for typical normal weight concrete, weighing 2300 kg/cu m (145 lbs/sq ft).

The ACI 318-89^(3.46) code limits (Table 4.3.1 of the Code) for the water-soluble chloride ion in newly placed hardened concrete are shown below as measured by AASHTO T260.^(3.47)

Maximum Chloride ion Content for Corrosion Protection		
Type of member	Maximum water-soluble chloride ion in concrete (by wt of cement)	Allowable admixed calcium chloride dihydrate (by wt of cement)
Prestressed concrete	0.06	0.25
Reinforced concrete exposed to chloride in service	0.15	0.62
Reinforced concrete that will be dry or protected from moisture in service	1.00	4.16
Other reinforced concrete construction	0.30	1.25

The three exposures for reinforced concrete in ACI 318^(3.46) Table 4.3.1 are confusing, and WJE believes the current limits are not correct. WJE is of the opinion that the use of 4.16 percent calcium chloride dihydrate by weight of cement in concrete that may be dry or protected from moisture is too risky. Deteriorated lightweight concrete columns, balconies, and exposed floor slabs investigated by WJE contained calcium chloride dihydrate percentages of 1.32 to 2.25 percent by weight of cement. The ACI 318 limits for "reinforced concrete exposed to chloride in-service" and "other reinforced concrete construction" are also far too high.

The 1987 Ontario Ministry of Transportation and Communications Technical Report, *Chloride Corrosion Threshold in Concrete*, by Hope and Ip^(3.48) presents data on admixed calcium chloride dihydrate in relation to corrosion of mild steel reinforced concrete. The test results show the admixed calcium chloride dihydrate corrosion threshold limit to be between 0.2 and 0.4 percent by weight of cement. Investigations by WJE and others generally agree with those limits. Based on experience, we have concluded that the higher admixed calcium chloride contents allowed by ACI can create corrosion of interior concrete members as well as exterior mild steel reinforced concrete members.

A 1985 paper entitled, *The Determination of the Chloride Content of Concrete* by Hope, Page and Poland^(3.49) showed that high quantities of chloride from their Ontario naturally chloride-contaminated aggregates do not create corrosion problems since the chloride is bound in the aggregate. WJE also investigated a 12-year old major high-rise building with normal weight concrete columns and lightweight concrete floor slabs in 1983. The normal weight concrete columns had calcium chloride dihydrate contents equivalent to 0.45 to 0.81 percent by weight of cement. The columns were performing satisfactorily. Their chlorides, however, are now known to have been derived from the local limestone aggregates; so the chlorides were bound within the aggregates, and not available for corrosion.

The interpretation of chloride ion test results can be difficult. WJE and its Erlin, Hime Associates Division (EHA) performed a test series on two different Ontario aggregates in 1991. These 1991 tests show that whenever the aggregates are tested by methods that include fine crushing, the crushing process liberates from 20 to 50 percent of the bound chlorides in the aggregates and overestimates the corrosion potential from the chloride ion content. These tests on aggregate and concrete by WJE/EHA, Hope, et al.,^(3.49) illustrate that the chloride content of hardened concrete and aggregates is highly dependent upon the crushing method and test techniques and the results must be interpreted with caution.

3.3.8 Corrosion of Non-Ferrous Metals

The most common deterioration of reinforced concrete in a high chloride environment is corrosion of the embedded reinforcing steel. However, corrosion of other metals can occur due to reactions with the portland cement or by exposure to chlorides.

Wet concrete will corrode metallic lead due to attack by calcium hydroxide. If the lead is embedded, only the surface is attacked and corrosion should cease after the concrete hardens. If the lead is only half embedded, with the other half exposed to the environment, the electrolysis effect known as differential aeration occurs and the lead may gradually disintegrate. Embedded lead parts can be protected by coating with a varnish or asphalt.

Aluminum is attacked by the alkalis in fresh concrete. The corrosion products do not protect the metal, and the corrosion may be progressive. Deterioration results in blister formation or pits and perforations. Aluminum should always be coated with a varnish or paint prior to its contact with concrete.

Zinc, such as on galvanized steel, is initially attacked by wet portland cement but the surface quickly passivates and is typically not attacked further. However, under certain conditions where chloride and moisture are available, compounds such as zinc hydroxychloride can be formed, resulting in expansion and spalling of the concrete. This has been identified as a problem in buildings where zinc-coated anchors have been embedded in mortar containing admixed chloride. Spalls caused by embedded zinc-coated anchors should be thoroughly evaluated to determine the cause of the failure. An appropriate repair can then be designed. Copper is tolerant of alkalis and so is not attacked by concrete. However corrosion can occur if chlorides and moisture are available.

As discussed, uncoated dissimilar metals should never be placed in contact or embedded near each other in concrete due to the possibility of galvanic corrosion. Embedments of metals, other than mild steel, should be electrically isolated from the reinforcing mats to prevent galvanic corrosion.

3.3.9 Corrosion of Prestressing Strand

Corrosion of prestressing stand and prestressing losses were determined to be a potentially significant degradation mechanism by NUMARC.^(3.2) The corrosion of prestressing can be managed

by programs of visual inspection of tendon anchor heads as well as periodic examination of the corrosion protection medium. Loss of prestressing can be detected by inspection and load monitoring programs. ASME Section XI, Subsection IWL ^(3.50) and the NRC Regulatory Guide 1.35 ^(3.51) should be referenced concerning the evaluation procedures for prestressing loss and corrosion.

Performance of prestressing systems has been quite good. Naus^(3.1) reports that the performance of prestressing tendons have been good, however, stress corrosion cracking has been seen in several plants. Most ungrouted tendons have performed well with minimal corrosion or wire failures. Generally, the grease and corrosion inhibitors within the ducts are working well.

Corrosion of prestressed steel may be aggravated by hydrogen embrittlement, fatigue cracking, moisture access to bundled tendons, and stress corrosion. Elevated temperatures on prestressing steel have a far greater consequence than on mild steel because the highly stressed steel may undergo stress relief (relaxation and creep). This effectively reduces the compressive forces that are transmitted to the concrete, which may lead to structural problems.

A number of recent and significant studies on corrosion of 7-wire prestressing strand have concluded that the time-to-corrosion and the chloride ion corrosion threshold for strand are both substantially greater than for conventional reinforcing bar. The 1987 FHWA report, *Protective Systems for New Prestressed and Substructure Concrete* by Pfeifer^(3.52) discussed these observations. A conclusion from this study was as follows:

Corrosion activity of unstressed gray prestressing strands with 1-in. cover in moist-cured concrete began when the chloride content at the strand level reached an average of almost 1.2 percent by weight of portland cement (acid-soluble). This unusually high corrosion threshold level is 6 times greater than that determined for gray bars. In addition, the average time-to-corrosion for these unstressed prestressing strands was 3 times as long as that measured for gray bars.

These data suggest that the unstressed 7-wire strand were able to tolerate approximately 6 times the reported reinforcing bar chloride ion corrosion threshold which was about 0.20 percent (acid-soluble) chloride ion by weight of cement.

The firm of Kenneth C. Clear Inc. (KCC) reported similar test results on unstressed prestressing strand and reinforcing bar in March 1991 to the ACI Task Group revising ACI 222-85. The KCC long-term corrosion study started in 1984. The test utilized a structural lightweight aggregate concrete. The reinforced concrete specimens, 300 by 300 by 125 mm (12 by 12 by 5 in.) were the typical slabs tested in the 48-week southern exposure test method described in NCHRP 244,^(3.53) as well as used in the FHWA RD-86/193 report.^(3.52) During this 48-week test period, KCC determined that the control specimens (conventional bar and strand) initiated corrosion activity as follows:

<u>Steel type</u>	<u>Average time-to-corrosion</u>
Rebar	7.5 weeks
7-wire strand	>42 weeks

The KCC time-to-corrosion period for mild steel is similar to that observed in the FHWA study.^(3.52) The KCC 42-week or greater time-to-corrosion period for prestressing strand is longer than the 19 to 19.5 weeks reported by WJE,^(3.52) from 4 slabs of normal weight concrete with unstressed strand. However, the KCC and WJE studies show that prestressing strand can tolerate from 3 to at least 5 times the cyclic corrosion testing prior to starting corrosion compared to steel reinforcing bars. Also, the chloride ion content at the level of the strand may be 6 times that of rebar when the strand begins to corrode.

After the KCC 48-week cyclic wet/dry test period, the slabs were stored out-of-doors in Herndon, Virginia and subjected to natural weathering on above-ground racks. These actively corroding slabs were observed for signs of cracking from August 1985 (when moved outside) to March 1990. The observed time-to-cracking was as follows:

<u>Steel type</u>	<u>Time-to-cracking</u>
Rebar	1 to 3.5 years
Strand	4.4 to 5.6 years

This shows that the rebar also caused cracking much sooner than strand.

A paper by Stark^(3.54) also showed that the 0.06 percent water-soluble chloride ion content by weight of cement shown in ACI 318 is not the corrosion threshold level for prestressed concrete. In fact, this research for PCI could not identify a reproducible chloride-ion threshold level for prestress steel for the various environmental conditions evaluated during this 1984 study. However, this paper suggested that the permissible water-soluble chloride content should not exceed 0.10 percent by weight of cement for prestressed concrete.

The WJE monthly progress report to FHWA,^(3.50) for October 1983, included the following observations concerning prestressing strand. The uncoated prestressing strand possesses a natural corrosion protection after the manufacturing process. During the wire drawing processes, the wires are coated with zinc phosphate and calcium stearate materials which impart an unintentional but effective corrosion protection system directly onto the assembled 7-wire strand. These materials are also very difficult to remove. Therefore, prestressing strand can possess a significant corrosion protection system from lubricants used to draw and manufacture strand. Similar lubricants are not used to manufacture reinforcing bars.

Ultrasonic cleaning is not considered adequate to remove the lubricants that are bonded to the steel wire. The 1987 FHWA^(3.51) final report on these corrosion tests on "as-received" and "ultrasonically cleaned" strand show that the cleaning process did not alter the chloride ion content corrosion threshold levels, which were 1.18 and 1.36 percent by weight of cement (acid-soluble), respectively. These values were 6 times that of rebar. The cumulative tests strongly suggest that strand and wires contain a natural corrosion-protection system which may be due to calcium stearate and zinc phosphate materials on the strand or wires.

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Table 3.1 — Primary Degradation Factors That Can Impact the Performance of Category I Concrete Structures

Material system	Degradation factor
Concrete	<p><u>Chemical attack</u> Efflorescence and leaching Salt crystallization Alkali-aggregate reactions* Sulfate attack Bases and acids</p> <p><u>Physical attack</u> Freeze/thaw cycling Thermal exposure/thermal cycling Irradiation Abrasion/erosion/cavitation Fatigue/vibration</p>
Mild steel reinforcing	Corrosion Elevated temperature Irradiation Fatigue
Prestressing	Corrosion Elevated temperature Irradiation Fatigue Loss of prestressing force
Liner structural steel	Corrosion Fatigue

*Includes reaction of cement-aggregate and carbonate aggregate

Table 3.2 — Critical Areas in Nuclear Power Plants Where Concrete Materials May Exhibit Deterioration^(3,27)

Material system	Degradation factor	Potential areas of deterioration
Concrete	Chemical attack	<ul style="list-style-type: none"> • Subterranean areas • Surfaces exposed to cooling water • In-containment floors/slabs • Containment/shield, auxiliary building structures (acid rain, ocean atmosphere, alkali aggregate reaction)
	Freeze/thaw cycling	<ul style="list-style-type: none"> • External structures where water may collect • Intake/discharge structures, particularly at water line of cooling water source
	Thermal exposure/thermal cycling	<ul style="list-style-type: none"> • Containment/shield structures (diurnal and seasonal effects) • Areas located near reactor coolant pressure boundary or hot piping systems
	Irradiation	<ul style="list-style-type: none"> • In-containment structures proximate to reactor coolant pressure boundary • Localized areas of specific containment design • Floor and slab elements
	Abrasion/erosion/cavitation	<ul style="list-style-type: none"> • Cooling water intake or discharge structures
	Fatigue/vibration	<ul style="list-style-type: none"> • Local areas in containment (e.g., near liner anchors) • Local areas under equipment supports
Mild steel reinforcement	Corrosion	<ul style="list-style-type: none"> • Outer layer of conventional steel reinforcing in all structures
	Irradiation	<ul style="list-style-type: none"> • In-containment structures proximate to reactor coolant pressure boundary
	Fatigue	<ul style="list-style-type: none"> • Local areas in structures subjected to repeated equipment loads
Prestressing	Corrosion relaxation	<ul style="list-style-type: none"> • Containment buildings • Fuel pool structures

Table 3.3 — Individual and Cumulative Effects of Various Factors on Concrete Shrinkage From Powers^(3.17)

Factor		Effect*	
Favorable	Unfavorable	Individual	Cumulative
Cement of optimum SO ₃	SO ₃ deficiency	1.5	1.5
Cement with 15 percent retained on No. 200	0 percent retained on No. 200	1.25	1.9
Less compressible aggregate (quartz)	More compressible (Elgin gravel)	1.25	2.4
More aggregate (1½-in. max. size)	Less aggregate (¼-in. max. size)	1.3	3.1
More aggregate (stiff mixture)	Less aggregate (wet mixture)	1.2	3.7
No clay in aggregate	Much bad clay in aggregate	2.0	7.4

*Multiplication factor for potential increase in shrinkage

Table 3.4 — Effect of Various Factors on Concrete Shrinkage From Meininger^(3.18)

Factor	Maximum effect, %
Coarse aggregate source effect	100
Fine aggregate source effect	20
Total aggregate source effect	150
Washing out minus No. 200 mesh	15
2½ vs. ¾-in. max. aggregate size	25
Fine aggregate grading fine	0
Cement source	15
Cement factor	10
Slump	5
Curing: 7 days vs. 3 days	5

Table 3.5 — Effect of Coarse Aggregate on Drying Shrinkage of Concrete*
 From Meininger^(3.18)

Coarse aggregate rock types	Shrinkage, millionths** drying period	
	7 days	182 days
Quartz	180	530
Igneous, andesite, sandstone	180	560
Greywacke, quartz, limestone, granite	200	620
Granite, quartzite	220	640
Schist, granite gneiss	210	660
Impure limestone, sandstone, igneous	230	640
Ingeous	210	700
Sandstone, limestone	240	700
Granite, granite gneiss	240	750
Sandstone	230	740
Sandstone, greywacke	290	920
Sandstone, greywacke	300	900
Sandstone, greywacke	320	990

*76 x 102 x 406 mm (3 x 4 x 16 in.) prisms dried at about 21°C (70°F) and 50 percent RH.

Stock sand used in all

**millionths (in./in.)

**Table 3.6 — Cumulative Effect of Adverse Factors on Shrinkage
From Tremper and Spellman^(3.19)**

Effect of departing from use of best materials and workmanship	Equivalent increase in shrinkage, %	Cumulative effect
Temperature of concrete at discharge allowed to reach 27°C (80°F), whereas, with reasonable precautions temperature of 16°C (60°F) could have been maintained.	8	1.00 x 1.08 = 1.08
Used 150 to 180 mm (6 to 7 in.) slump where 75 to 100 mm (3 to 4 in.) slump could have been used.	10	1.08 x 1.10 = 1.19
Excessive haul in transit mixer, too long a waiting period at jobsite, or too many revolutions at mixing speed.	10	1.19 x 1.10 = 1.31
Use of 19 mm (¾ in.) max. size aggregate under conditions where 38 mm (1½ in.) could have been used.	25	1.31 x 1.25 = 1.64
Use of cement having relatively high shrinkage characteristics.	25	1.64 x 1.25 = 2.05
Excessive "dirt" in aggregate due to insufficient washing or contamination during handling.	25	2.05 x 1.25 = 2.56
Use of aggregates of poor inherent quality with respect to shrinkage.	50	2.56 x 1.50 = 3.84
Use of admixture that produces high shrinkage.	30	3.84 x 1.30 = 5.00
Total increase	Summation = 183	Cumulative = 400

Table 3.7 — Reactivity of Various Chemicals with Portland Cement Concrete and Reinforcing Steel/Liner Plates^{*(3.27)}

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

* Source: ACI Committee 515, "A Guide for the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete," Table 2.5.2, ACI 515.1R-79, pp. 515.1R-6 to 515.1R-11, American Concrete Institute Manual of Concrete Practice 1990, Part 5 - Masonry, Precast Concrete, Special Processes, Detroit, 1990.

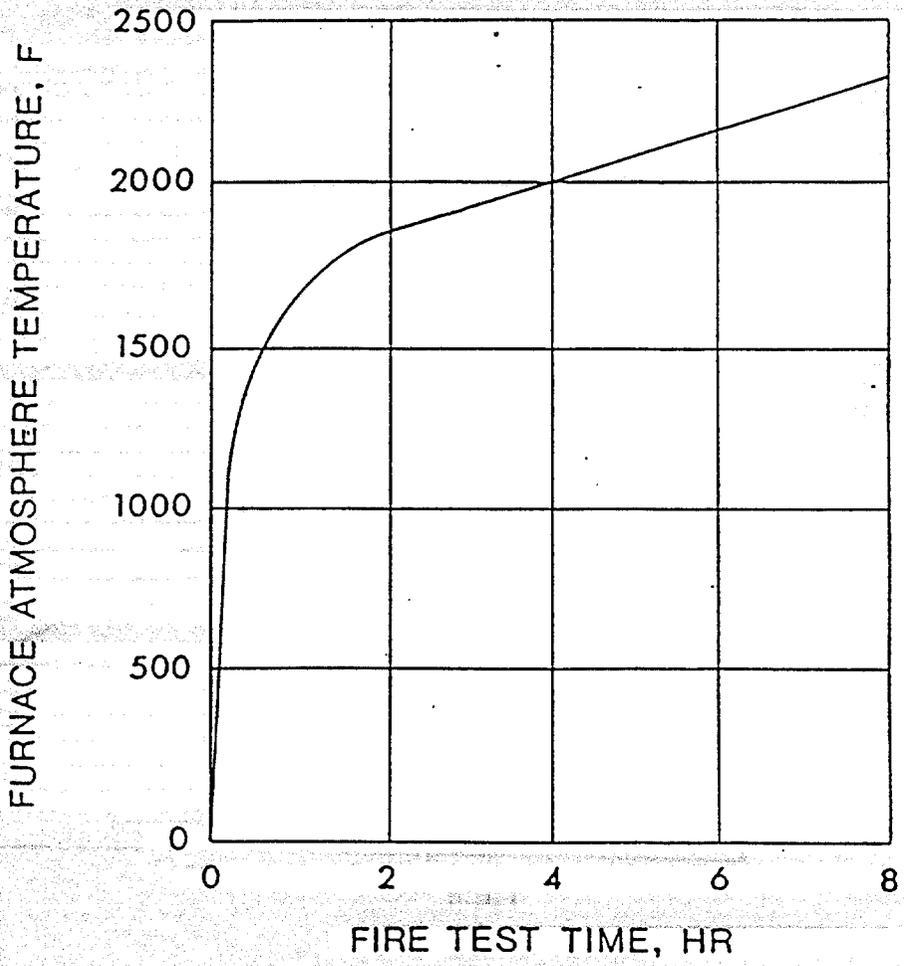


Fig. 3.1 Time-temperature relationship for a ASTM E119 fire

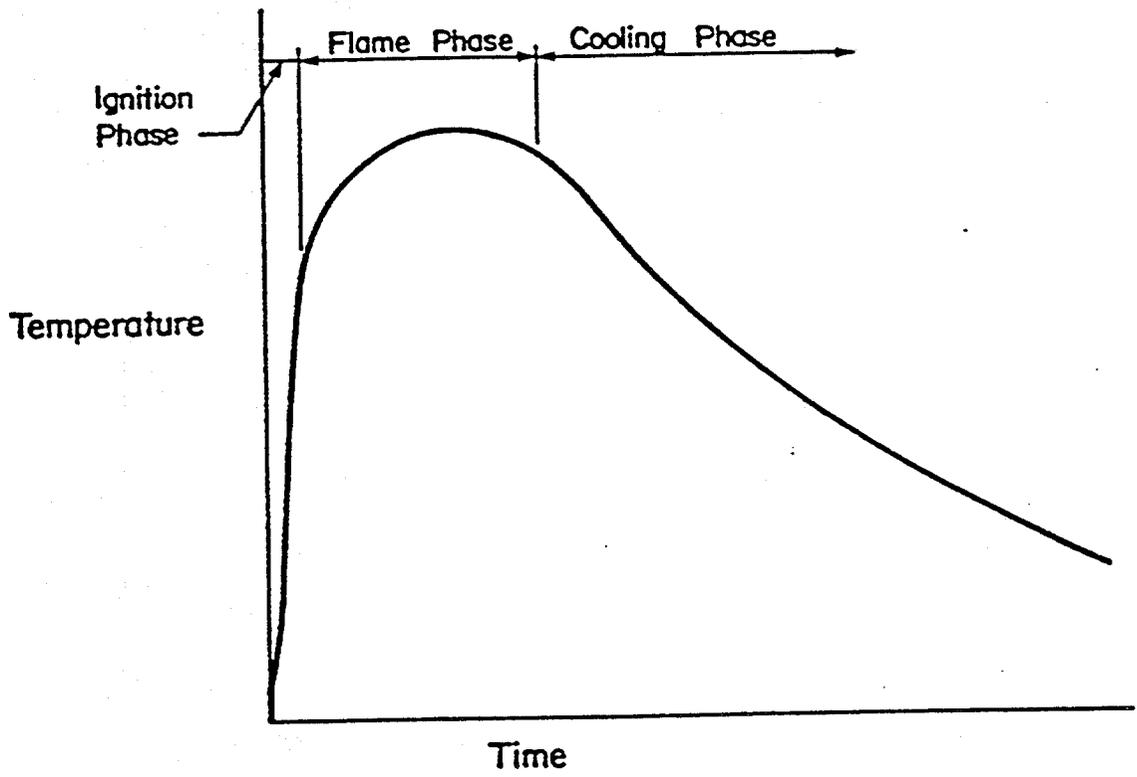
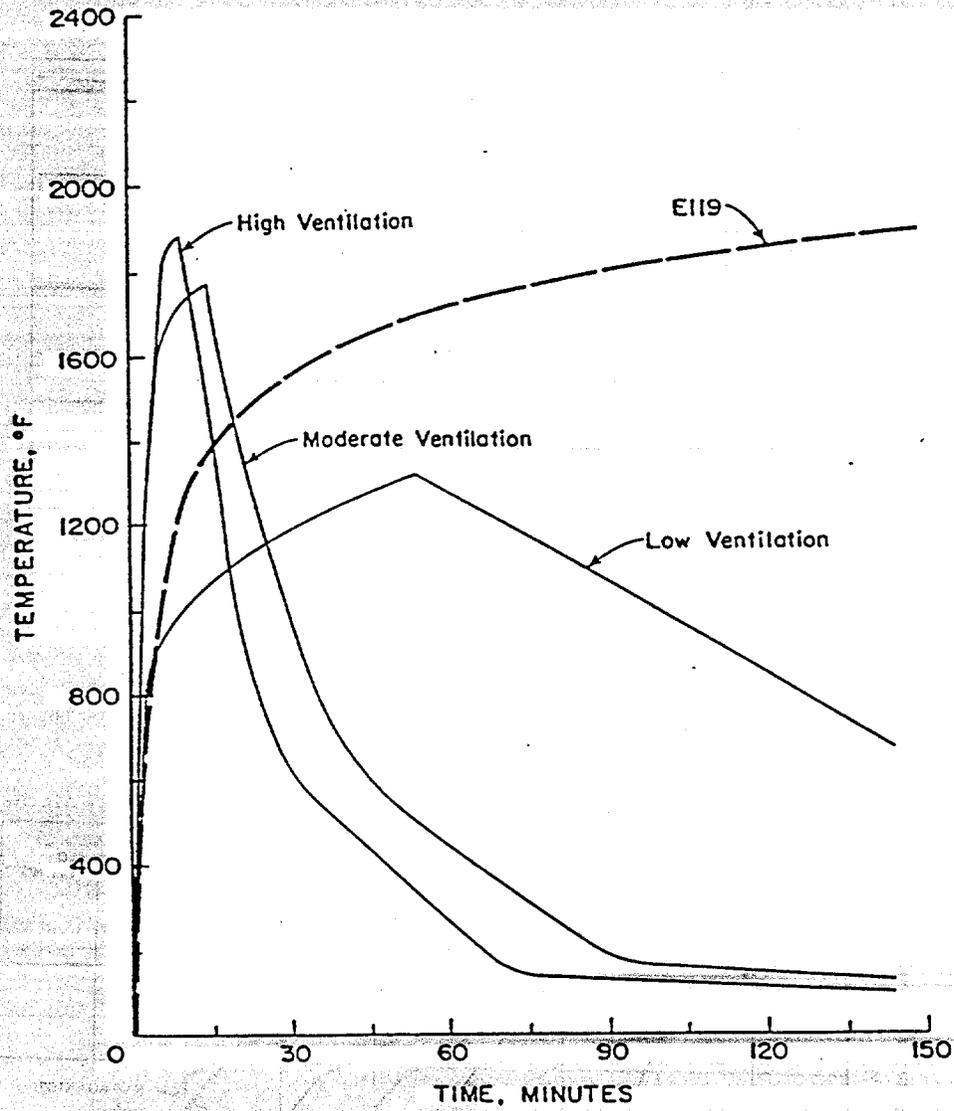


Fig. 3.2 — Phases of a building fire



Time-Temperature Curves

Fig. 3.3 – Comparison of time-temperature relationships for real and ASTM E119 fires

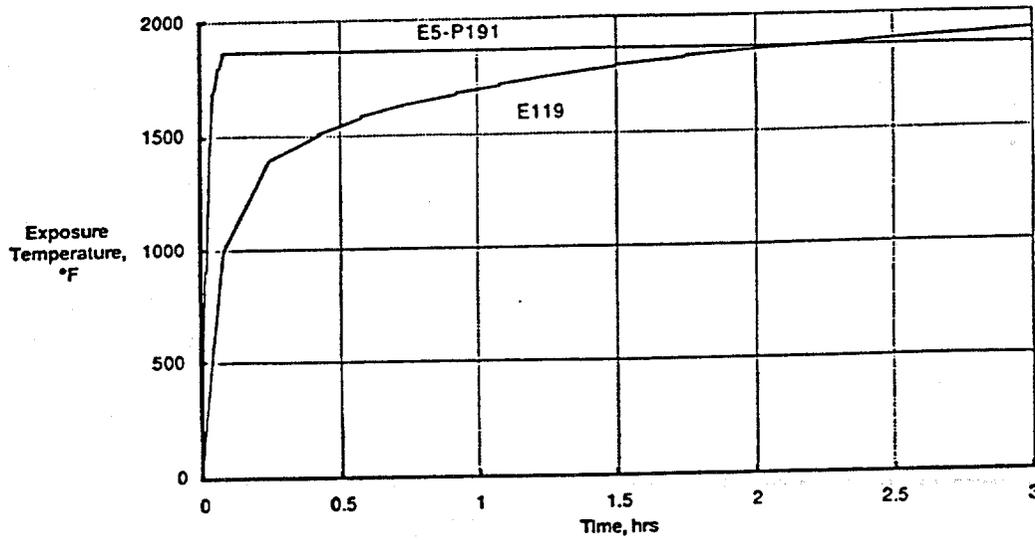


Fig. 3.4 – Furnace time-temperature curves^(3.2)

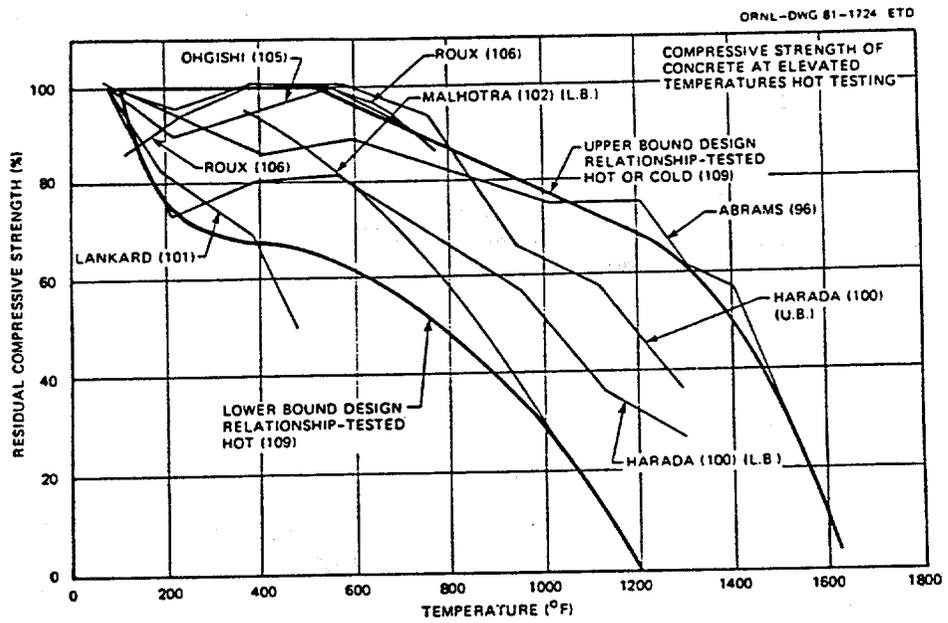


Fig. 3.5 – Effect of temperature exposure on compressive strength (hot) [Naus, Ref. 3.10]^(3.13)

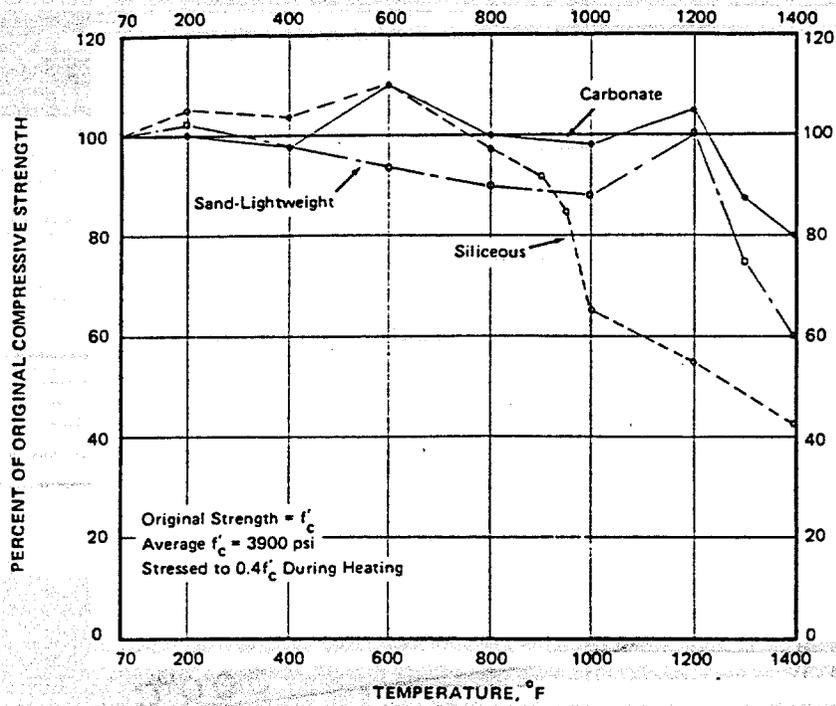


Fig. 3.6 — Compressive strength of concrete made with calcareous, siliceous, and lightweight aggregates at elevated temperatures^(3.11)

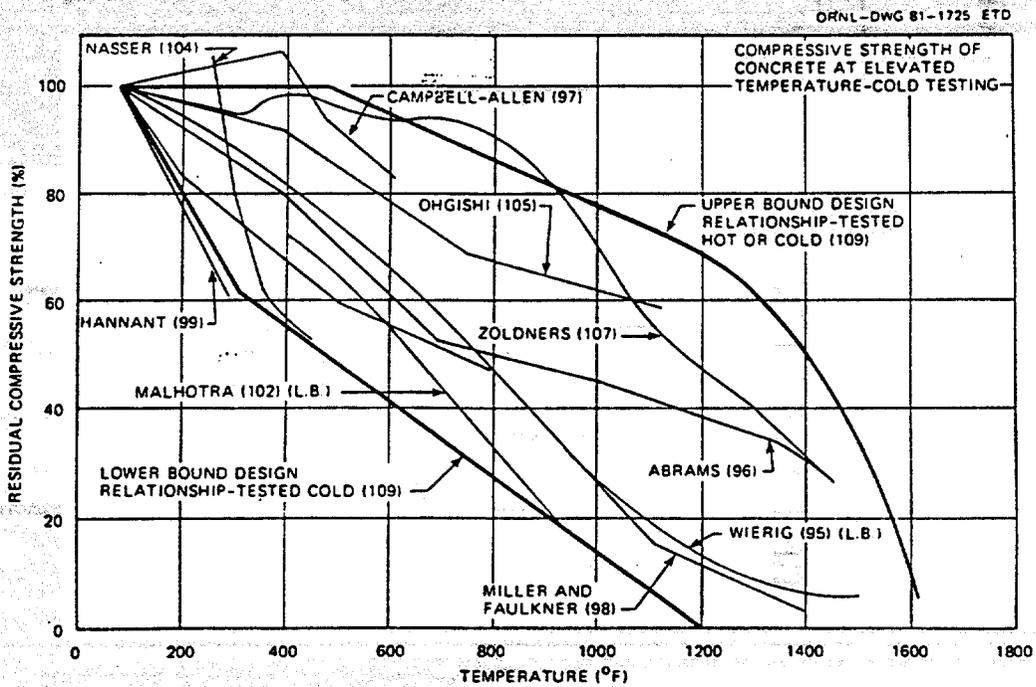


Fig. 3.7 — Effect of temperature exposure on compressive strength (residual) [Naus, Ref. 3.10]^(3.13)

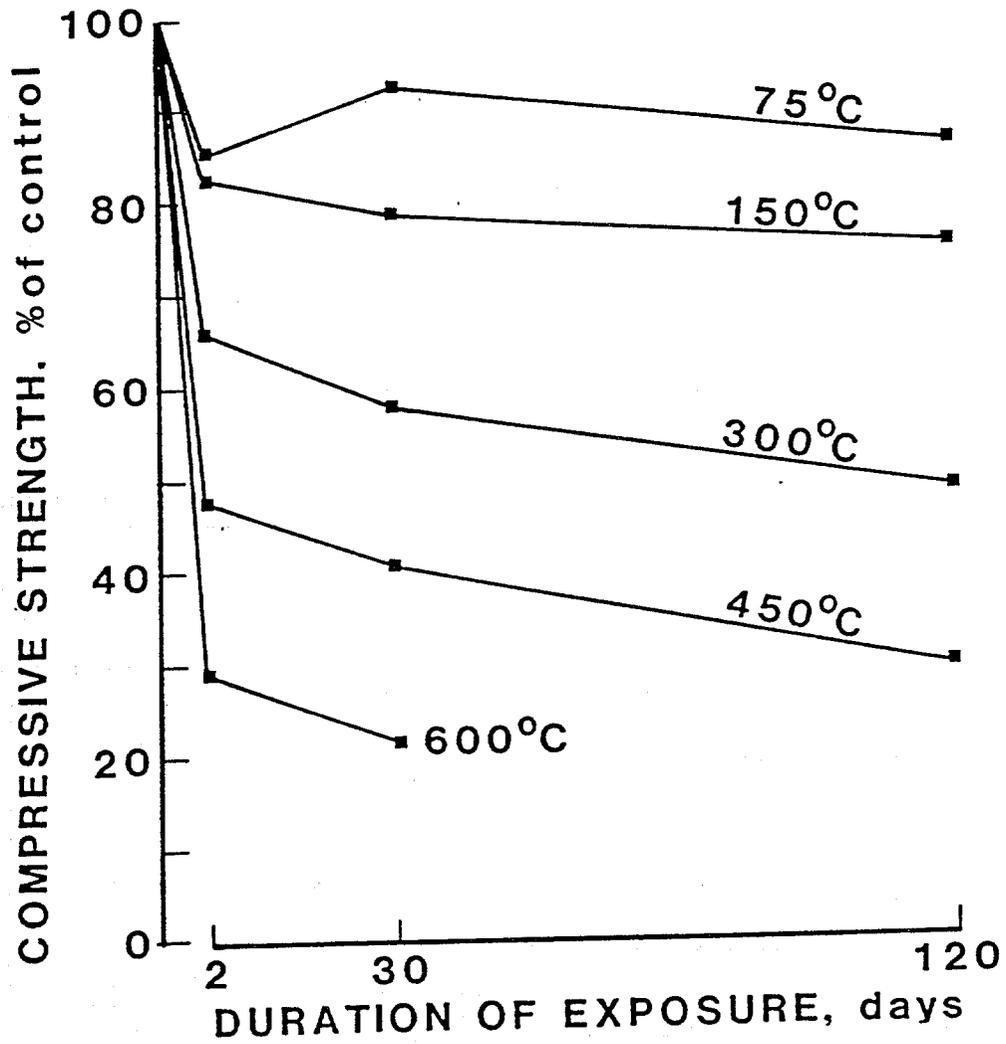


Fig. 3.8 — Effect of duration of exposures on compressive strength of concrete exposed to various temperatures^(3.9)

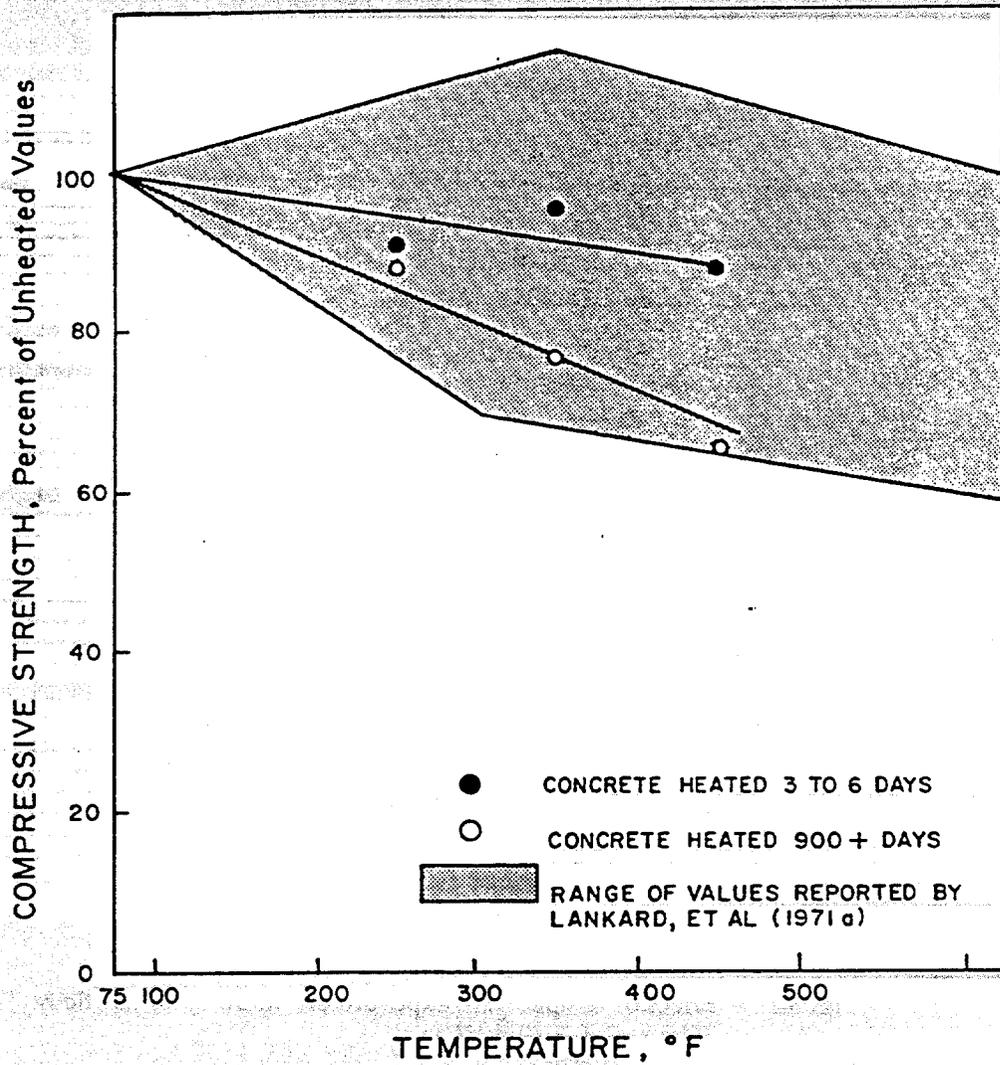


Fig. 3.9 — Effect of duration of exposures on compressive strength of concrete exposed to various temperatures^(3.8)

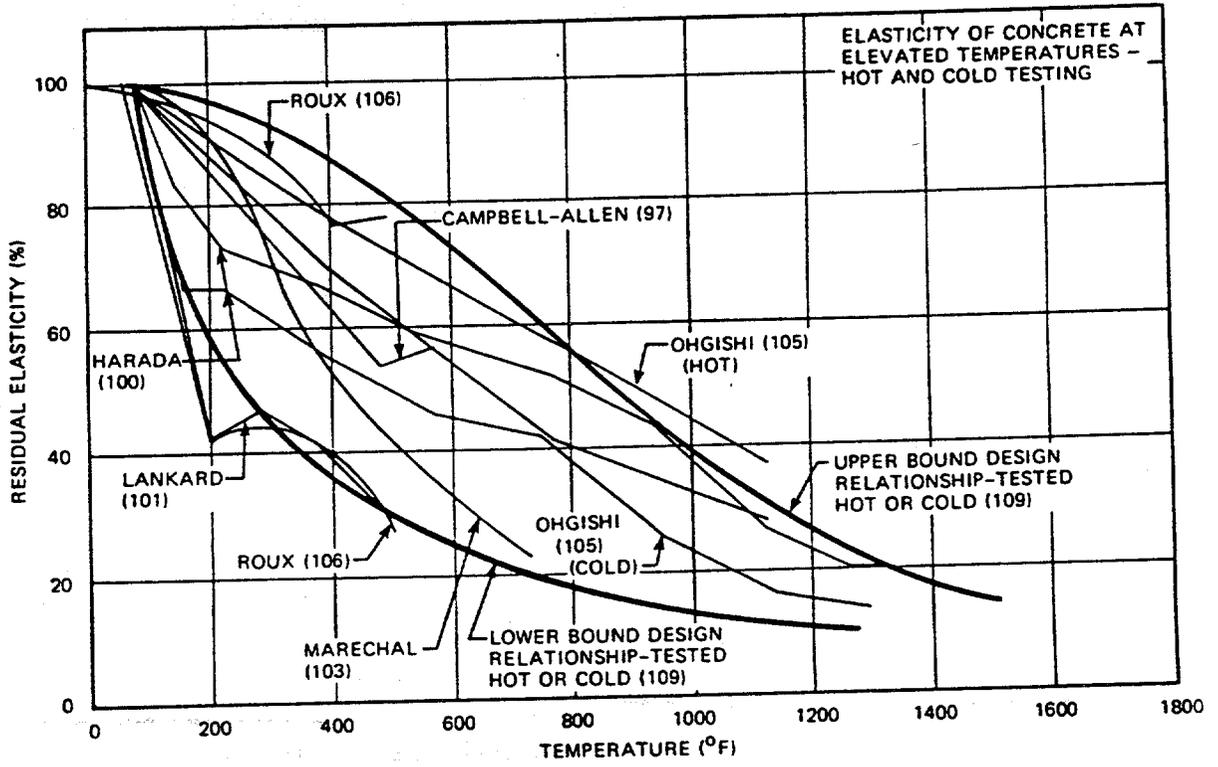


Fig. 3.10 – Effect of temperature exposure on modulus of elasticity [Naus, Ref. 3.10]^(3.13)

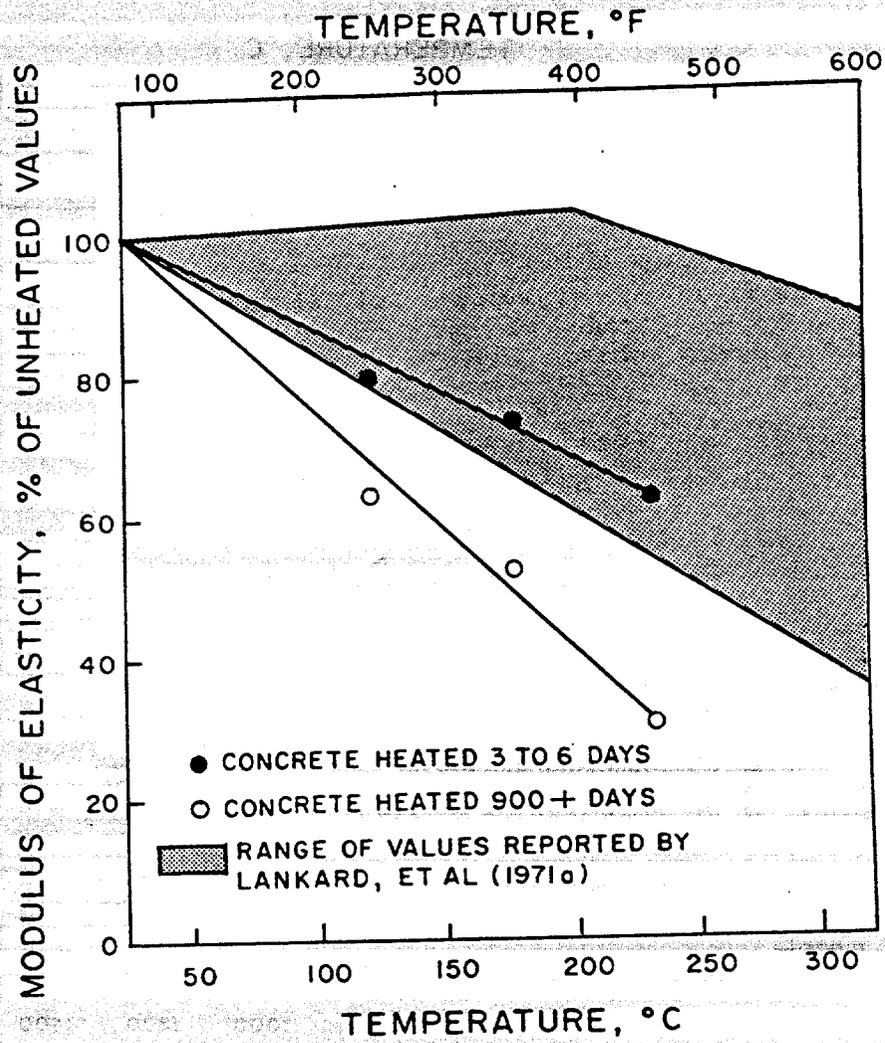


Fig. 3.11 – Effect of temperature exposure on modulus of elasticity^(3.8)

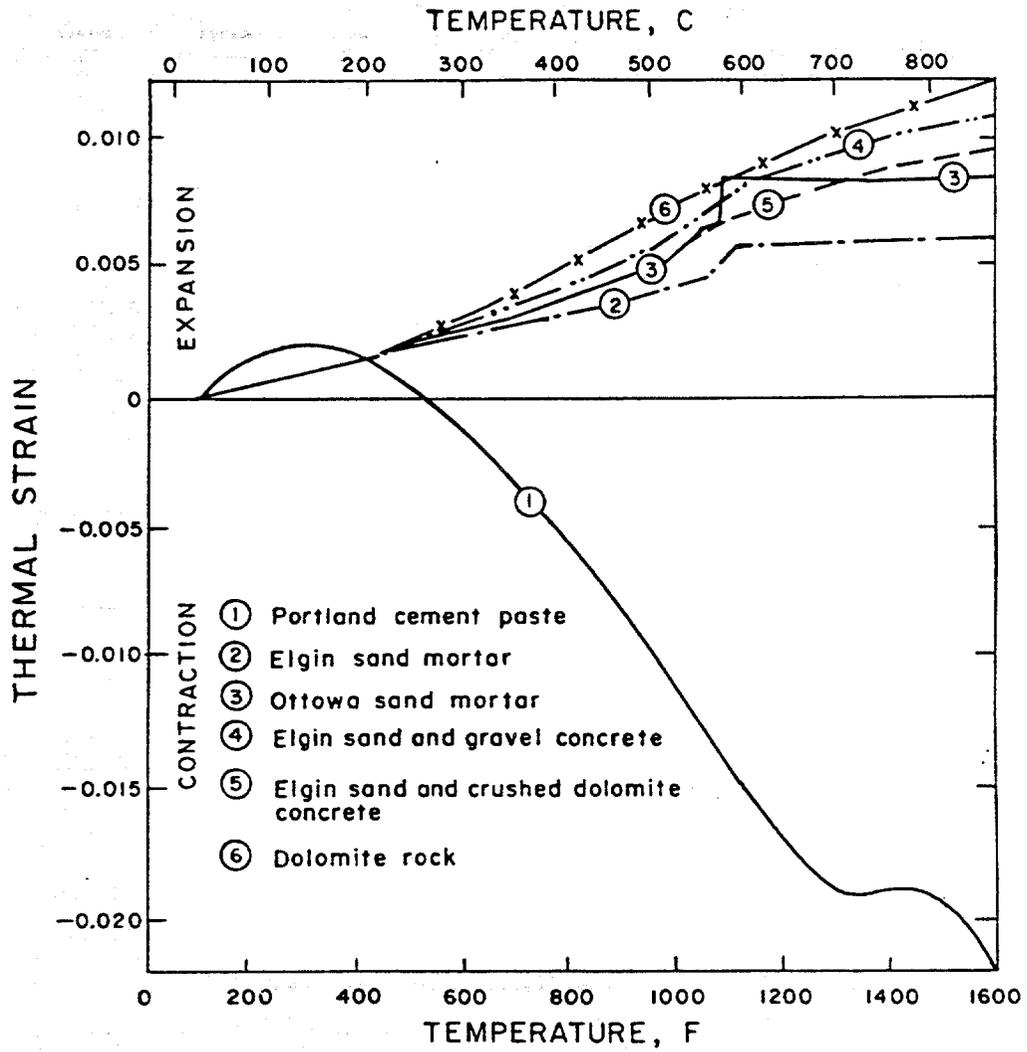


Fig. 3.12 — Thermal strain of paste, aggregate, and various concretes^(3.8)

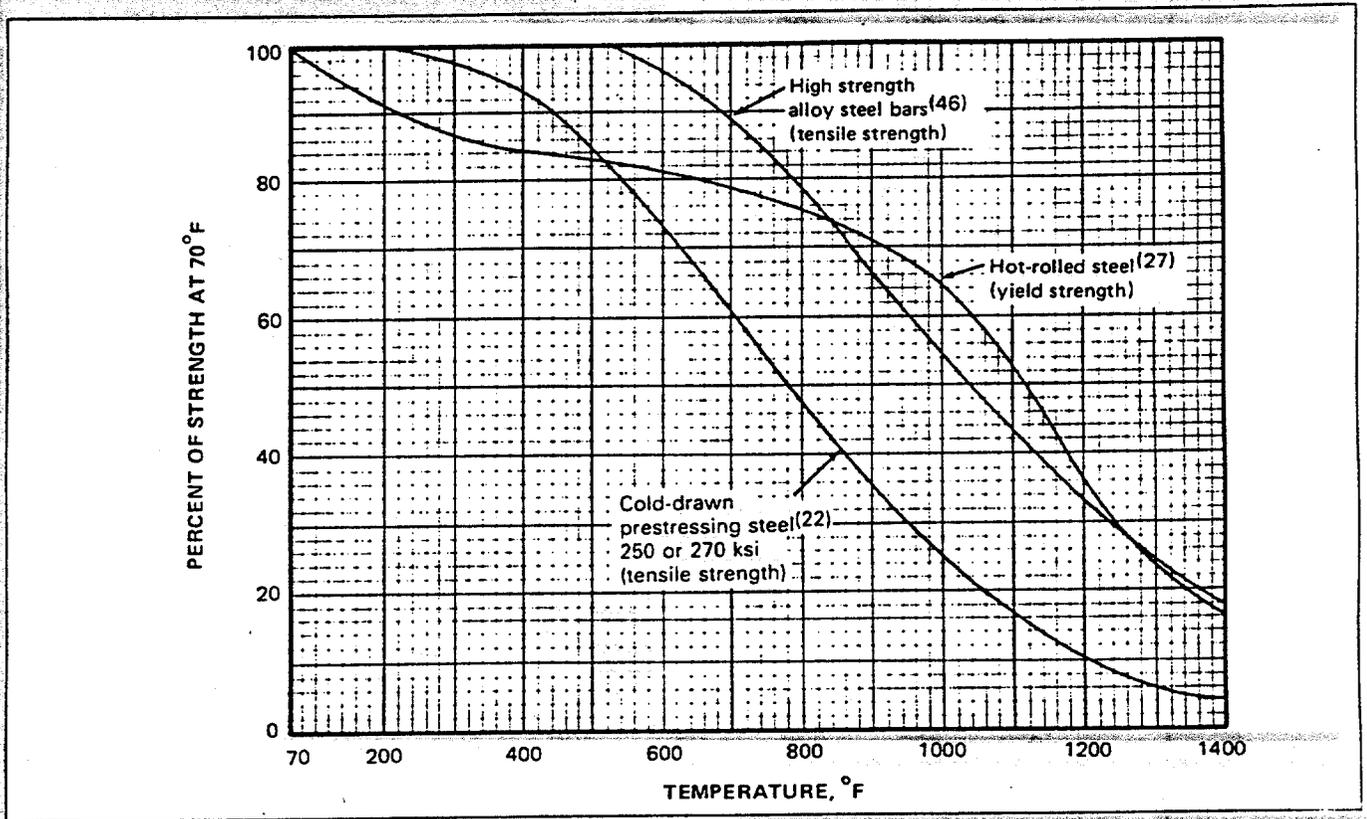
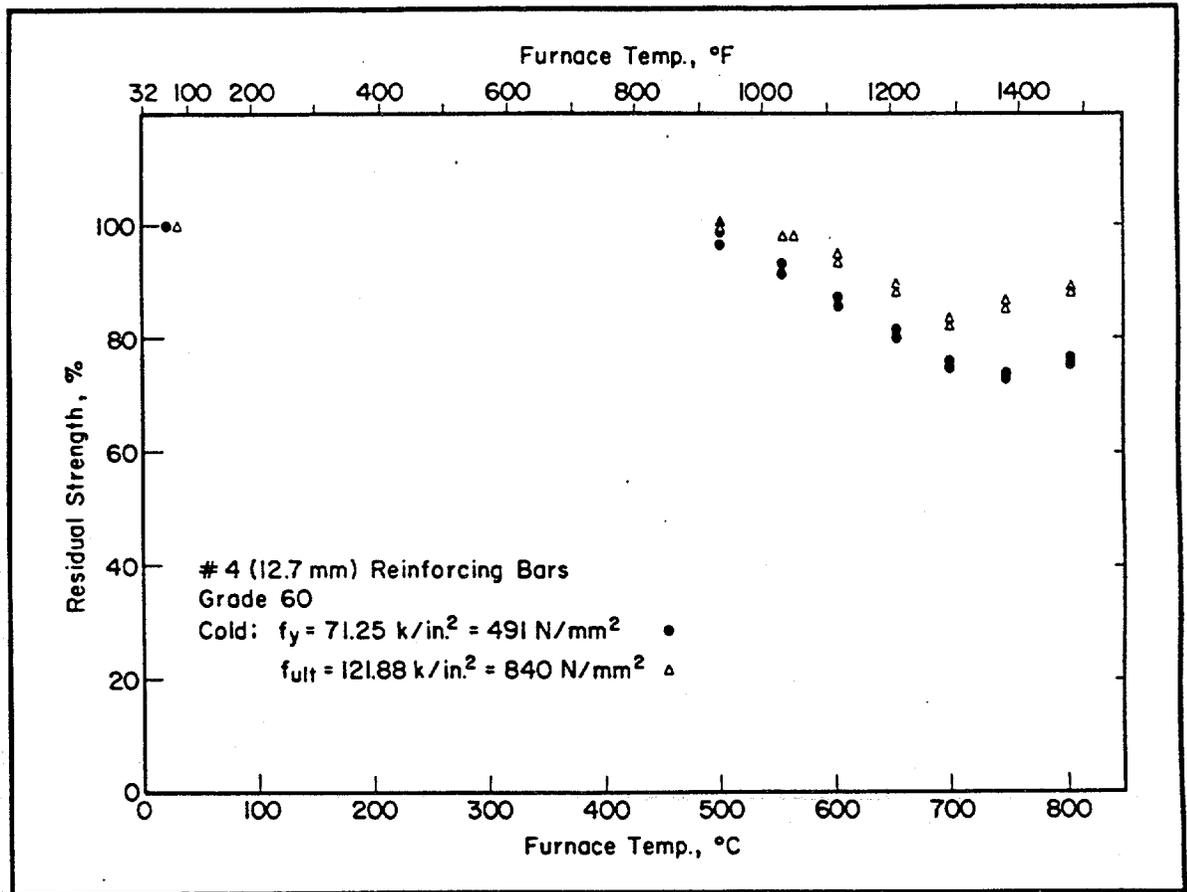


Fig. 3.13 – Temperature-strength relationship for hot-rolled and cold-drawn steels^(3.11)



Temperatures above 932 F (500 C) caused reductions in both yield stress and ultimate stress.

Fig. 3.14 — Residual yield and ultimate strengths of Grade 60 bars after exposure to various temperatures^(3.16)

Pulse Velocity versus Strength Data

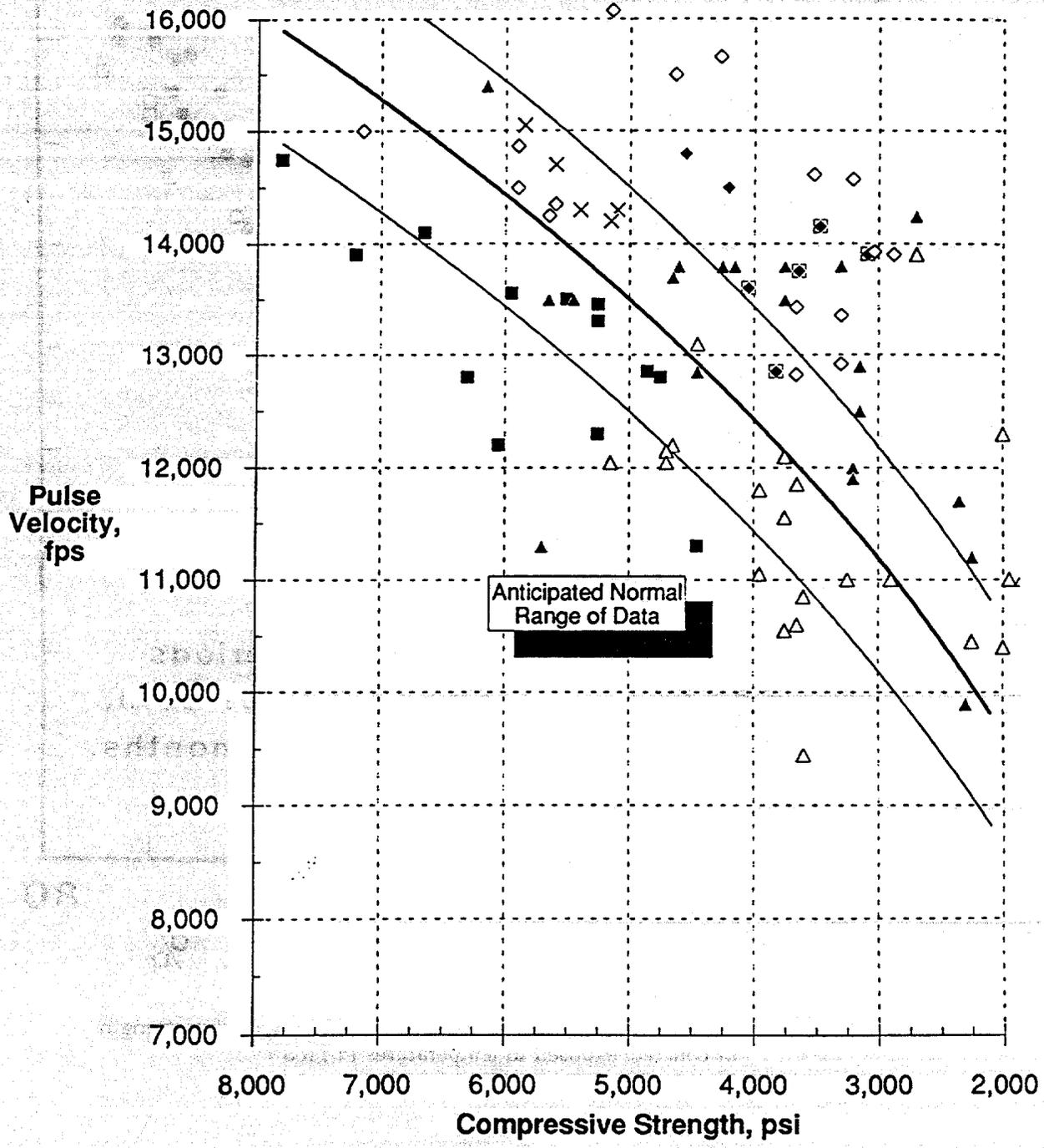


Fig. 3.15 — Relationship between pulse velocity and compressive strength for a variety of concrete mixes

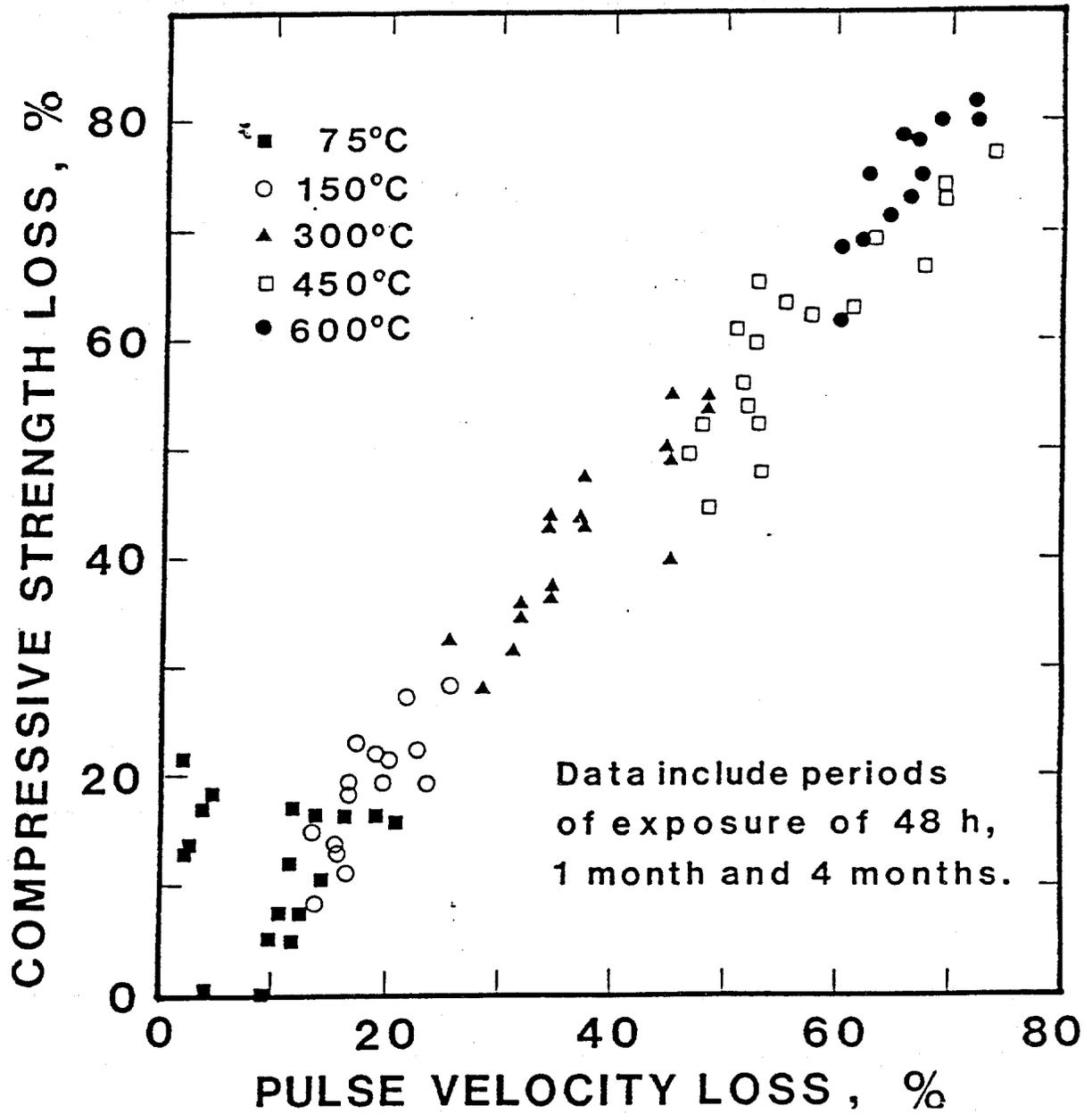


Fig. 3.16 — Relationship between pulse velocity and compressive strength of concrete exposed to temperatures to 1100°F^(3.9)

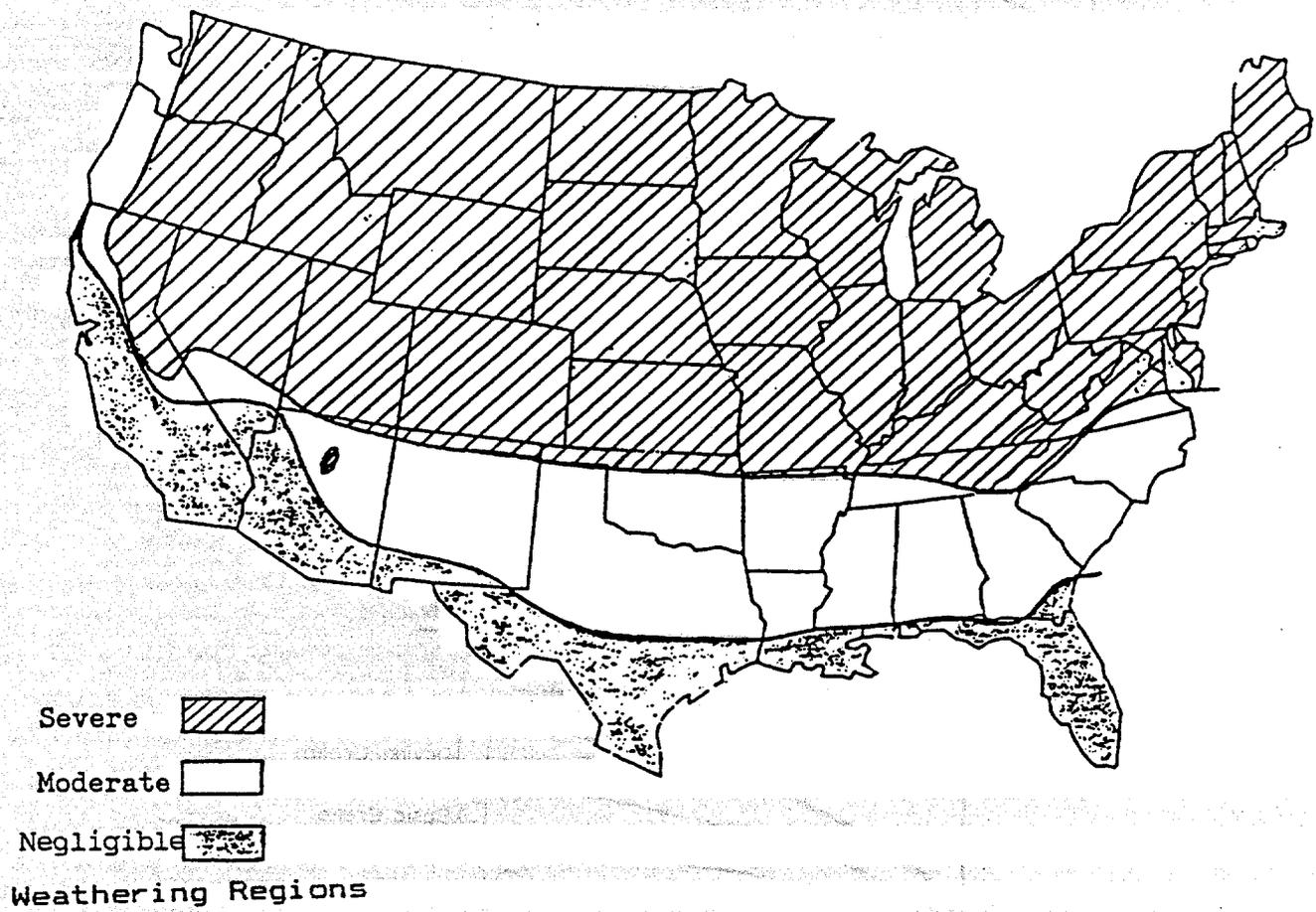


Fig. 3.17 - Location of weathering regions⁽³³⁾

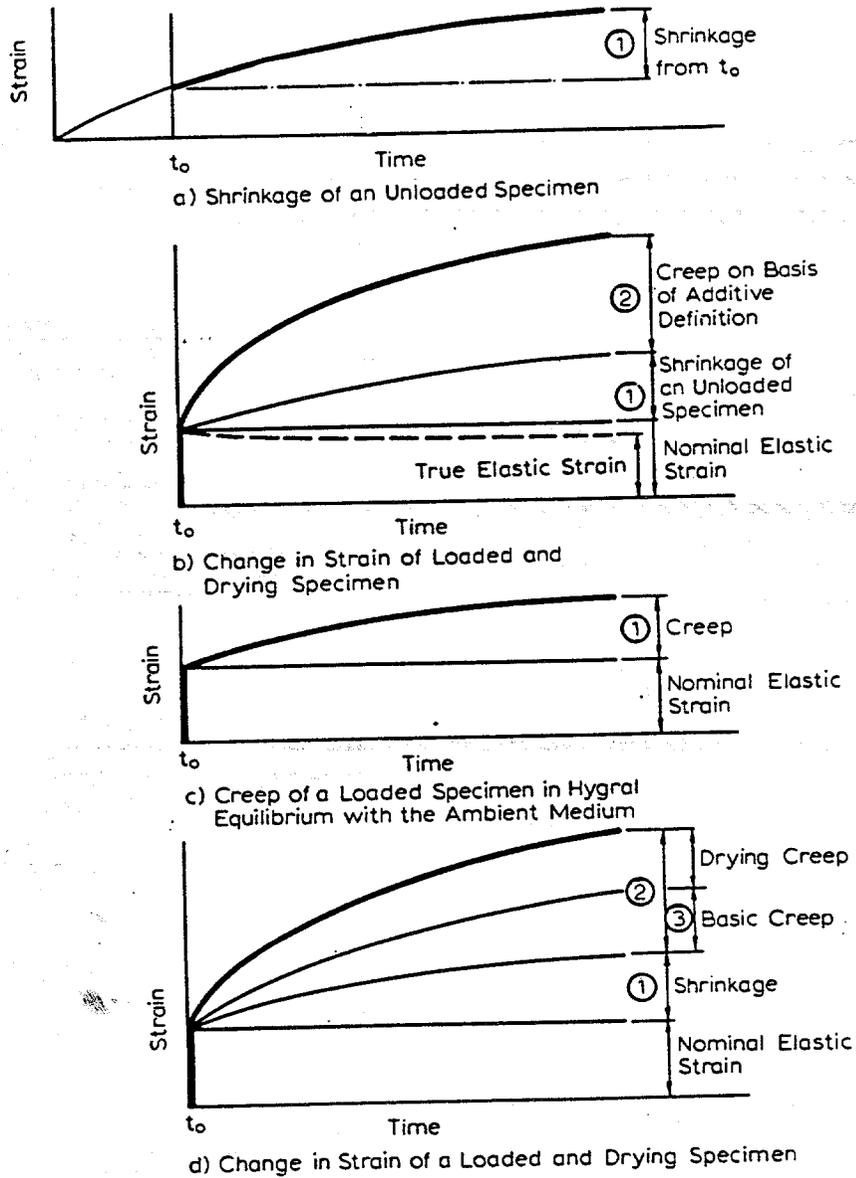


Fig. 3.18 — Time-dependent deformations in concrete subjected to a sustained load^(3.31)

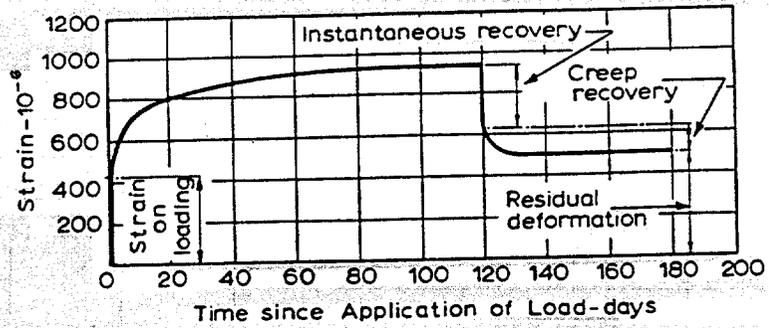


Fig. 3.19 — Creep and recovery of a mortar specimen, stored in air at a RH of 95 percent, subjected to a stress of 14.8 MPa (2150 psi) and then unloaded^(3.32)

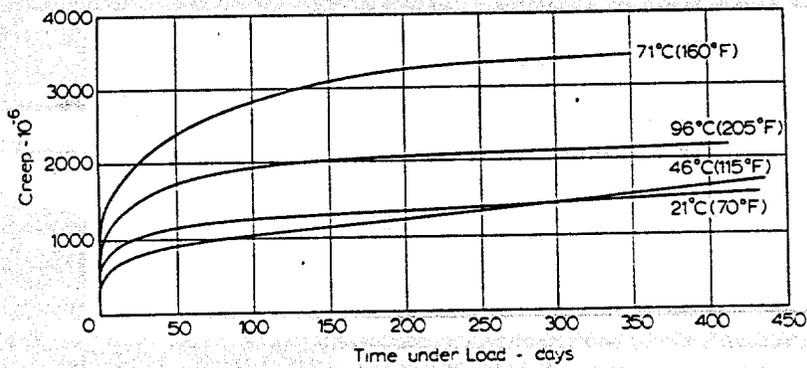


Fig. 3.20 - Relationship between creep and time under load for concretes stored at different temperatures (stress/strength ratio of 0.70)^(3.33)

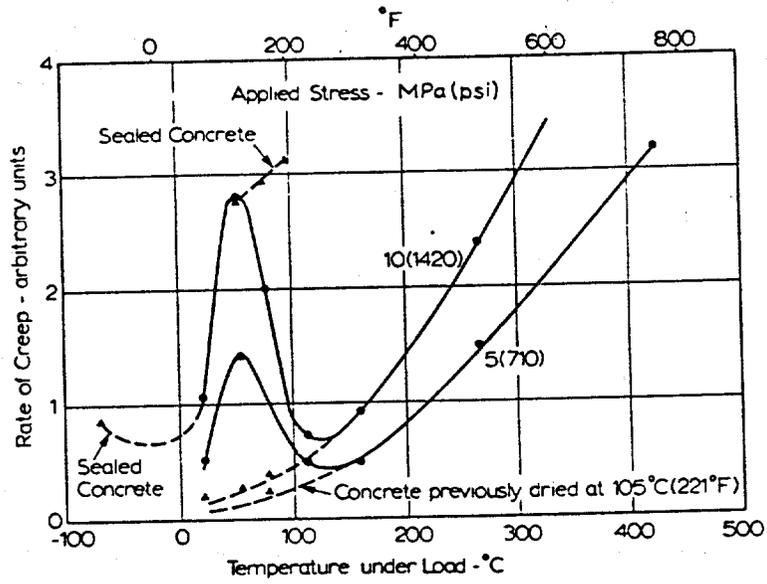


Fig. 3.21 — Influence of temperature on rate of creep^(3.34)

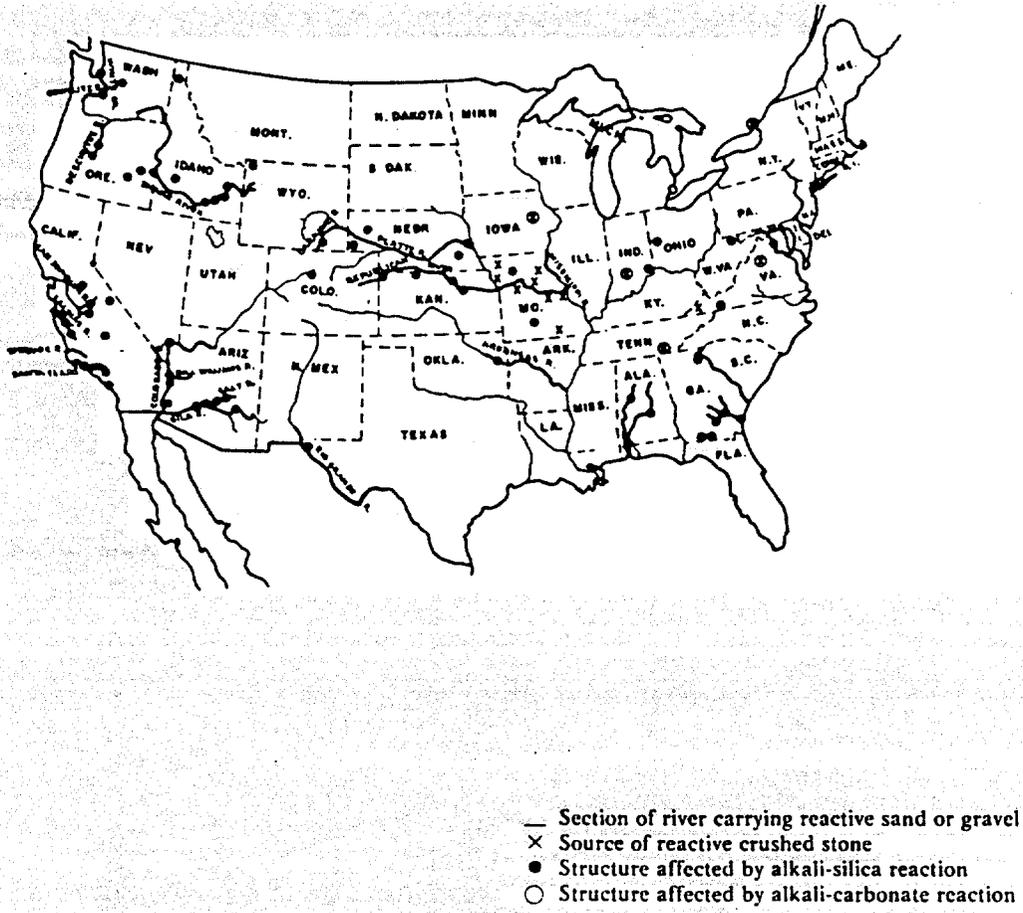


Fig. 3.22 — States known to have aggregate sources containing alkali reactive aggregate ^(3,38)

4. REPAIR TECHNIQUES

4.1 Initial Repair Considerations

Effective maintenance must be based on a thorough knowledge of the structure and its needs, in addition to repair materials and techniques. Knowledge of the structure can be gained from the project specifications, as-built drawings, and construction materials test reports. However, to learn how serviceable the structure has been, and how well it has withstood its environmental conditions, a condition survey is required. The methodology of a condition survey was discussed in detail in Chapter 2 of this report.

The cause of the deterioration must be determined before a repair technique is selected. The extent of the damage must be identified, and the condition of the portions of the structure to remain must be determined. The extent of deterioration is often underestimated. Therefore, estimates should be conservative; shoring and additional support for load-carrying members should be carefully considered prior to demolition or removal of deteriorated concrete. If the entire structure is of questionable integrity, complete reconstruction may be needed. Based on careful examination and analysis, the repair methodology is selected. This must be done on an individual case basis and requires careful judgement by an experienced engineer.

A repair material must not be entirely accepted as suitable for all the uses recommended by the manufacturer. Dozens of repair formulations have been encountered that were totally unsuitable for the recommended repair, for a variety of reasons. Also a material should not be specified unless its ingredients are known. Several fast-setting materials currently on the market are unstable under moist conditions. This is because they depend on the setting of plaster for their early strength and on portland cement for later strength development. Plaster consists of calcium sulfate, which can result in severe internal sulfate attack later on, if sufficient moisture is available. This causes expansion, cracking and loss of strength. The patches usually grow until they either fall out of the excavation or crack the surrounding concrete.

The most appropriate materials for patching are those that are closest in composition to the material to be patched. Usually, this means portland cement concrete for large patches or portland cement mortar for small ones. By patching with a cementitious material, the final thermal and structural properties of the repair will be similar to the base concrete. Many proprietary formulations are based on portland cement, and many of these include a latex emulsion. These latex emulsions may be acceptable and offer some advantages, if the proper formulation is used. Non-portland cement binders have also been used successfully. One example is magnesium phosphate cement, which shrinks much less than portland cement during hardening. This is a decided advantage in patching materials. However, as with all materials, magnesium phosphate concrete has limitations, as discussed later in this report.

Repair materials should have low shrinkage. Other properties, such as thermal coefficient of expansion, strength, modulus of elasticity, and creep should be close to those of the substrate concrete. The thermal coefficient of expansion is a measure of the expansion and contraction of a material subjected to temperature changes. The modulus of elasticity is a measure of the stiffness of a material. A low modulus material will deflect and deform more under load than a high modulus material. Creep is a measure of long-term deformation under substantial load. A repair material with a slightly lower thermal coefficient, lower modulus of elasticity and higher creep, is optimum so that stresses are minimized.

Polymers used as binders, in patching mortars can be effective when used in small quantities. However, when used to fill large cavities, the differences in thermal coefficients between these resin-based materials and the substrate concrete become important. Even mixed with large amounts of

aggregate, most polymer concretes expand and contract several times the amount experienced by portland cement concrete under similar thermal exposure conditions. In large quantities, this can cause enough stress at the interface between patch and concrete to cause bond failure. Polymers should be considered for overlays and coatings, especially in environments aggressive to portland cement materials.

Special attention should be given to repairs that are exposed to fluids. The chemical composition of the fluids should be well known and the repair materials must be compatible. Exposure to acids and sulfates will attack and degrade portland cement materials but not most polymers. Strong solvents may attack or soften some polymer materials. Dilute acids can sometimes be more aggressive than concentrated acids to portland cement concrete. Hard water that contains large amounts of calcium salts are less corrosive than soft water. Ultra-pure water can aggressively leach portland cement concrete. Low strength solutions of sodium chloride (1-3 percent) may be more corrosive to exposed steel than high strength solutions (10-20 percent).

The location of repairs must also be considered. Limited access may restrict what type of equipment can be used for installation. Steel congestion may determine the consistency of the repair material required. Good ventilation is required when using polymer materials. If ventilation cannot be provided, cementitious materials or polymer-modified cementitious materials should be used. Special considerations must be given to repairs within containment areas having limited short-term access. Materials that can be placed rapidly with minimal equipment and produce the least waste are best. High service temperatures may adversely affect the performance of some polymers. Some resins may embrittle after long-term exposure to elevated temperatures while other resins may flow. Elevated temperatures also increase the solvent attack on polymer resins.

The selection of a repair material and technique, as a function of the importance of the structure and the expected service life, is important. The required service life of the repair may dictate its selection disregarding cost. Typically, the volume of material is small and the cost of the installation and labor is usually much larger than material costs, so the cost of the repair material usually does not dictate its selection within reasonable limits.

Conservative estimates of the durability of repairs should be made since installation procedures are responsible for most premature failures of repairs. The design of the repair and the material selection can be difficult, but the problems that result in loss of durability are most often due to poor installation. Proper surface preparation, batching, mixing, placement and curing are all important for long-term durability. Engineers developing repair strategies should always determine if there is in-house experience with the use of a particular material in similar applications. If so, they should consult with the persons familiar with previous repairs and review its performance. This evaluation of old repairs may include nondestructive testing or removal of core samples. Specifying materials that have previous success within the plant is good practice since workers are familiar with the material and problems during installation are less likely.

Review of manufacturer's data, contact with other people who have used the materials, and discussions with consultants specializing in repair are all valuable in establishing the anticipated durability of the repair. Within reason, the best repair technique and material should be selected regardless of cost, and conservative estimates of performance should always be made. On rare occasions, it may be advisable to use a cheaper material and expect frequent replacement. This must be a decision based on each individual case. Consideration of the impact of excess material disposal should also be considered. If the product presents a future disposal problem, frequent replacement would not be acceptable.

If the durability of the repair is of major importance, or the volume of the repair is large, testing of the repair material is recommended. Experience shows that variation occurs during the

manufacturing of most materials, including prepackaged repair materials and polymer materials. Product formulations sometimes change without corresponding changes to the data sheets or notification by the suppliers. A material that performed well in the past may not perform well now, due to changes in formulation or raw material supplies. Changes in ownership sometimes result in changes of product formulations. Material testing to ensure that materials meet the project and manufacturer's specifications, and accelerated testing under the exposure conditions expected in-service are good practices. If possible, tests should be performed on material from the manufacturing lot to be used for the repairs. Tests for cementitious materials might include set time at expected temperature extremes, flow, density, strength gain, bond, freezing and thawing resistance, resistance to aggressive media, and abrasion resistance. Additional tests for polymer materials might include viscosity, specific gravity, solids content, flash point, vapor pressure, bond to wet and dry concrete, and properties at elevated temperatures.

Repair documents — The ACI Committee Reports 201.2^(4.1) and 546^(4.2) discuss the repair of concrete and are excellent references. ACI has also produced a number of documents used in seminar series. Two documents of particular interest are entitled *Troubleshooting Concrete Problems—And How to Prevent Them in the Future*^(4.3) and *Concrete Repair Basics*.^(4.4) Persons involved in repair of concrete structures should review these documents to become familiar with typical repair procedures and techniques.

Both the U. S. Corps of Engineers and the U. S. Bureau of Reclamation have recently produced concrete repair manuals.^(4.5, 4.6) The manual by the Corps of Engineers entitled *Evaluation and Repair of Concrete Structures*^(4.5) provides a standard format for repair techniques. The document includes chapters on evaluation of concrete in concrete structures, causes of distress and deterioration of concrete, selection of materials and methods for repair and rehabilitation, concrete removal and preparation for repair, materials and methods for repair and rehabilitation, maintenance of concrete, specialized repairs and case histories. This document is a good reference for any engineer involved with repairs. The document does not, however, provide extensive information as to the types of materials available for repairs. The discussion on material applications and limitations tends to be brief. No information is provided on the service life of a repair once such techniques have been used, nor does it provide a ranking system for the durability of repairs of different types.

The U. S. Corps of Engineers REMR Notebook^(4.7) includes material data sheets on specific products and test data. The manufacturer's data are included and if testing has been performed by the Corps of Engineers, the Corps' test data are included. A computer database of the materials notebook has been established and is accessible by computer modem. The database is free to use and has over 1700 products listed by application or type. Specific instruction on the REMR database can be received from the U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi by calling (601) 634-2814 or (601) 634-2752. The database may be accessed directly by modem by dialing (601) 634-4223. An annotated bibliography of REMR Technical Reports^(4.8) is also available. It contains many useful references for specific repair or maintenance problems.

The following sections briefly discuss typical repair techniques and specific concerns related to the material selection and installation. The purpose of this discussion is to provide information, gained by experience, to improve the durability of repairs and to address many common misconceptions.

4.2 Resurfacing With Toppings and Overlays

If damage was a result of poor quality concrete, the deteriorated concrete should be removed and replaced with higher quality concrete. If portions of the concrete to remain in place is of inferior quality, it should be protected from the environment to prevent continued deterioration. If the concrete is good quality and deteriorated from severe exposure, additional means may be necessary

to ensure that the repair and the surrounding concrete is durable. The repair may include using a very high quality repair material and installing a secondary or tertiary means to protect the repair and the remaining concrete.

The purpose of a topping or overlay is to establish a new wearing surface and restore or raise the existing grade. The reason for the deterioration should be identified so that the topping or overlay can be formulated to resist the aggressive agent. Concrete that has only light surface deterioration can be protected by applying a unfilled or partly filled coating, typically under 0.8 mm (0.030 in.) thickness. Toppings up to 10 mm ($\frac{3}{8}$ -in.) can be applied using polymer grouts typical of those formulated for repair of industrial floors. Overlays over 10 mm ($\frac{3}{8}$ -in.) thick can be a variety of materials, from polymer concrete to portland cement and polymer-modified cement concretes.

If a concrete must withstand chemical attack, such as acids, changing the mixture proportions is relatively ineffective in providing protection. A sacrificial means is to incorporate carbonate sand and gravel; however, this approach admits defeat, albeit at a later date. The most effective method of protection is by the use of membranes, either liquid-applied or preformed. Epoxies and methacrylates have been found effective among the liquid-applied materials, while butyl and neoprene rubbers have performed well among the preformed membranes. These are discussed in more detail in the section of this chapter on membranes.

4.2.1 Surface Preparation for Toppings and Overlays

Factors related to the installation of toppings and overlays are most important for their long-term durability. Proper attention to surface preparation, condition of the substrate, mixing and placing the concrete, and curing are essential for a durable topping. Complete removal of the unsound or scaled concrete is very important. Durability of the repair depends on the materials adhesion to the base concrete. Delaminated concrete should be removed. Abrasive blasting, using steel shotblast equipment or hydroblasting, using very high pressure water, are the best methods of surface preparation. Deeply scaled and deteriorated concrete should be removed by using small chipping hammers followed by abrasive blasting.

Surface preparation by acid etching has been used, but experience shows that this method is not as reliable as mechanical abrading methods. The acid works by dissolving portions of the cement paste and exposing the aggregate. However, the remaining paste is thus weakened. The acid must be thoroughly washed off the surface to prevent a further weakened bond interface. Abrasive blasting is much preferred over acid etching and should be considered for final preparation even after acid etching.

The best adhesion for cementitious toppings is achieved on a saturated surface dry (SSD) surface that is continuing to dry. The slightly dry concrete substrate pulls minor amounts of moisture from the repair material and provides the best bond. New concrete does not bond well to a totally wet substrate concrete. Even visible free moisture or sheen on the surface will decrease the bond strength. Free water on the surface increases the water content of the paste at the bond interface, reduces its strength and increases its shrinkage. Testing clearly shows that new concrete bonds better to dry concrete than to even SSD concrete. However, especially for thin overlays or repairs, very dry substrate concrete may pull moisture from the repair material and prevent complete hydration. Therefore, it is recommended that the concrete be thoroughly wetted once 24 hours prior to placing the cementitious overlay or patch and then allowed to dry so no visible moisture is present on the surface.

The surface preparation for application of polymer grouts is similar, except that the substrate must be kept as dry as possible prior to patching. Moisture significantly reduces the bond of most polymers. Additional discussion of materials and the surface preparation for specific materials are included in the material section of this chapter.

Failures of toppings and overlays generally occur due to loss of bond or cracking. This can be due to poor surface preparation, excessive shrinkage of the repair material, or thermal incompatibility. Some fine cracks must be expected as the overlay dries and shrinks; however, no large cracks or delaminations should develop. Delaminations due to loss of bond can easily be located by sounding with a chain drag or a light hammer. The bond strength of the topping to the base concrete can be measured by direct tensile pulloff tests. A minimum of 0.7 MPa (100 psi) and preferably 1.4 MPa (200 psi) should be achieved, depending on the type of topping and strength of the base concrete. Direct tensile pulloff tests can also be helpful to determine the adequacy of the surface preparation. The test bobbins are affixed directly to the prepared substrate and tested in tension. Virginia DOT requires a minimum of 1.4 MPa (200 psi) prior to overlaying bridge decks with a polymer concrete. Direct tensile and shear bond tests are described in ACI Committee Report 546^(4,2)

4.2.2 Toppings

Thin toppings, less than $\frac{3}{8}$ -in. thick, are typically done with polymer resin grouts. Binders such as epoxy, acrylic, polyester, or urethanes are used. Polymer toppings provide a durable, chemical resistant surface that also protects embedded steel from carbonation or chloride ingress. Surface preparation includes removal of deteriorated concrete, roughening of the surface, removal of contaminants, repair of porous surfaces, and controlling moisture in the base concrete. The base should be roughed but should not contain large holes or spalls. Deep holes or large spalls should be repaired prior to placing the thin topping. This will ensure that these areas are filled and consolidated. Repair of deep holes and spalls is usually not necessary for thicker overlays.

The base slab must be dry for most polymer toppings. However, if it cannot be dried, some polymer toppings are available that tolerate moisture. Typical recommended substrate temperatures for placement of polymer toppings is between 10°C (50°F) and 30°C (90°F). Low temperatures cause resins to thicken and prevents them from wetting and penetrating into the surface. High temperatures result in rapid setting and higher thermal shrinkage stresses. Special resin formulations, usually containing MMA or urethane, are available for use in cold temperature extremes, even below freezing.

The selection of topping material must be based on the exposure conditions and expected abrasion and loads. The resins commonly used for thin toppings are discussed in detail in the organic resin materials section of this chapter. They are typically applied in several thin coats to fill holes, level the surface and achieve the proper thickness. Primer resins are often recommended to improve bond. Thin toppings can be applied by spray or roller. Thicker toppings, of 3 mm ($\frac{1}{8}$ in.) to 9 mm ($\frac{3}{8}$ in.), are filled with fine sand and can be applied by trowel or screed.

4.2.3 Overlays

Overlays are used where a significant increase in thickness is needed. Overlays can provide a high strength surface and add protection to the embedded steel from corrosive environments. Overlays are often designed to be an integral part of the load-carrying capacity of the structure and can be used to improve drainage and grades. Smooth hard-troweled surfaces, or surfaces with high skid resistance, can be provided. For thicknesses between 6 mm ($\frac{1}{4}$ in.) to 25 mm (1 in.), polymer concretes or grouts are typically used. For overlays between 25 mm (1 in.) to 38 mm ($1\frac{1}{2}$ -in.) thick latex-modified concrete (LMC) are typically used. Thicker overlays are commonly done with normal portland cement concrete. Modified high-alumina cement concrete or silica fume-modified concretes have been used on a limited basis. Modified high-alumina cements are used when rapid setting is important, and silica fume adds strength. LMC and silica fume concretes provide a durable surface and low chloride permeability but are not rapid setting. Overlays typically contain 10 to 13 mm ($\frac{3}{8}$ to $\frac{1}{2}$ -in.) maximum size aggregate and are applied using a vibratory screed. The largest size aggregate possible should be used to reduce shrinkage. The

characteristics and properties of each of the overlay concretes are discussed in the material sections of this chapter.

4.2.4 Bonding Agents for New to Old Concrete

Primers are not usually needed for repairs using portland cement concrete or grout. The paste fraction of the concrete or the grout makes an excellent bonding agent. The repair material paste should be aggressively scrubbed into the surface immediately prior to placing the bulk of the material.

The use of separate grout or mortar bond coats are not always beneficial to bond. Separate cement sand slurries or cement latex slurries can be used as bonding agents, but they must be carefully proportioned, mixed and applied. Slurry coats of just cement and water are sometimes specified; however, a slurry containing sand is preferred to reduce the cement and water content, and shrinkage.

Several problems can occur when mixing and placing cementitious bond coats. If the grout dries prior to placing the patch or overlay, the bond strength will be severely reduced. Bond grouts or pastes have high cement and water contents, and as such, high shrinkage. They are used along the bond interface where the least possible shrinkage is desirable. Therefore, the mix proportions of the grout are important, because if the grout contains excessive water, it will act as a bond breaker. The person mixing the grout often does not understand the importance of proportioning the ingredients and often overwets the grout at the request of the applicators.

Non-re-emulsifiable latex emulsions are commonly used as bonding agents. Latex-modified cement slurries are especially susceptible to rapid drying and must be covered immediately to prevent skinning, which will reduce bond. The most common types of latex used in concrete are styrene butadiene (SBR) and acrylics. These latexes can be used as bonding agents when mixed as a cement slurry or incorporated into the concrete mixture. However, placing straight latex emulsion directly on the surface will decrease the bond strength. Latex emulsions may coagulate if subjected to high temperatures or if frozen prior to use and should not be used.

WJE has conducted a number of tests on non-re-emulsifiable architectural grade acrylic latex-based liquid emulsions used for bonding new-to-old-concrete. Results from these tests are shown in Table 4.1. When no bonding agent was used on a wet brushed surface, the bond strength increased to 3.1 MPa (458 psi) at 7 days then slowly decreased to near 2.0 MPa (300 psi) after 98 days, when the overlay was continuously cured in water. A significant loss of bond was observed when these specimens were cured for 10 or 28 days in water, followed by air drying, and tested at 98 days. Sandblasting of the surface and applying the overlay without a separate bonding agent resulted in good bond strength of over approximately 1.4 MPa (200 psi) after 98 days for all curing conditions.

When the latex was used neat or diluted on a wet brushed surface, much lower tensile strengths were obtained than those that had no bonding agent. When a latex grout consisting of cement and latex was used on the brushed surface, an increased bond was observed comparable to the sandblasted surface without a bond agent. The data clearly show that, if used, a latex bonding agent should be mixed with cement to form a grout.

Re-emulsifiable polyvinyl acetate (PVA) admixtures can be placed long before the application of the concrete since the fresh concrete will soften and bond to the dry PVA. However, they should not be used in areas that will be subjected to future moisture because the PVA will re-emulsify and lose its bond strength. Since concrete is typically in a moist environment, its use is limited and usually not recommended.

Epoxy bonding agents have been used for cementitious repairs with some success, but these should also be avoided in general. They are available as epoxy latex emulsions or as 100 percent solids epoxy primers. The epoxy must be specially designed for bonding new-to-old-concrete. It should be selected based upon successful past field performance. The patch or overlay must be placed before the epoxy hardens or the bond will be reduced. Special precautions are necessary to achieve a good bond, and personnel who install the repairs should be experienced in using epoxy bonding agents. Epoxy also forms a moisture and vapor barrier. Use on slabs-on-grade or external surfaces of tanks can result in moisture being trapped under the epoxy and failure of the bond due to a buildup of vapor pressure. Freezing and thawing of the saturated concrete can also form ice lenses beneath the epoxy, causing failure of the bond.

4.3 Spall Repair

Spalls can occur due to impacts, corrosion of embedded metals, erosion, or problems such as alkali-silica reaction. Numerous organic and inorganic materials are available for patching spalls. Bituminous repair materials will not be discussed, since they are considered temporary and not applicable to most repairs in nuclear power plants. Inorganic repair materials include portland cement concrete, LMC, modified high alumina cement concrete, and magnesium phosphate concretes. Organic repair materials include epoxy, acrylic, polyester, and urethanes. If the repair must be rapid setting, magnesium phosphate, modified high alumina, and most polymer concretes can support loads within 2 to 3 hours. Normal portland cement concrete and LMC generally require 1 to 3 days of wet curing. Rapid setting magnesium phosphate concretes and polymer concretes should be placed on a dry substrate and should not be wet cured. A table comparing the typical properties of inorganic and organic patching materials is shown as Table 4.2

The repair mortar is generally mixed in rotating drum or pug mill mixers. Small batches can be mixed in a bucket by hand or by a drill-powered mixer. Small inexpensive electric mixers are available that mix small batches in 5-gal buckets. Some resins, such as methacrylate and urethane resins, can be premixed in mixers or the resin can be flooded over preplaced aggregates.

When batching mortar, proper handling, accurate proportioning and thorough mixing are required. The surface of the base concrete and the exposed steel should be primed, if recommended, and some of the mortar should be scrubbed into the surface prior to placing the balance of the repair material. The mortar for small patches is generally placed and compacted into place with a piece of lumber or a small vibrator. Good consolidation of the repair materials is essential to ensure good bond strengths, intimate contact between the patch and the substrate, good consolidation and long-term durability. The surface is generally finished by hand troweling. However, if a trowel is the only tool used to compact the repair material, large quantities of air can become trapped between the substrate concrete surface and the patch material, which results in poor bond. A sound repair requires intimate contact between the cementitious portion or resin in the patch material and the substrate. On horizontal surfaces, the repair material should be brushed into the substrate with stiff brooms or brushes. Then, the remaining concrete should be vibrated into place. On vertical surfaces the repair material should be applied using scrub brushes, working the patch material into the surface and allowing the coarser material to drop away. Then, the remainder of the patch can be laid up, compacted, and completed with a trowel.

4.3.1 Surface Preparation for Spall Repairs

As previously discussed, surface preparation is of critical importance for any spall repair. The portland cement concrete substrate must be clean, sound and surface dry. Grease, oil and loose particles must be removed. Unsound concrete can be identified by sounding with a hammer or chain drag. The edge of the spall should be sawcut or chipped to a near vertical edge, several inches outside of the distressed area. Care should be taken not to cut the reinforcing steel. If a significant

cross section of the reinforcing steel has been lost (over 30 percent), the bar should be removed and an appropriate splice made with a new bar. If the patch is shallow, it is good procedure to not only square off the edges of the excavation but to undercut, or dovetail the edges. This provides a mechanical anchor, in addition to the chemical bonding that will develop.

Some grout materials are well suited for tapers and feather edging, but straight, undercut edges produced by sawcutting are preferred. Feather edges on spall repairs do not work since they tend to ravel along the edge of the repair. Dry cutting is preferred for polymer patching materials since these materials are usually moisture sensitive. If wet sawcutting is used, the concrete should be thoroughly dried prior to patching with a polymer concrete or magnesium phosphate-based concrete.

The importance of surface preparation cannot be over stressed. If the concrete is contaminated with oil or grease, or severely deteriorated due to cyclic freezing or carbonation, the surface should be carefully roughened to remove the contaminants and low strength concrete. Good practice is to mechanically roughen or chip the substrate surface. It is recommended that chipping hammers no larger than 30 lbs be used, and that large 90-lb pavement breakers never be allowed. Large hammers actually decrease the quality of the concrete due to fracturing of the substrate. When bond tests of repairs on such concrete are performed, the failure usually occurs in the substrate concrete at a very low strength.

The excavation must be extended to solid material everywhere. Abrasive blasting of the surface after chipping is recommended, to further clean and provide additional fine texture. In addition, the surface must be blown with compressed air and cleaned of dust and any chips that may have been loosened during excavation.

Other suitable means of surface preparation are steel shotblasting and abrasive grit blasting. Steel shotblasting is recommended for floor toppings because it provides the best surface for bonding; it is rapid and relatively dust free. However, it does not work well on irregular surfaces such as spalls. High-pressure water jetting can be a very effective method of surface preparation, but the concrete should be dried prior to patching. High pressure waterblasting, in excess of 140 MPa (20,000 psi), has been used successfully for surface preparation and does not produce significant microfracturing of the substrate. The equipment for waterblasting concrete is expensive and does not resemble a high pressure water or steam cleaner. As discussed, the best bond strengths will develop if the patch is applied to a relatively dry substrate. However, prior dampening does not cause a serious reduction for portland cement based materials, so long as all excess water is blown or blotted away so the surface is dry.

4.3.2 Corrosion Considerations During Spall Repairs

If steel is included in the excavation, the concrete must be removed behind it so that the steel is completely enclosed in the patch material. This is especially important where the steel has begun to corrode, and particularly if chlorides are present. With no corrosion, the steel serves as a convenient mechanical anchor for the patch. The concrete around the steel should be removed to a minimum of 12 mm ($\frac{1}{2}$ in.) to 20 mm ($\frac{3}{4}$ in.) behind the corroded reinforcing bars. If this is not done, continued corrosion of the steel will exert tensile stresses on the patch and the repair area will spall again in the future. When corrosion is involved, encapsulation of the steel is imperative, to prevent the establishment of electrochemical corrosion cells along the bar. With a bar that is embedded halfway in original, chloride-bearing concrete and halfway in fresh patch material, strong corrosion cells can develop. The half of the bar in the chloride-contaminated concrete becomes anodic and corrodes at a rapid rate, driven by the other half acting as the cathode. New delaminations can develop this way in a matter of a few months to a few years.

Similar reactions can accelerate the corrosion of the steel surrounding the repair of a spall that occurred due to the corrosion. The concrete usually contains a large amount of chloride ions that has allowed the steel to corrode and resulted in the spall. The deteriorated concrete is removed and replaced with repair concrete that does not contain chloride. However, the surrounding original concrete still contains large amounts of chloride. A new corrosion cell then develops, with the part of the steel in the original concrete becoming anodic and the steel in the patch becoming cathodic. This corrosion cell, set up by the difference in chloride ion concentrations, can be very aggressive and cause corrosion of the steel immediately outside of the repaired area at an accelerated rate. In this manner, repairs can accelerate the rate of corrosion of adjacent bars. A high resistance patching material or barrier coating on the steel will tend to reduce this effect. Repair materials with high electrical resistance, such as polymer concrete, tend to isolate the bars in the repaired area from the surrounding concrete.

Protection of embedded steel and other metals against corrosion is of primary importance in most types of structures. Several very effective measures are now available. The FHWA advocates the use of epoxy coatings as a primary protection for reinforcing bars in new structures. However, in view of recently reported problems, in particular the Florida Keys bridges, many astute designers are specifying secondary and even tertiary protection systems.

Several materials are available to be applied directly to the cleaned steel to resist or prevent further corrosion. Some materials act as barriers and others act as sacrificial anodes. The most commonly used barrier type of coating is epoxy resin. The epoxy resin acts as a barrier to chloride ions and oxygen and electrically isolates the section of bar from the concrete. This increases the electrical resistance of the corrosion cell and suppresses corrosion.

The exposed bar should be abrasively blasted to a near white surface to achieve good adhesion for any coating material. Two coats of epoxy should be applied, since the effectiveness of the coating is dependent upon achieving a continuous coating of reasonable thickness. One coat often results in very small pinholes or holidays in the coating. The bond of the concrete to the reinforcing bar may be slightly reduced by the application of epoxy, but stress transfer to the bar still occurs due to the deformations.^(4.9) If bond is important, clean dry grit may be broadcast into the second coat of wet epoxy. Beams tested in flexure with coated and uncoated reinforcing steel carried similar loads and had similar deflections, but the cracking in the epoxy-coated beams were fewer and wider.^(4.9)

Polymer-modified cement-based anti-corrosion coatings for reinforcing steel consist of a mixture of acrylate and cement or epoxy emulsions and cement. Compared to epoxy, they should not decrease the concrete bond to the reinforcing steel and should also maintain a high pH surrounding the bar. However, these coatings are susceptible to deterioration due to continued chloride ingress, and further research on their effectiveness is needed. Other bar coatings incorporate some barrier properties but also contain metals that are sacrificial to mild steel. The advantage of these materials is that the corrosion products of the sacrificial material is not as expansive and detrimental to the concrete as mild steel corrosion products. Sacrificial metal primers require intimate contact with the steel, so the steel must be cleaned to a near-white condition. Laboratory tests indicate that these coating materials improve the corrosion resistance of the coated steel but their effectiveness in preventing corrosion of the unexposed steel in chloride contaminated concrete adjacent to the repair is not yet known. Many of these materials are new and little information is available on their long-term effectiveness.

Increased concrete cover is an effective method for preventing or delaying corrosion of embedded reinforcing steel and can be used as a secondary protection after applying a bar coating in the repair area. Corrosion of steel with only 6 mm (¼-in.) cover is almost immediate but steel with over 50 mm (2 in.) of cover is very difficult to corrode by external chlorides. The chloride

profile in concrete due to externally applied chloride solutions decreases exponentially with depth so small increases in cover provide large increases in protection to the steel.

To accomplish similar protection as increased cover, the repair concrete can be designed for extremely low permeability to water and dissolved salts. The most effective means of accomplishing this include the use of high-range water reducers (HRWR) to reduce the water-cementitious materials ratio to about 0.30 to 0.35, and use of silica fume or latex modifiers. HRWRs, with or without silica fume add little to the cost of a project, compared to the increased quality attained. Epoxy-modified and other polymer-modified, or polymer concretes having very low permeabilities are also available. While more expensive than normal concrete, they have many advantages. They are rapid setting and have high strength, high electrical resistance, and very low permeability to water and chlorides. LMC, polymer-modified, and polymer concrete are not generally practical for massive placements, but has been used for toppings, overlays and thin repairs on major repair projects.

Several admixtures for concrete are available that delay the corrosion of steel embedded in concrete. These materials should be considered when casting new concrete in a corrosive environment and as a secondary protective system in repairs. Corrosion-inhibiting admixtures fall into two categories, calcium nitrite and organic corrosion inhibitors. Calcium nitrite corrosion inhibitors have been available for many years and have shown good performance in laboratory testing^(4.10) and in the field. Organic corrosion inhibitors delay corrosion mainly by slowing the rate of ingress of chlorides into the concrete. Laboratory testing by WJE on calcium nitrite and organic corrosion inhibitors has been very promising. Higher dosages of air entraining admixtures are required to achieve proper air entrainment when using organic corrosion inhibitors, and a slight decrease in compressive strength may be expected. Further research on the effect of organic corrosion inhibitors on the bond and stress transfer of prestressing steel is needed prior to their specification in bonded prestressed concrete applications.

Application of a sealer or membrane to the surface of the concrete can also be extremely effective in reducing permeability and corrosion of embedded steel. Their application to already low permeability concretes can further reduce the permeability. The sealers that have gained the most acceptance are the silanes, siloxanes, epoxies and acrylates. Perhaps the most versatile and most easily applied are the silanes and the siloxanes. These materials, unlike other effective barrier-type sealers, penetrate the concrete slightly. Silanes, being smaller in molecular size, generally penetrate more deeply than siloxanes. Both can be used to treat fine cracks. While they do not fill the cracks, they penetrate the surfaces of the cracks, making them hydrophobic. Silane-treated cracks less than about 0.5 mm (0.020 in.) wide can be effectively sealed, preventing further penetration of the water due to periodic wetting without significant external pressures. Further discussion is provided in the section of this chapter on sealers.

Cathodic protection is an electrical method used to stop active corrosion. Low levels of electrical current are applied to the affected area, making the embedded steel cathodic and preventing corrosion.^(4.11) The DC current is typically supplied through a rectifier to a wire mesh or conductive coating on the surface of the concrete. It is important that uniform current is supplied to ensure all areas are protected and to prevent areas of high current density that can deteriorate the concrete.

Sacrificial cathodic protection is commonly used in structures in contact with soil. Anodes, such as zinc or magnesium, are placed in the soil and connected to the structure. The anodes then corrode in preference to the embedded steel.

Cathodic protection can effectively stop corrosion and is especially useful in concrete containing large amounts of chloride. The systems must be carefully designed and should be frequently monitored.

Besides cathodic protection, the most effective means of providing protection to embedded reinforcing steel is a properly applied epoxy coating to the steel. Thus, if epoxy coating of the steel is the primary bulwark against corrosion, increased cover using low permeability concrete can be the secondary protection, and a corrosion inhibitor or surface sealer can constitute the tertiary protection system. These cumulative systems will help achieve longer durability than if only one system is used.

4.3.3 Shallow Spall Repair

Shallow spalls are considered to be those that are less than 20 mm ($\frac{3}{4}$ in.) deep. Most often these are delaminations caused by corrosion of reinforcing steel with inadequate cover. Shallow spalling of concrete surfaces can also be caused by freezing and thawing of non-air-entrained concrete, which causes cracking of the surface and exposure of the coarse aggregate. Excessive heat and fires can cause shallow spalling of the concrete surface. More serious concrete problems, such as internal sulfate attack and alkali reactive aggregates, cause extensive pattern cracking and shallow to deep surface spalling. Patching concrete that has deteriorated due to internal sulfate attack or alkali reactive aggregate may only be temporary. Total replacement of such concrete may be inevitable.

Small and shallow spalls should be repaired using mortars that do not contain coarse aggregates. Dry pack grouts are commonly used for small repairs on vertical or overhead surfaces. Many portland cement-based prepackaged materials are available that are well suited for shallow depth repairs, even on vertical or overhead surfaces.

The most common polymer concretes for shallow spall repairs contain epoxies and methyl methacrylates (MMA). Polyester grout or concrete is also well suited for shallow spall repairs on horizontal surfaces. High molecular weight methacrylate (HMWM) concretes work well for shallow, horizontal surface repairs but should be limited to small volume repairs. Special formulations must be used for patching vertical surfaces since resin concretes tend to self-level. General information on the use of epoxy and MMA concrete systems is given in ACI 503R^(4.12) and 546.1R^(4.2) committee reports and in the materials section of this chapter.

A thin patch is susceptible to cracking and delaminations. Careful attention must be paid to the initial and drying shrinkage properties of the repair materials. Proper curing is also important. Thin repairs have very high restraint from the base concrete, so even moderate shrinkage may cause cracking. Once cracks occur, shear stresses develop and delamination along the crack or edges may occur.

4.3.4 Deep Spall Repair

Deep spalls are treated in a manner similar to that for shallow spalls; however, coarse aggregate is added to the repair mixture. The maximum aggregate size in the repair concrete should be no larger than one-third the depth of the repair or one-half the clear spacing of exposed reinforcing steel. The use of coarse aggregate reduces the shrinkage, and the coefficient of thermal expansion of the repair material. For polymer concrete, less resin is also needed, thus reducing cost. Cementitious materials are best for large volume repairs since the final properties will be similar to those of the structure. If rapid setting is required, Type III cement, modified high alumina or magnesium phosphate concretes can be used. Caution must be used when placing large volumes of magnesium phosphate concrete because the temperature of the concrete during hardening can get very high. If low permeability and rapid strength gain are needed, MMA, epoxy and urethane polymer concretes can be used for deep, moderate volume repairs. These are discussed in detail in the material properties section of this chapter.

In mass concrete, one of the major problems is the heat buildup that occurs because the size of the element prevents dissipation of the heat. This is not a common problem with repairs

since they tend to be smaller in volume. Class F fly ash can be used as a partial cement replacement to reduce the maximum temperature of the repair mass. The lower the maximum temperature, the less cooling will occur. It is the rapid cooling of the surfaces that causes cracking and high stresses. Consequently, as the temperature reaches its peak, it may be necessary to insulate the repair concrete to reduce the rate of cooling. The concrete should not be allowed to cool more than 7°C (20°F) per 24 hours. Differential cooling, with external portions of the concrete losing heat more rapidly than the interior, is the cause of considerable cracking in concrete. Up to 50 percent fly ash can be used in mass concrete as a partial cement replacement to lower the maximum temperatures reached.

Patches more than 125 mm (5 in.) deep may be built up in two or more layers to prevent excessive heat build up and shrinkage. However, the surface of each lift should be roughened and cleaned to ensure good bond between the lifts, since some materials do not bond well to themselves.

Sawcutting of the edges and surface preparation procedures similar to shallow spall repairs should be followed for deep repairs. After chipping, the concrete surface to be repaired should be sandblasted, then air-swept to remove any debris. Anchor bars should be anchored into the existing concrete at reasonable intervals. These anchors resist the stress at the bond line, due to drying shrinkage and thermal movements. If the repair will be subjected to a severe corrosive environment, stainless steel anchors or pins should be used.

The repair concrete should be scrubbed into the cleaned surface and compacted using internal vibrators. This approach provides some assurance that the patch material will be well compacted and in intimate contact with the substrate. This is particularly important where steel has been exposed and must be encapsulated.

The cementitious patching materials should be moist cured for a minimum of 3 days and preferably 7 days or as long as the manufacturer recommends, whichever is greater. If the patch has been formed, the forms can be considered the curing protection. If forms are not used, the best cure is wet burlap covered with polyethylene sheets. However, if this is not practical, a resin or acrylic based curing compound should be used, at double the rate recommended by the manufacturer. Polymer concrete and magnesium phosphate concrete repairs should not be moist cured.

4.3.5 Overhead or Vertical Repairs

Many proprietary repair materials are available for the repair of vertical and overhead spalls. Dry packing using standard cement and sand is also often used and can be very effective and economical. ACI 546.1^(4.2) discusses the use of dry packing. The dry pack should be proportioned at approximately equal parts cement to sand. Just enough water should be added to make the materials workable but it should have a dry appearance. The mortar should be scrubbed into the surface then compacted using a rod or hammer. It should be placed in several thin lifts compacting each lift until the proper grade is achieved and the exterior can be compacted using a trowel.

The specification for vertical surface repairs submitted by The Tennessee Valley Authority (TVA) and practices recommended by Caltrans include sloping the top and bottom cuts of the vertical repairs at a 3 to 1 slope when using standard repair concretes as shown in Fig. 4.1. This is for both full thickness and partial thickness repairs. If it is impractical to slope the top surface, the repair concrete should be stopped less than 3 in. below the top edge and the repair completed with dry pack. This procedure will aid in compaction and reduce the tendency for cracks along the edge.

Many instances of early failure in vertical patches have been seen, and in nearly every case, the reasons for the failure is found in lack of preparation of the existing concrete, use of an inappropriate material for patching, poor techniques for installing the patch, or lack of proper curing of the patch material.

4.3.6 Spall Repair Near Joints

Spalls often occur along expansion joints. Careful workmanship around the joint is required so that point loads do not occur in repair. The joint should be isolated with caulk and a foam filler that is compatible with the repair material to be used. The foam filler should be cut and installed so the joint is established as designed and that the joint is not bridged with the patching material. If a compression seal is to be installed, the joint should then be widened by sawcutting full depth prior to installing the seal. Many resins react with asphaltic materials and dissolve expanded styrofoam. Closed cell urethane foam is compatible with most polymer resins while expanded polystyrene foam is not.

4.4 Material Properties

4.4.1 Inorganic Concrete Materials

Portland cement-based materials have several advantages for repairs. They are relatively inexpensive, easy to use, and readily available. They have been used extensively in the field and have familiar handling characteristics. They are also generally compatible with the portland cement concrete substrate. Portland cement concrete has the advantage of being the product of a well-controlled, large industry that has been thoroughly researched and studied. The main disadvantages may be occasional poor bonding characteristics, slow strength gain, high modulus of elasticity, high abrasion loss, poor chemical resistance, and the potential for excessive shrinkage and cracking.

Inorganic patching materials can be separated into categories of portland cement-based, high-alumina cement-based, and magnesium phosphate-based materials.

Portland cement concrete — Portland cements are divided into eight types as described in ASTM C150^(4.13) and shown below:

- Type I - For use when the special properties specified for any other type are not required.
- Type IA - Air-entraining cement for the same uses as Type I, where air-entrainment is desired.
- Type II - For general use, more especially when moderate sulfate resistance or moderate heat of hydration is desired.
- Type IIA - Air-entraining cement for the same uses as Type II, where air-entrainment is desired.
- Type III - For use when high early strength is desired.
- Type IIIA - Air-entraining cement for the same use as Type III, where air-entrainment is desired.
- Type IV - For use when a low heat of hydration is desired.
- Type V - For use when high sulfate resistance is desired.

Portland cements are manufactured by finely grinding clinker produced by calcining to incipient fusion a mixture of argillaceous and calcareous materials. Typically, limestone (calcareous) is blended with materials high in silica and alumina such as clay or shale (argillaceous). The materials are ground separately, proportioned and pulverized. They are fed into a kiln and burned to incipient fusion to form the clinker. Gypsum is added to control set when the clinker is ground to portland cement. The four principal compounds present in portland cement are tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium aluminoferrite (C_4AF). Type III cement is high in C_3A , which increases the early strength gain but also increases the heat of hydration and volume change. Type III is commonly used for repairs where rapid strength gain is needed and high heat of hydration and higher shrinkage can be tolerated. Type IV cement or Type I with pozzolan (IP) are used for large or massive pours to decrease the heat generated by hydration and reduce the overall volume change.

The w/c is one of the most important factors in obtaining high strength properties, as shown in Fig. 4.2.^(4.14) The proportioning of ingredients for a small repair is as important as for any concrete pour. The water should be proportioned using an amount that results in a workable mix without excess. Water-reducing admixtures are available to provide workability without an increase in water. These are described in ASTM C494^(4.13) and are available from most concrete suppliers.

The chemically combined water after complete hydration of the cement varies, but is about 20 percent by weight of the cement or 63 percent by absolute volume. The actual value is dependent on many factors, including the chemical composition of the cement. A like amount is strongly adsorbed on the cement hydration products. Water absorbed by the aggregate must also be considered when determining the minimum amount of water required. If the aggregate is saturated and contains excess water, the weight of the excess water should be deducted from the quantity of mix water. If the aggregate is dry, the water required to saturate the aggregate should be added to the batch water. It is good practice to moisten dry aggregate prior to batching, and allow time for the aggregate to absorb the water. The properties of the batched concrete will then be more consistent during mixing and placing. The PCA document *Design and Control of Concrete Mixtures*^(4.14) is an excellent reference on the batching and placement of concrete mixtures.

Durability of portland cement concrete: Several aspects of manufacturing concrete can affect its long-term durability. Initial care can prevent future deterioration due to cyclic freezing, aggressive chemical attack, internal sulfate attack, or alkali-reactive aggregate deterioration. The selection of the proper cement type will help mitigate deterioration, due to sulfates and other aggressive ions. Air entrainment by the addition of an air-entraining admixture, such as vinsol resin, will prevent damage due to cyclic freezing. This is discussed in detail in the section on freezing and thawing deterioration. If aggressive chemical attack is anticipated, the concrete should be coated with a sealer or membrane.

Aggregates: The selection of the aggregate for concrete repairs is as important as when building a new structure, especially if the repair must have good long-term durability. It is best to use aggregates that have a good long-term performance history or that have been thoroughly tested. Because they typically make up 60 to 80 percent of the volume of concrete, the properties of aggregate in large measure determine the properties of the concrete. Aggregates are extensively tested for new structures, but usually not for repairs. Some aggregates are in themselves not durable to freeze-thaw, resulting in surface popouts, D-cracking or general deterioration, especially when the concrete is exposed in a wet condition to freezing and thawing. Other aggregates are not effective in restricting drying shrinkage of the concrete. Still others react with alkalis in the cement, resulting in detrimental expansion and deterioration.

One group, consisting primarily of glassy siliceous minerals, produces an alkali-silica gel that swells profusely with changes in moisture, causing overall cracking and degradation. Alkali-reactive aggregates can be identified by petrographic examination or chemical and physical testing. These aggregates can then be avoided, or the detrimental expansion can be mitigated by the addition of mineral admixtures such as pozzolan or silica fume. Mitigation of ASR is not always successful and it is discussed in detail in the section of this report on deterioration due to ASR.

Another group of aggregates, consisting of fine-grained dolomitic rocks that contain small amounts of clay, reacts with the alkalis in a process known as dedolomitization. In this process, the dolomite is converted to calcite and magnesium hydroxide, an expansive reaction. Fortunately, test procedures are available to identify these materials before any repair or construction starts. Reactive aggregates can be found in nearly every state in the United States. Many of the aggregate have never reacted due to the local cement being naturally low in alkali, but if cements are shipped in from elsewhere, alkali aggregate reaction may result. Careful selection of the aggregate used to extend prepackaged cementitious materials is also required to ensure long-term durability of the repair.

Another problem that can develop with certain aggregates is corrosion of embedded metals due to significant soluble chloride contents within the aggregate. Generally, these aggregates occur along the coasts where testing of aggregate for soluble chlorides is a routine matter to eliminate potential problems. Some aggregates found in the midwest and other interior states test high for chlorides in spite of the fact that they have been used in concrete successfully for many decades. This is because the chlorides are chemically bound in the aggregates and not available for reaction. Testing for chloride is commonly done using an acid-soluble chloride technique on crushed samples of the aggregates. The crushing exposes more of the surface of the aggregate, and the acid dissolves more chloride than would be available in service. Therefore, water-soluble chloride contents on samples with minimal crushing may be a more reliable indicator of actual performance. This problem with testing is currently being studied by ASTM and ACI committees and is discussed in further detail in the section of this chapter on corrosion considerations.

Shrinkage: Shrinkage is an important consideration when choosing a repair material. The lower the shrinkage the more compatible the repair material will be, since the base concrete is fully cured and is dimensionally more stable. The repair materials must not shrink significantly, to retain bond and to make the repair monolithic with the structure. In some applications, such as patching core holes, repair materials with final net expansion are desirable. However, uncontrolled expansion can be very detrimental. A slightly lower modulus and higher creep in the patching material may also help dissipate initial shrinkage and long-term drying stresses.

Good quality portland cement concretes generally have drying shrinkage of about 500 to 800 microstrain. The shrinkage is related to the water content and many repair products sold as low shrinkage materials may actually have higher shrinkage than conventional concretes, as shown in Fig. 4.3. Suppliers of proprietary repair materials sometimes report the shrinkage of the patching material in percentage, such as 0.1 percent. This looks like a small number but actually is the same as 1000 microstrain, a large number and significantly more than normal concrete. Also, it is important to review shrinkage data for the aggregate loading to be used, and not to compare data on repair mixes extended with coarse aggregate if a straight grout is to be used.

Dry pack was cited with good success by many plants for the repair of small spalls. Dry pack grout is mixed with minimal water to a no slump consistency which will hold the shape of a ball when squeezed by hand. The low water content reduces the shrinkage of the materials and allows easy placement on vertical and overhead surfaces. The low shrinkage of dry pack is a primary reason for its widespread use and success.

Admixtures: Many people believe that if a concrete admixture meets ASTM test standards, the admixture will reduce or at least not increase the shrinkage. However, the industry standard for concrete admixtures, ASTM C494,^(4.13) allows up to a 35 percent increase in shrinkage when an admixture is used. Careful selection of admixtures for repair concretes are, therefore, necessary to ensure that the shrinkage of repair concretes are not increased to an unacceptable level. Commonly, repairs are performed with high cement content, high strength mortars. This results in a repair material with a high thermal coefficient, high shrinkage, high modulus and low creep. The advantage of higher strength concretes are that they are generally less permeable assuming they do not crack. A lower cement content material may be more compatible and durable, if the reduction in strength can be tolerated. Strength is typically not as important as good bond and no cracking.

ACI 212.3R^(4.16) reports in detail on admixtures for concretes. Admixtures are classified as air-entraining, accelerating, water-reducing, set-controlling, and admixtures for flowing concrete.

Air entrainment: To withstand the rigors of freezing and thawing (particularly in the presence of deicers), the use of an air-entraining admixture is mandatory. This technology has been

available for over 50 years; however, some specification writers still do not understand where it should and should not be used, and how much. It is commonly specified in new concrete but is often forgotten in concrete used for repairs. Basically, it should be used in all concrete to be exposed to freezing weather, either during construction or in repair of completed structures. The amount of air-entrainment required is discussed in detail in the section of Chapter 3 on freeze-thaw deterioration of this report.

Air-entraining admixtures form microscopic air voids in the concrete during mixing. These air voids improve the workability of the fresh concrete and improve the resistance of the hardened concrete to freeze-thaw damage and scaling. In addition, air-entraining admixtures reduce the bleeding and segregation of the fresh concrete.

Air-entraining admixtures stabilize the air voids that form during the mechanical mixing of concrete. The resulting air content of the fresh concrete can be measured by gravimetric (ASTM C138), volumetric (ASTM C173), or pressure (ASTM C231) means.^(4.13) As with all admixtures, the concrete should be thoroughly mixed after adding any admixtures to ensure their even distribution.

A number of problems can develop when using air-entraining admixtures. Air-entrainers can make high cement content concretes very cohesive and sticky, resulting in finishing problems. Also, except for low cement content concretes, entrained air reduces the strength of the concrete. However, the water reduction allowed by the improved workability of the air entrainment somewhat offsets this problem. Because of the improvement in freezing and scaling resistance, air entrainment should always be used when the repair concrete will be exposed to freezing, especially where the concrete is often wet and where deicers may be applied.

Accelerating admixtures: Accelerating admixtures speed up the initial hydration of the cement, resulting in a higher early strength and a shorter setting time. Accelerating admixtures allow earlier stripping of forms and earlier finishing. They also increase the resistance of patches or plugs to hydrostatic pressure. It must be emphasized, however, that accelerating admixtures do not lower the freezing point of fresh concrete.

Portland cement can be manufactured to increase early strength gain by adding accelerating admixtures or by grinding the cement finer. However, the initial shrinkage may be higher, so proper attention to aggregate gradations and curing is essential when using these cements.

The most common accelerating admixture is calcium chloride. An addition of 2 percent calcium chloride by weight of cement has, in the past, been commonly used to achieve high early strengths. Higher dosages may result in decreased ultimate strength. Because it promotes corrosion, calcium chloride should not be used in concrete containing any embedded metal, including reinforcing steel. Many types of nonchloride accelerators are now available from admixture suppliers.

Accelerating admixtures affect concrete in a number of ways. They increase the creep and shrinkage in the hardened concrete, and although the early freeze-thaw resistance of accelerated concrete is higher than that of regular concrete, its long-term freeze-thaw resistance is lower. In addition, the resistance to sulfate and alkali-silica attack is lowered by the addition of accelerating admixtures. Caution must also be used when using accelerating admixtures in hot weather, as extremely rapid setting and excessive cracking can occur.

Typical compressive strengths of portland cement concrete, accelerated with calcium chloride and a commercially available chloride-free accelerator, are listed below for 8-bag portland cement concrete with 9.5 mm ($\frac{3}{8}$ in.) maximum aggregate.^(4.17) While the strength gain is rapid, the shrinkage of these mixes is expected to be high.

	Compressive strength 2-in. cubes, MPa (psi)			
	3 hrs	6 hrs	24 hrs	7 days
2 percent by weight of cement CaCl_2	—	2.9 (425)	20.7 (3010)	44.8 (6495)
4 percent by weight of cement CaCl_2	1.2 (180)	5.9 (855)	27.0 (3915)	49.8 (7215)
Daraset (7 percent by weight cement)	—	1.6 (240)	31.4 (4555)	57.3 (8315)

Retarders: Retarding admixtures delay the initial set of the concrete. They are used during hot weather to extend the working time of the concrete. Retarders are also used in grouts that will be pumped or that require long setting times. In addition to their set-delaying effect, retarders tend to reduce bleeding and segregation in fresh concrete.

Retarders should be tested before use to determine their effect on the water requirement, slump, air content, setting time, strength, and shrinkage of the concrete. Concrete treated with a retarder must be properly cured, as its longer setting time makes it more susceptible to plastic shrinkage cracking. Thin overlays are especially susceptible to cracking so retarders should generally be avoided.

Water-reducing admixtures: Water-reducing admixtures are used to increase the slump or flow of concrete without increasing the water content. This is helpful when the concrete placement will be difficult to consolidate effectively, or when a more workable concrete is needed.

Water-reducers can also be used to decrease the amount of water required for a given slump. The decrease in required water will lower the w/c, leading to increased strength, reduced shrinkage, and decreased permeability. Water-reducing admixtures can reduce the water demand by 5 to 10 percent. High range water reducers (HRWRs), also referred to as superplasticizers, may reduce the water demand in the concrete by up to 10 to 20 percent.

The HRWRs themselves have added immeasurably to the versatility of concrete. When the earlier nuclear plants were constructed, HRWRs were not yet in wide usage. This did not occur until the late 1970s. Contractors faced a very difficult challenge, in attempting to consolidate high cement content, low slump concretes within forms that contained so much steel that merely penetrating between the bars with an internal vibrator required a masterful touch. The arrival of the HRWRs changed all that. Now a concrete can be batched with enough water for a 25 mm (1 in.) slump, then dosed with enough HRWR to boost it to an 200 mm (8 in.) slump, without detrimentally affecting its hardened properties.

In general, the workability and finishability of concrete is increased when water-reducers are used. The amount of air-entraining admixture may often be reduced when using water-reducing admixtures, because the two types of admixtures often increase each others effectiveness.

Concretes containing water-reducers can experience rapid slump loss before initial setting. This often necessitates that the admixture be added to the concrete in the field instead of at the batching plant. As with retarders, water-reducers should be tested before use to determine their effect on water requirement, slump, air content, setting time, strength, and shrinkage of the concrete. This is especially important when two or more admixtures are being combined.

Slump: A short discussion of slump is necessary due to the tendency of engineers to overly rely on this test. Slump is a measure of the concrete consistency. A low slump concrete is stiff and more difficult to work and consolidate. Low slump mixtures must be used for patching vertical or overhead surfaces without forms. The slump of formed or horizontal placements should be between 75 to 130 mm (3 to 5 in.) to achieve good consolidation and corrosion protection while avoiding segregation.

The importance of concrete slump on the final properties of the concrete is commonly overrated. Slump tests have been used for many years but their meaning is often not properly understood by engineers. The slump test, ASTM C143,^(4.13) is a rapid method to provide information on the workability of the concrete, but provides little information on the final physical properties of the concrete. In normal concrete mixtures, a mix with a higher slump is more easily vibrated. However, a mixture with a good gradation or added sand can have a lower slump but be more workable and respond better to vibration than a coarse-graded mix with a higher slump. It should not be assumed that a higher slump mix will be more workable. If the mix is too wet and has a very high slump, segregation will occur.

It is generally agreed that the concrete should have the driest consistency that is practical and allows for good consolidation. However, the slump or consistency required will vary depending upon the application and placement procedures as well as the concrete mix ingredients. Slump should not be used to compare the consistency of mixtures containing different proportions of ingredients. Slump is not a measure of the water content or the water to cement ratio. With the introduction of HRWR water-reducers, a low water content mix can be supplied at a variety of slumps.

The amount of air entrainment will effect the slump of the concrete. Higher air content mixtures will have higher slumps. Slump is also affected by the mix temperature and cooler concrete will have higher slumps. In general, a 7°C (12°F) increase in temperature will result in a 25 mm (1 in.) loss of slump, as shown in Fig. 4.4. The age of the concrete also has an important effect on the slump test results.

The slump test is most useful when it is used in conjunction with knowing the air content and the unit weight of the plastic concrete. If the slump is high or on target and air content is on target but the unit weight is low, this indicates that the concrete has an excess of water or an error in batching or change in the aggregate gradation has occurred.

The standard slump test procedure is not applicable for fiber reinforced concrete mixtures. An inverted slump cone test has been developed for fiber reinforced concrete, ASTM C995.^(4.13) An inverted slump cone is filled, supported 75 mm (3 in.) over a bucket and a vibrator is inserted into the concrete. The time required for the concrete to flow out of the cone is recorded. This indicates how well the mixtures will respond to vibration.

Admixtures for flowing concrete: Admixtures for flowing concrete can provide for cohesive concretes with slumps over 190 mm (7½ in.). These flowing concretes can be self-leveling without excessive bleeding or segregation. Flowing concrete is often used for pours containing large amounts of congested reinforcement, locations that are difficult to access, pumped concrete applications, or wherever obtaining proper consolidation is potentially difficult.

Flowable concrete sets in the same amount of time as regular concrete, and can be finished in the same manner as regular concrete. The strength gain, permeability, and freeze-thaw resistance of flowable concrete are also similar to that of regular concrete.

Flowing concrete requires more care in batching and placing than regular concrete. The mix must be designed with sufficient fines to keep the concrete from becoming "harsh" and hard to finish. Higher additions of air-entraining admixtures must also be used to obtain the same amount of air entrainment as in a nonflowable concrete.

Mineral admixtures: The choice of a cementitious system for new construction or repair is no longer a matter of deciding on a list of suppliers and then collecting mill test certificates for their cements. A great deal of concrete, particularly that used in high strength, mass or in extreme environmental situations, contains more than portland cement in its cementitious component.

Fly ash has been in increasing use for decades, while silica fume has recently become the mineral admixture of choice for high strength applications. Both admixtures can significantly reduce the permeability of the concrete when properly used. Within the last 5 years, concrete with a guaranteed 56-day strength of 97 MPa (14,000 psi) has been used in structures in Chicago. The Union Square buildings in Seattle employed concrete with a strength of 131 MPa (19,000 psi). Such strengths would have been impossible to attain without silica fume and its required counterpart, a HRWR admixture. Due to its extremely high fineness (0.2 microns, 60 times finer than portland cement, "finer than tobacco smoke"), the silica fume without a HRWR would cause such a high water demand that high strength would be out of the question. However, caution must be used when specifying these types of high strength concretes for repair due to their high-early modulus, high thermal coefficient, and low creep. Shrinkage testing should be performed and attention to extended moist curing is essential.

Preplaced aggregate repairs — Preplaced aggregate repairs have the advantage of minimizing the paste volume and reducing the shrinkage of the repair. None of the plants responding to the survey reported that they used preplaced aggregate techniques. Preplaced aggregate concrete is a system in which coarse aggregate is placed in the form, and a special cement-sand grout is then pumped into the form, starting at the bottom, to fill the voids between the aggregate and create a solid concrete unit. The point-to-point contact of the preplaced aggregate particles reduce shrinkage to a negligible amount. Drying shrinkage, following proper curing, is normally in the range of 200 to 400 millionths as compared to ordinary concrete, which ranges between 500 and 800 millionths. Excellent information on preplaced aggregate concrete can be found in Chapter 7 of Ref. No. 4.18 and Chapter 38 of Ref. No. 4.19.

A common usage of preplaced aggregate concrete is in constructing high-density biological shielding of nuclear reactors and other equipment using radioactive substances. Heavy aggregates such as barite or even iron ore are required, which because of their mass, angular shape, and friable nature, are extremely hard to handle within a normal concrete during mixing, transporting, placing and consolidating.

Hand placing of the aggregates is sometimes necessary due to grouting pipes and other embedments. This type of concrete construction is especially suited for underwater concrete. Many bridge piers have been constructed by using the preplaced aggregate concrete method. Proportioning of the grout to be used with the various sizes of coarse aggregate should be accomplished as stipulated in ASTM C938.^(4.13) The grout pipes should be between 20 to 30 mm ($\frac{3}{4}$ to $1\frac{1}{4}$ in.) in diameter, and spaced vertically within the placement at approximately 1.5 m (5 ft) intervals. The aggregate should not be placed until grout pipes are in place. Grout is pumped through the injection pipes, starting at the lowest point in the form. Pumping should be done slowly and at a uniform rate, without interruption, so that as the grout rises in the aggregate-filled form will completely fill all voids. Vibration of the forms in the area of the top surface of the grout during pumping helps to remove air bubbles and improve appearance of the completed concrete.

Shotcrete — The ACI defines shotcrete in ACI 116R^(4.20) as "mortar or concrete pneumatically projected at high velocity onto a surface; also known as air-blown mortar, pneumatically applied mortar or concrete, sprayed mortar, and gunned concrete." Shotcrete can be used for large volume repairs and for general applications over large surfaces when increased cover is needed.^(4.21,4.22,4.23) In a repair project where thin repair sections less than 150-mm (6-in.) depth and large surface areas with irregular contours are involved, shotcrete is generally more economical than conventional concrete because of the saving in forming costs.

Two processes can be used to mix the ingredients. In the dry-mix process, the damp materials are premixed and pneumatically conveyed from the gun through the delivery hose to the nozzle,

where the mixing water is added. In the wet-mix process, all of the materials including the mixing water are premixed before entering the delivery hose.

The operation of dry-mix equipment normally follows this sequence:

1. Cement and damp aggregate are thoroughly mixed.
2. The cement-aggregate mixture is fed into the gun (shotcrete material delivery equipment usually consisting of double chambers under pressure).
3. The material is introduced from the gun into the delivery hose, generally by a metering device such as a feed wheel, rotor, or feed bowl.
4. This material is carried by compressed air through the delivery hose to the nozzle. The nozzle is fitted inside with a perforated water ring through which water is introduced under pressure and mixed with the other ingredients.
5. The mortar or concrete is jetted from the nozzle at high velocity onto the surface prepared to receive the shotcrete.

The operation of wet-mix equipment normally follows this sequence:

1. Cement, aggregate and water are thoroughly premixed, in fact, the mixture is often furnished by a ready-mixed concrete supplier.
2. The premixed mortar or concrete is pumped or otherwise forced through the delivery hose without the use of compressed air.
3. The air is injected at the nozzle to break up the stream of concrete and impart the velocity necessary for shotcrete placement.

Shotcrete suitable for most requirements can be produced by either the dry-mix or wet-mix process. All dry-mix processes use stiff mixtures with a w/c of about 0.4. The positive displacement wet-mix process requires a wetter mixture which is gunned at lower velocities. The advantages of each system are shown in Table 4.3.^(4.21) The cost of the various processes is roughly equivalent, but will vary somewhat with the contractor's background and preference.

A number of manufacturers have formulated special shotcrete mixtures that are primarily used with the dry-mix process. These prepackaged materials are ideally suited for repair operations and significantly reduce the problems associated with measurement of the shotcreted volumes and payment items. One of the desirable methods of measurement and payment is by counting the bags of cement or prepackaged mix used. Prepackaged bags contain all of the ingredients, consequently, the results are more uniform and the actual quantities used reflect more closely to the volume shotcreted. Prepackaged shotcrete can be manufactured with a combination of materials such as cement, aggregate, chemical admixtures (water reducers accelerators; high range water reducers; retarders, etc.), mineral admixtures (fly ash, slag cement, silica fume), various types of fibers, and air entrainment.

Shotcrete has been used to repair deteriorated concrete bridges, buildings, navigation structures, dams, tunnels and many other types of structures. The performance of these repairs has generally been good; however, there are some instances of poor performance. A major cause of poor performance has been inadequate surface preparation of the concrete substrate after the deteriorated concrete has been removed. Satisfactory shotcrete repair is contingent upon proper surface preparation of the concrete to which the shotcrete is to be applied. Another durability problem occurs when the substrate concrete is not frost resistant and the depth of shotcrete is not great enough to prevent frost damage to the remaining substrate concrete, especially if the remaining concrete is exposed and in a saturated condition.

One of the major problems in shotcrete repair is overrun of the estimated quantities. Consequently it is recommended that an engineering study be accomplished to determine the actual depth of deteriorated concrete to be removed. The specifications for a shotcrete repair project should always stipulate how the quantity of shotcrete applied will be measured and paid. ACI 506R^(4.25) provides a good discussion of this subject in Appendix A (Payment for Shotcrete Work). It should be clearly understood by all parties of the contract: how the measurements will be determined, how rebound will be determined, and especially how these determinations relate to payment items.

ACI developed a recommended guide for use in verification of a shotcrete nozzleperson capabilities.^(4.21) This guide was developed because the key person in a shotcrete crew is the nozzleperson; their capabilities drastically affect the success or failure of a shotcrete project. The recommended examination includes a workmanship demonstration and a written or oral examination.

Nozzlepersons should be evaluated prior to serving in that capacity on critical projects such as nuclear power projects. This evaluation should include a workmanship demonstration by spraying shotcrete into an area at least 75 mm (3 in.) deep that contains reinforcing steel and has vertical sides. These panels should be cored and evaluated for strength, voids behind reinforcing and rebound inclusion in corners and at the base of the vertical sides. In evaluating the workmanship demonstration, attention should be paid to the resulting uniformity of the shotcrete's physical properties as well as appearance qualities.

High early strength is often needed when shotcrete has been specified; however, the high early strength of shotcrete, like normal concrete, has a tendency to reduce its final ultimate strength. Twenty-eight day compressive strengths are generally inversely related to the quantity of fast-setting admixtures and to the water-cementitious materials ratio. Calcium chloride is one of the few admixtures that does not significantly affect the final strength of high early strength shotcrete. Unfortunately, calcium chloride must not be used near reinforcing steel because the danger of corrosion of the embedded steel. Eight-hour compressive strengths of 2.7 to 5.5 MPa (400 to 800 psi) with corresponding 28-day compressive strengths of 35 MPa (5,000 psi) are not uncommon with many accelerating admixtures. Without the accelerators, 28-day compressive strengths up to 85 MPa (12,000 psi) have been observed although 50 to 60 MPa (7,000 to 9,000 psi) is more common. Dry-mix shotcrete generally results in higher compressive strengths than wet-mix shotcrete; however, wet-mix shotcrete tends to be more uniform throughout its depth. Prepackaged shotcrete materials contain various ingredients (silica fume, fibers, etc.) and develop varying maximum compressive strengths, from around 55 to 95 MPa (8,000 to 14,000 psi).

Flexural strengths evaluated from sawed or shot prisms general range from 3.5 to 10.0 MPa (500 to 1,500 psi), with 4.8 to 6.2 MPa (700 to 900 psi) being most common. Some of the prepackaged materials are advertised to produce 28-day flexural strengths of as much as 11 MPa (1600 psi). The tensile strength of shotcrete is generally determined by using the splitting tensile strength test, ASTM C496,^(4.13) performed on cores from shot panels. As with concrete, the results range from about 8 to 12 percent of the compressive strength.

The drying shrinkage of shotcrete depends on the ingredients, but it generally falls within the range of 600 to 1100 microstrain (0.06 to 0.11 percent). This is slightly higher than conventional concrete, consequently, the applicator needs to pay close attention to moist curing in order to reduce plastic and drying shrinkage cracking.

Rebound is considered a property of shotcrete; it is aggregate and cement paste that bounce off the surface being shotcreted due to collision with the hard surface, the reinforcement, or other aggregate particles. Rebound should not be worked back into the placement. The quantity of rebound will vary with the position of the nozzle, position of the surface being shotcreted (overhead, sidewall, or floor), the air pressure, cement content, water content, maximum size and grading of aggregate, amount and size of reinforcement, and the thickness of the layers. Typical rebound that should be anticipated for the repair various areas being shotcreted are:

<u>Surface</u>	<u>Percent rebound (by weight)</u>
Floors and slabs	5-15
Sloping and vertical walls	15-30
Overhead work	25-50

Many of the prepackaged materials contain special ingredients that reduce these rebound percentages. Some companies indicate that the use of their product should result in 10 percent or less rebound on sloping and vertical walls. Due to rebound, the cementitious content of the in-place shotcrete is higher, resulting in greater strength than would be expected if rebound had not occurred.

Expansive concrete and grouts: As discussed, initial shrinkage of the concrete or grout is an important consideration, especially for repairs to existing structures. The substrate concrete is normally fully cured, and it is dimensionally stable compared to the new concrete. Shrinkage and drying of normal portland cement concrete results in stress at the bond line interface between the old and new concrete and may result in cracking or delaminations. Expansive concretes are used to reduce the effect of the shrinkage, achieve good bonding, and prevent cracking and delaminations.

It is important to understand how these materials expand and this effect on repair installations. The reasons given for using expansive cement concrete are that the material will grow and fill the cavity or that it will push against the sides of the cavity, thereby locking itself in. Both of these concepts are in error.

During the time the material is plastic, very little if any expansion occurs. This is the only time that expansion could assist in filling the cavity. The other premise, that expansive material wedges into the cavity, is also based on false concepts. Expansive cements are formulated to expand perhaps 500 microstrain (0.05 percent), against a small percentage of reinforcing steel. If restrained by too much steel, or heavy external abutments such as the sides of a patch cavity, no expansion will take place, except in directions perpendicular to the restraint. A small amount of compressive stress may be built up in the patch material, but this stress disappears quickly when the material starts to shrink during air drying.

Nonshrink grouts can be divided into several types: expansive cements, gypsum forming, gas forming, metal oxidizing and calcium aluminate types. Mixtures of portland cement plus expansive cement and plaster, portland cement plus calcium aluminate cement with plaster, and straight expansive cement are available. All of these materials are capable of expanding, but only the straight expansive cement (Type K) should be considered reliable.

Expansive cements contain anhydrous calcium sulfoaluminate, which is converted to ettringite and causes expansion. Anhydrous calcium sulfoaluminate admixtures are also available that can be added to normal cement. The initial expansion occurs over several days, followed by normal

drying shrinkage. The final expansion of the concrete is dependent on the amount of admixture, the curing and the restraint of the concrete. The water content should be kept low and the patch should be properly cured to obtain the best results using expansive cement.

Gypsum is composed of calcium sulfate (CaSO_4). Gypsum hardens by the hydration process and is, therefore, compatible with the hydration of portland cement. Some proprietary materials contained as much as 80 percent of gypsum as CaSO_4 . The gypsum additive may cause the concrete mortar to be slightly expansive, which in controlled amounts is desirable in patching materials to counteract shrinkage. Gypsum reacts with tricalcium aluminate, thereby, forming calcium sulfoaluminate. If the sulfate ion concentration is too high, destructive excessive expansion can occur. However, this destruction can be avoided by using a cement with a low tricalcium aluminate content. The materials that are formulated with excess sulfate and tricalcium aluminate are doomed to failure unless they are protected from moisture. Extended expansion, beyond the first few days after mixing and placing, destroys the integrity of the material, causing cracking, delaminations and spalling.

Gypsum-based materials generally have very high-early compressive strengths. Compressive strengths greater than 48 MPa (7000 psi) in 24 hours and over 83 MPa (12,000 psi) in 28 days have been observed. Most other physical properties are similar to portland cement concrete. Bonding properties are fairly good to a SSD or dry portland cement concrete substrate. However, gypsum-based materials have shown significant loss of compressive strength after prolonged exposure to moisture because of the solubility of gypsum. Gypsum concrete may also have high absorption rates, generally twice that of portland cement concrete.

Metal-oxidizing grouts expand due to gas generated either by the reaction of the cement with aluminum powders or fluid coke formation. The concrete expands due to the formation of the gas. The time and rate of reaction is dependent on temperature and the final expansion can be unpredictable. Careful selection of expansive grouts is necessary to prevent premature failures. In general, where long-term durability is needed, they are not recommended.

4.4.2 Alumina Cement

Alumina cement is a rapid setting cement developed in France in 1908 as ciment fondu.^(4.24) It has high early strength gain and is used for rapid repairs or repairs during cold weather. Alumina cements are commonly used in refractory and heat resistant concretes. Normal portland cement is dehydrated and damaged at temperatures of 250°C (500°F); alumina cements retain strength up to 870°C (1600°F), at which temperature it begins to melt and fuse, providing additional strength. If finely ground topaz or silica flour is added the strength can be maintained up to temperatures between 870°C (1600°F) and 1200°C (2200°F). Therefore, high alumina cement is an excellent material for repairs of concrete subjected to very high temperatures and low humidity. High alumina cements are not resistant to strong caustics or to acetic, hydrochloric or nitric acids.

Alumina cement is manufactured from bauxite. Like portland cement, these cements gain their strength by hydration with the addition of water. The w/c is more critical than for regular portland cement; therefore, it is important to follow the recommended mix ratios. As the name implies, the cement has a high concentration of aluminates. The calcium to aluminate ratio is typically 40:60, while normal portland cement is near 50:50. It is more expensive than normal portland cements and cannot be used for mass concrete due to its rapid cure and high heat of hydration.

Alumina cement concrete is unstable under long-term exposure to high humidity conditions and elevated temperatures over 20°C (68°F). The aluminate structure undergoes a phase shift and crystal structure change, resulting in significant loss of strength of up to 70 percent. Consequently, there is a potential for serious failures if used for structural members.^(4.24) The addition

of calcium sulfate to the alumina cement will reduce the strength loss due to moisture exposure, and these modified high-alumina cements are available for use under long-term high humidity environments.

The modified high alumina-cement concrete has rapid strength gain. Set times vary between 15 and 30 minutes, depending on temperature. Set retarders and accelerators are available from some manufacturers. The setting time may be retarded by the addition of borax or accelerated by the addition of lithium carbonate. Several proprietary materials can provide compressive strengths in excess of 20 MPa (3000 psi) in 3 hours and 35 MPa (5000 psi) in 24 hours. Modified high-alumina cements generally bond better to a damp substrate than magnesium-phosphate concrete, but both bond best to a dry substrate. Prepackaged modified high-alumina materials are usually single-component and water activated. They may be extended up to 100 percent with 10 mm (3/8-in.) pea gravel aggregate. A cure seal is recommended when environmental conditions are hot, dry, or windy. The prepackaged materials are generally self-leveling, but can be modified for low flow applications. Initial shrinkage is generally less than portland cement, thereby increasing its compatibility with the substrate. After a surface skin has formed, the material will not bond effectively to itself. Thus, for deep patches, the patch should be placed in a single lift or abrasive cleaning of the surface between lifts will improve the bond.

Magnesium-phosphate patching material: Magnesium-phosphate cement is produced by calcining mined magnesite, forming magnesium oxide. The hardening reaction is initiated by adding ammonium phosphate solution, which yields magnesium phosphate. This is a chemical reaction, not a hydration reaction. The recommended mix proportions must be adhered to for best results.

Magnesium phosphate concretes bond very well to dry portland cement concrete substrates, but not as well to damp or wet portland cement concrete.^(4.25) Further, most magnesium phosphate concretes cannot tolerate moisture for as long as a week after placement without risk of debonding. This applies to a forced surface dried, saturated substrate (SSD), as the moisture equilibrium, when restored, will result in debonding. Generally, no additional bonding agent is required, but epoxy primers have been used to help ensure the bond.

Magnesium phosphate concrete has much less shrinkage than portland cement concrete during hardening and can be formulated to be slightly expansive. This aids bonding in confined areas such as core holes and when grouting dowels.

Two types of magnesium phosphate concrete are commercially available, one-component and two-component. The two-component consists of the dry material and the phosphate solution. The single-component magnesium phosphate concrete consists of dry materials and is activated with water. Magnesium phosphate concretes can be formulated for hot or cold weather use. Set times are typically 15 to 30 minutes. Most materials are self-leveling and, without modification, do not work well on steep slopes or vertical surfaces.

The preparation of the portland cement concrete substrate is very important for a successful patch. The bond surface must be abrasively blasted to provide surface profile and be clean and dry. Magnesium phosphate will react with aluminum so aluminum tools or mixing bowls should not be used. Excess water should not be added to aid finishing as this will result in a high water content at the surface and may result in high surface abrasion loss. Since reaction gases must be allowed to escape, curing seals cannot be used as they inhibit the hardening process.

Water should not be added to improve the workability of the two-component system or above the recommended dosage for the single component. Prepackaged magnesium phosphate concretes may generally be extended up to 60 percent with sand-free pea gravel aggregate.

Most magnesium phosphate patch failures are due to loss of bond. Bond failures are generally caused by poor workmanship, decarbonation, or moisture present in the portland cement concrete substrate. If the magnesium phosphate concrete sets up too rapidly, a decarbonation reaction with the portland cement may occur and greatly reduce the bonding capacity.^(4.25) The carbon dioxide gases liberated by the decarbonation reaction cause bubbles to form at the bond line, thereby significantly decreasing the bonding area. The decarbonation reaction effect may be mitigated by slowing the setting time of the magnesium phosphate concrete and by abrasively preparing the bonding surface by removing most of the portland cement paste from the coarse aggregate at the bonding surface. The set time may be retarded by the addition of borax or other buffers. Laboratory tests indicate that 30 g (1 oz) of borax per 23 kg (50 lbs) of mix will retard the set time 5 minutes at 22°C (72°F).^(4.25) Magnesium phosphate concrete will generally bond well to itself and can be used in freeze-thaw climates.

Polymer-modified portland cement-based materials — The addition of a latex admixture to normal portland cement concrete enhances its bond strength, tensile strength, improves its resistance to chemicals and reduces its permeability. The latex emulsions include styrene butadiene, polyvinyl acetate (PVA), acrylics and epoxy emulsions. Styrene butadiene and acrylic modified mortars are most common. Latex emulsions can also be added to cement-sand slurry bond coats. Re-emulsifiable PVA bond coats can be placed long before the application of the concrete since the fresh concrete will soften and bond to the PVA. However, they should not be used in areas that are subjected to future moisture as the PVA will re-emulsify and lose its bond strength. Non-emulsifiable PVA bonding agents are available, but the patch must be applied over the wet bonding agent. Latex-modified bonding agents were discussed earlier in this chapter. Epoxy emulsion additives are relatively new but hold much promise for the future.

LMC is most commonly used for overlays of deteriorated structural slabs or bridge decks. It can also be used for patching of both horizontal and vertical surfaces. ACI Committee 548 recently published a document entitled *Standard Specification for Latex-Modified Concrete (LMC) Overlays*.^(4.26) This document includes recommended specifications for LMC overlays. The LMC mix proportions for overlays are recommended to have a minimum cement content of 390 kg/cu m (658 lb/cu yd), a minimum latex admixture content of 121 l/cu m (24.5 gal/cu yd), maximum w/c of 0.4, a slump between 75 mm to 200 mm (3 to 8 in.), and a maximum air content of 6.5 percent. An increase in air entraining admixture may be necessary to achieve a desired air content due to defoaming agents used in the latex admixture.

The substrate should be SSD prior to placing the LMC. A bond coat of latex, water, cement, and sand slurry is sometimes applied to the prepared surface, but tests show that excellent bond can also be achieved by scrubbing some of the LMC concrete into the surface immediately ahead of the placement. The thin layers of separate latex bonding agents may form a skin on the surface shortly after their placement. This skin will prevent bond. Therefore, it may be better to not use a separate bonding agent but to scrub some of the LMC concrete into the substrate surface immediately prior to placing the overlay or patch.

The LMC concrete will also form a surface skin or film and if the skinned surface is disturbed by heavy finishing, the surface will tear and crack. Rapid application of a minimum 2-day wet curing method should be performed to prevent drying of the surface which can result in cracking.

4.4.3 Organic Resin Concretes

Organic polymer materials can be divided into the polyester-styrene, epoxy, methacrylate, and urethane families. These materials can be either formulated on the jobsite or obtained commercially. Polymer patching materials have several advantages over inorganic materials. They are generally rapid-setting and quickly develop high compressive, flexural, tensile and bond strengths. The rapid-

hardening properties of polymer concretes reduces the time required for installation and curing. They have good adhesion to most surfaces, are impervious to water and deicing salts, are not subject to freezing and thawing damage, and are highly abrasion resistant.

Despite the higher material costs, polymer concretes are often the most cost-effective repair material, when one considers their good long-term durability and the speed of installation.^(4.27) The cost of the materials is usually secondary, due to the small amount of materials used and the large cost of labor required to install the repairs.

Moisture in the concrete can be very detrimental to the bond of the polymer concrete. The substrate moisture can be evaluated by using a vapor test developed by the Rubber Manufacturers Association, Inc. A small dish of calcium chloride is placed within a small chamber sealed to the prepared surface. The weight gain is measured after a minimum of 60 hours. If the moisture vapor transmission is greater than 14.6 kg/1000 sq m/24 hrs (3 lbs/1000 sq ft/24 hours), the surface should not be overlaid with a moisture sensitive polymer topping or membrane. Moisture in the concrete can also be evaluated by tightly covering the surface with rubber or plastic sheeting and, if visible moisture has accumulated under the sheeting after 24 hours, it is considered too high for application of most polymer toppings. If no moisture is present, the surface may or may not be dry enough for the topping and if in doubt the vapor test should be performed. Concrete resistivity tests and relative humidity measurements can also be used to estimate moisture levels within the concrete. If moisture cannot be avoided, formulations of epoxy resins are available that will tolerate moist conditions.

The concrete surface and the air temperatures must be within the range specified by the material manufacturer for proper curing. High temperatures can cause rapid hardening and result in consolidation problems, high shrinkage and poor bond. During cold weather, generally less than 4°C (40°F), polymerization may be incomplete. In cold weather, enclosures using heated air are recommended and localized heating should be avoided. Obtaining good results in cold weather is difficult; repairs should be delayed until warmer weather, if possible. If below-freezing repairs are necessary, MMA-based and urethane-based materials have provided good performance. Workers experienced in cold weather repairs using polymers are recommended for this work.

Appropriate safety precautions should always be observed when working with organic polymers. Manufacturers' material safety data sheets (MSDS) should be requested from the material manufacturer and consulted before using any polymer resins. Skin contact with most organic polymers should be avoided. Precautions on inhalation should be observed for a number of the resins. Solvents, such as acetone or methylene chloride, are generally used for tool cleanup. Cleaning of tools can be done by wiping with rags saturated with an appropriate solvent. Mixers can be cleaned by charging them with a solvent and coarse aggregates, then operating the mixer. Soap and water wash should be used for final cleaning of tools and of any resin or solvents splashed on the skin. The solvents and some resins constitute a toxicity exposure concern and a potential fire hazard. Information on proper storage and the expected shelf life of the polymers is available from the product manufacturer.

Information on the required safety precautions of using any material must be discussed with and made available to any workers under current right-to-know safety laws. Even portland cement can be dangerous due to inhalation of dust and severe burns due to prolonged contact with wet concrete. Safety training is required for all workers handling polymer or other materials. Morning "tailgate meetings" should also be held to review important safety aspects. Any questions on safety and handling should be directed to the material manufacturer. Specific federal, state and local requirements on the transportation, use, and disposal of polymer resins and components must be followed.

Properties of polymer concretes — The properties of polymer concrete are based on the polymer, the aggregate and the additives or curing agents used. Resin selection should be based upon demonstrated long-term performance in similar applications. Material suppliers should be consulted before use, since the actual properties of specific resins can vary significantly. The curing times and strength development can be formulated over a wide range. The curing times and strength gain of most organic materials are temperature sensitive, that is, as the temperature increases, the working time and set time decrease. The following discussion of the properties of the resins is general. Table 4.2 shows the typical properties of each general resin type compared to portland cement-based materials.

One of the main advantages of polymer concretes, besides their rapid strength gain, are their high flexural and tensile strengths. Generally, polymer concretes have flexural strengths 4 to 7 times higher than portland cement concrete. The direct tensile strength is limited by the aggregate-matrix bond strength but is generally over three times the tensile strength of portland cement concrete. The flexural and tensile strengths are reduced significantly if the aggregate used to fabricate the polymer concrete is wet. Compressive strengths vary significantly depending upon the flexibility of the system. Rigid polymer systems can obtain compressive strengths well over 70 MPa (10,000 psi), but flexible systems may be less than 14 MPa (2000 psi). The bond strength of polymer concrete to hardened dry portland cement concrete is generally excellent and normally exceeds the tensile strength of the portland cement concrete. Bond to wet portland cement concrete is significantly less than to dry portland cement concrete and may be inadequate. Proper resin formulations, the use of primers, good surface preparation and adequate substrate drying are essential to ensure good bond strengths.

The thermal coefficient of expansion of polymer concretes can be over 3 times as much as conventional portland cement concrete. Even normal temperature variations can cause delaminations of improper polymer formulations. Spalling of the surrounding concrete can occur even if the polymer material has good bond strength but is thermally incompatible and subjected to large temperature variations. The inclusion of large well-graded aggregates will reduce the resin content and the thermal coefficient of expansion of the patch, thereby improving the thermal compatibility of the patching material to the substrate.

Polymer materials are exothermic; therefore, they typically harden at an elevated temperature. As the resins cool to ambient, stresses develop that are additive to the shrinkage stresses. Polymer materials typically have higher creep characteristics than portland cement concrete, so if the repair stays bonded and does not crack due to initial shrinkage and thermal stresses, creep should eventually relieve these initial stresses.

Shrinkage strains vary for each polymer, with polyesters generally high and epoxies low.^(4.27) Linear shrinkage of poorly formulated polyester concretes can be as high as 30,000 microstrain (3 percent) and for properly formulated epoxies it can be negligible. Therefore, shrinkage of some polymer concretes can be significant and must be considered.

The properties of some resins change dramatically under high temperatures, especially when the temperature approaches or exceeds the heat distortion temperature (HDT) of the resin. At the HDT, the resin begins to soften and will distort or flow under loading. Some polymer patching materials have thermal coefficients of expansion similar to that of portland cement concrete below 30°C (85°F), but at higher temperatures the thermal coefficient may increase dramatically. If polymer patching materials or overlays will be subjected to elevated temperatures, they should be tested for thermal properties at the expected service temperatures.

Since polymer concretes do not contain significant amounts of water, they are not subject to damage due to freezing and thawing. The following discussions relate to the particular properties

of the polymer resin types.

Epoxy concrete — Epoxy resins used for polymer concrete are typically two-component systems. One component is the epoxy resin and the second component is the hardener or curing agent. Because of their structure, epoxy resins form strong bonds with portland cement concrete, steel and most construction materials. Epoxy resins are commonly used to bond structural cracks in concrete, for grouting dowels, as primers for repairs using portland cement concrete, and as polymer concrete or mortar patching materials. Epoxy concretes and mortars are also used for overlays or thin wearing courses on bridge deck or pavement surfaces. Most epoxy mortars are well suited for applications requiring thin tapers.

Epoxy resin cure times and strengths can vary dramatically. Epoxies generally are relatively slow setting and continue to gain strength for weeks. They are resistant to most chemicals and can be formulated to cure under a wide variety of moisture and temperature conditions. Epoxy resins are available that cure and bond under wet conditions, such as bonding fresh concrete to hardened concrete and for underwater repairs. They have good adhesion properties, high tensile strengths and excellent abrasion resistance. Epoxy resins tend to have high viscosity and putty-like mortar consistency that may be sticky to finish. Compared to other thermosetting resins, epoxies have low shrinkage. However, epoxy resins create rapid bonds, and even the relatively low shrinkage strains can create large stresses on the bond line with the portland cement concrete substrate.

Epoxy resins are not tolerant to significant deviations from the designed component ratio. The two components must be mixed close to the specified ratio or a significant decrease in strength and other properties may occur. Batching should be done in complete units or by using accurate volumetric or weight measures. The normal accepted tolerance of the mixing ratio is plus or minus 5 percent. The individual components should also be thoroughly mixed prior to combining. Equipment is available for automatic metering of resins that helps ensure that proper mix ratios are obtained.

Epoxy concrete can be user-formulated. The resins are usually purchased in two-component systems, to be mixed 1:1 or 2:1. Epoxy resins conforming to ASTM C881^(4.13) Type III are suitable for concrete repair. The resins should be mixed with fine aggregate at a ratio of 4 to 7 parts sand to 1 part resin, by weight. Silica sand passing the 100 sieve can be used for small or shallow repairs. For larger repairs pea-gravel sized or larger aggregates can be added, depending upon the dimensions of the repair. The ACI Committee 503R Report^(4.12) provides guidelines for the use and application of epoxy compounds and recommended mix designs.

Commercially available epoxy-based patching materials are usually supplied in three components: resin, hardener, and aggregate. Only the amount of material that can be used within the suggested pot life should be mixed at a time, although mixing complete packaged units is preferred. Priming the substrate with neat resin is recommended by some material suppliers.

Numerous formulations of epoxy resins are available. The selection of the material for a particular application should be based on specific information and field performance data. Epoxy resin concretes are also available in different formulations for different temperatures. The ratio of the A and B resin components should never be adjusted to change the curing time. Epoxy resins are considered allergenic; therefore, safe handling practices must be exercised.

New formulations of epoxy-modified portland cement concrete patching materials are being developed, using emulsified epoxy resins. The epoxy resins increase the adhesion and strength, and reduce the permeability of normal cementitious repair materials. The epoxy-modified portland cement concrete are lower cost than epoxy polymer concrete, and have a thermal coefficient of

expansion closer to portland cement concrete. Very little performance data are available on these new materials.

Methacrylate-based patching materials: MMA has been used in the polymer concrete industry for many years.^(4.28) It has provided a good level of performance and long-term durability. High molecular weight methacrylate resins (HMWM) are a more recent development, and the long-term performance of HMWM as a patching material has not yet been proven. The MMA resins are more flammable and higher fuming than the HMWM resins. Due to the high modulus of elasticity and thermal coefficient of expansion of the HMWM resins, they should be restricted to small volume repairs.

Methacrylate polymer concrete can be mixer-blended or the resin can be poured over preplaced aggregates. Due to its low viscosity, the MMA resin has excellent wetting properties and excellent bond strengths. MMA resin is a clear thin liquid and highly volatile, with a sharp, pungent odor. Therefore, it may not be practical for interior use. It will provide between 30 and 60 minutes of working time, then cure very rapidly. The resin can be formulated to cure over a wide variety of temperatures and is well suited for use in cold weather repairs. MMA concretes have good adhesive properties, and can be feather edged if necessary. They also have excellent flow characteristics, and can be used for repairs around closely spaced reinforcing or form work.

The portland cement concrete surface should be dry abrasively cleaned and an MMA primer applied. Bonding to wet or even saturated surface-dry portland cement concrete has not been satisfactory. The material has excellent abrasion resistance and is self-bonding. As with all of the organic polymers, a curing compound is not required. However, since the resin is highly volatile, a plastic sheet should be placed over the completed repair for several hours, to reduce resin loss due to evaporation.

The chemicals used in MMA concretes are flammable and may be toxic. Trained workers and proper handling and safety practices are required during the handling, mixing and placing operations. Eye and skin protection should be used, as well as chemical respirators. Extended exposure to the vapors may cause dizziness, headaches or nausea. Proper ventilation is essential. Contact with the skin may cause a rash and should be avoided. The resin is flammable, so open flames must be avoided. The cured resin will burn, but will not sustain combustion. HMWM resins avoid most of the acute toxicity concerns; however, its long-term durability has not yet been demonstrated.

Polyester-styrene concrete: Isophthalic-, vinyl ester-, and fumarate-based resins have been used for polyester polymer concrete. These polyester groups are listed in the general increasing order of bonding strength, chemical resistance and cost. Polyesters have excellent resistance to acids. The polyester resins are less expensive than most other resin concretes; however, there are no commercially available prepackaged polyester concretes formulated for concrete repairs. Polyester resin concretes are most commonly used for large overlays on bridge decks. The California Department of Transportation has had over 10 years of success using polyester concrete overlays on bridge decks subjected to heavy traffic and freezing and thawing.^(4.27) HMWM is typically used as a bonding agent for these overlays.

Polyester resins are commonly manufactured with styrene, which is volatile. Appropriate safety precautions should be observed. Due to the need for formulation, experienced personnel should be used to formulate and install the polyester concrete. Polyester concrete might be considered for large repair projects due to the potential cost savings or when acid resistance is important. The experience using polyester concrete for structural repairs is limited and the long-term durability of such repairs is unknown.

Urethane concrete: Urethanes can be placed over a wide variety of temperatures, and are used when high productivity or rapid curing are required. Urethane resin set times can be formulated to be very rapid; as short as several seconds. Due to the low resin viscosity and the rapid curing times, preplaced aggregates can be used. Two-component mixing guns are well suited for mixing and distributing the resin.

The advantages of a urethane patching material are that set times may be very rapid, it may be used in cold conditions, it bonds well to portland cement concrete and asphaltic concrete, and it is relatively easy and rapid to place. However, some urethane resins expand uncontrollably when mixed with water or moist aggregate. Careful attention must be given to the resin formulations and construction practices and the substrate surface must be dry. Primers, such as epoxy, have been used to increase the bond strength to concrete.

The modulus of elasticity of urethane concretes are typically low, less than 2.8×10^3 MPa (0.4×10^6 psi). Due to the low modulus, the compressive strengths and flexural strengths are also low. A low modulus material has several advantages for patching since they can accommodate long-term thermal and shrinkage strains better than a high modulus material. Therefore, they may be more compatible with hardened portland cement concrete substrates subjected to large temperature variations. The long-term durability of urethane concretes for repair of structures is unknown.

Fiber reinforcement: Many prepackaged materials contain fibers. The fibers can be steel, polypropylene, or glass fibers. The fibers improve the impact resistance, the ultimate tensile strength and may reduce the tendency for cracking due to initial shrinkage. Another advantage is that the fibers may prevent dislodging of pieces if the repair concrete cracks or spalls. However, fibers are difficult to incorporate into the mixture, especially in the relatively large quantities required to provide beneficial effects.

4.5 Repair of Delaminations

Delaminations are horizontal voids in concrete slabs or domes that commonly occur due to corrosion of embedded reinforcing steel. The delaminations will usually result in spalls if not repaired. The extent of delaminations can be located by dragging a chain over the surface of the concrete, hitting the concrete surface with a hammer, or using acoustic or impact-echo techniques. Delaminated areas sound dull compared to sound areas, which have a solid ring when impacted. Delaminations can also occur at the bond interface of overlays or in two-layer slab construction. These are often caused by poor surface preparation of the base slab or drying shrinkage and thermal stresses of the overlay concrete.

Concrete delaminated due to the corrosion of embedded reinforcing steel should generally be removed and replaced as discussed in the spall repair section. Sawcutting and removing the delaminated concrete, preparing the surface and replacing the concrete should provide a durable repair. Delaminations of toppings or areas where the concrete is not removed can be repaired by injection of epoxy resin. HMWM resins have also been used with limited success. Several drill holes should be made into the delamination, using a vacuum attachment to remove the fines caused by the drilling. Coring the holes leaves less fines than drilling but if water is used, the area should be allowed to dry for several days. The resin can be injected in a manner similar to that used for crack injection. The resin is injected under low pressure or by gravity feed until the resin is seen in adjacent drill holes. The process is continued until the delamination has been filled. Resin injections of delaminations are not always initially successful, and subsequent injections may be required. Dowel pins are sometimes inserted into deeply drilled injection holes while the resin is still liquid to provide additional shear transfer. The long-term durability of repairing delaminations with epoxy is generally good, but the condition of the interface and the installation procedures are very important factors. If the delamination was due to corroded reinforcing steel, continued corrosion and delamination must

be expected unless the corrosion is addressed separately.

4.6 Crack Repair

Cracking in concrete occurs for a number of reasons, including plastic shrinkage, drying shrinkage, thermal effects, deflection, stresses, fatigue, or reactive aggregate expansion. Many cracks in structures do not pose problems and do not need to be repaired. The cause of the crack and the potential for problems due to the cracking should be determined. If the cracking may endanger the structural integrity of the member, or its long-term durability, a structural review by a qualified engineer should be performed to determine the reason for the cracking. If the crack width is increasing due to loadings or settlement, replacement or strengthening of the member may be required. If the crack is not structurally significant, but is moving due to cyclic temperature changes, it may be treated as a joint, and routed and sealed with an elastomeric sealant. Cracks in areas subjected to chemicals or aggressive solutions should be filled, to prevent infiltration into or through the concrete. If the crack does not have significant movement, epoxy and HMWM resins are used to structurally bond the cracks, by either pressure injection or by topical gravity feed application. Epoxy injection is commonly used for large or vertical cracks, and HMWM is commonly used for repair of fine cracks on bridge decks.

Cracking was the most commonly reported deterioration in the plant survey. Twenty-five of the 29 plants, or 86 percent, reported cracking problems. Repair techniques for cracks included dry packing, epoxy injection, grout injection, flexible sealing, and routing and sealing. Drilling and plugging of cracks was reported by 3 plants, and 2 plants reported using stitching of cracks using deformed reinforcing bars or steel plates.

A survey by the U. S. Army Corps of Engineers Waterways Experiment Station^(4.29) revealed that cracking was the most frequently occurring problem in concrete construction, accounting for 38 percent of the total defects reported. Common types of cracks include: craze cracking (map, alligator, or pattern cracking), plastic shrinkage cracking, thermal and drying shrinkage cracking, chemical reaction cracking, corrosion cracking, structural cracking, and of course combinations of these mechanisms.

4.6.1 Mechanisms of Crack Formation

Concrete cracking can be classified into two categories: microcracking and macrocracking. The formation of microscopic cracks is the first form of cracking that develops in concrete. Fractures that are visible with little or no magnification form later and are typically referred to as macrocracks.

Any structure that is built of concrete can be expected to crack. The cracks may consist only of microcracks adjacent to aggregate particles, or they may be continuous, wide cracks that extend entirely through the concrete member. The microcracks within the paste, being discontinuous and very narrow, never need to be filled and, indeed, could not be. Large macrocracks may need to be filled, either to re-establish structural integrity or to prevent the penetration of water, other liquids, or aggressive solutions.

A description of cracking is presented in ACI 224R, *Control of Cracking in Concrete Structures*.^(4.30) A detailed study by Smadi and Slate^(4.31) of cracking under short- and long-term loads categorized cracking into three stages, corresponding to three areas of the stress-strain curve. While this study considered uniaxial compressive loads, work referenced by ACI 224R^(4.28) and by Ansari^(4.32) on uniaxial tension specimens indicated similar microscopic behavior, although initiating at different stress levels.

Microcracks occur in all concrete. Microcracking is randomly oriented and located at the interface between the mortar and coarse aggregate. These interfacial bond cracks begin when the concrete is hydrating and shrinking, before any loads are applied. Stresses develop at the paste-aggregate interface as a result of the volume change of the cement paste and the restraint provided by the aggregate. Lower strength concretes have been found to be more susceptible to this form of microcracking because of their lower tensile strengths, and they typically have a greater percentage of coarse aggregate and correspondingly greater restraint.^(4.31)

Smadi and Slate^(4.31) observed that higher strength concretes exhibited less shrinkage microcracking than weaker concretes. With additional external loading, as a percent of respective ultimate strength, higher strength concretes developed interfacial bond cracks and mortar cracks slower than lower strength concretes. As a result, the inelastic deformations of higher strength concretes were less than lower strength concretes. Overall, the better microcracking behavior of high strength concretes was credited to higher bond strengths at the aggregate-mortar interface, higher mortar strength, and smaller aggregate content.

The second stage of cracking in reinforced concrete includes the development of additional interfacial bond cracks and a small amount of mortar cracking. Some combined interfacial and mortar cracking begins to develop during this stage, but is limited to connected interfacial bond cracks around adjacent aggregates nearly in contact with each other. The interfacial bond cracks in this stage are significantly wider, longer, and more frequent than in the first stage, and the stress-strain curve becomes nonlinear. For short-term loading, compressive stresses are at a stress level up to approximately 70 to 75 percent of ultimate for normal-strength concrete, and about 85 to 90 percent of ultimate for high-strength concrete.^(4.31)

During the final stage of concrete cracking, the frequency and length of combined cracks increase, and mortar cracks develop. The rate at which the cracking develops during this stage accelerates, increasing nonlinearly with increasing stress. Initially, carrying capacity is not lost at this stage because of redundancy of the cracked structure. Instead, alternate load paths through the aggregate and mortar are developed. Eventually, the carrying capacity is decreased, however, and the stress-strain curve descends. With sustained loads during the third stage, there is a sharp increase in cracking.^(4.31) Eventually failure can occur if high enough stresses are sustained.

A review of research pertaining to cracking of monolithic reinforced concrete was performed by the University of Illinois.^(4.43) Shrinkage was the primary cause of early cracking, and the most difficult to control. Shrinkage increases the bond strength between the concrete and reinforcement until cracking initiates. This results from radial compressive stresses caused by shrinkage around the restraining reinforcement.

The first cracking occurs in the weakest section of the member, usually at random weak planes. Cracking in beams was found to initiate only near the exterior concrete surface, with crack widths in the order of 0.025 mm (0.001 in.).^(4.31) Cracks then propagate to the reinforcement, and some slip between the concrete and steel occurs. Cracks of this size typically do not allow the rapid ingress of water or soluble salts and are not full depth. Repair of such cracks is rarely necessary.

Secondary cracking occurs as a result of stiffness and strength incompatibility of the concrete and steel, and the associated bond. The steel stresses during this phase are usually greater than 96 MPa (14,000 psi). During this stage, additional cracks form with increasing stress. Since the cracks are still very narrow and less than 0.05 mm (0.002 in.), they do not penetrate full depth, and under normal pressures should not leak.

Concrete is in the final stage of cracking or "equilibrium stage," when additional cracks do not form, and cracks widen instead. At this stage, stresses in the steel often exceed 200 MPa (30,000 psi), and slippage of the reinforcement continues.

The effects of creep in both the reinforcement and the concrete under sustained loading will cause the width of cracks to increase with time. Crack widths are commonly considerably less at the reinforcement than at the outer surface of the member due to the restraint of the reinforcing steel.

When a crack develops in reinforced concrete, the concrete fractures and slippage occurs between the reinforcement and the concrete. The tensile stress that had been carried by the concrete prior to fracture is then transferred to the reinforcement located within the cracked region. The concrete stresses on both faces of the fracture are then near zero, and decreasing slippage between the concrete and reinforcement extends into the uncracked section on each side of the fracture. The tensile force in the reinforcement is gradually transferred to the concrete within this bond-slip distance, and at the end of the bond-slip, the tensile stresses have been fully transferred to the concrete. In other words, strain compatibility between the concrete and reinforcement does not exist with the bond-slip distance.

Within the bond-slip length, the concrete tensile stresses are typically less than the tensile strength of the concrete. As a result, no additional cracking occurs within the bond-slip distance. This bond-slip distance determines the minimum crack spacing that can develop.

4.6.2 Crack Width and Durability

The characteristics of the transverse cracking that will lead to problems such as accelerated corrosion of the reinforcing steel, deterioration of the concrete, leakage to structural members and components beneath the deck, and appearance concerns are difficult to assess. It is important to understand the reason why cracking is regarded as a problem, to place limits on acceptable crack widths, and to recommend procedures for the effective repair of the cracks.

Once a member cracks, deterioration may occur rapidly. In concrete exposed to corrosive environments, rapid corrosion of the embedded reinforcing steel may result. Cracks enable corrosive chloride ions and carbon dioxide to penetrate into the concrete and to the reinforcing steel surface. Thus, certain cracks to the depth of the steel will tend to increase deterioration of reinforcing steel due to corrosion.

Cracks that allow water penetration may result in rapid saturation of the concrete, rendering marginally air-entrained concretes or concretes with reactive or unstable aggregates less durable during winter freezing conditions and wetting and drying cycles. Loss of fluids and infiltration of ground water are other concerns.

Large crack widths may cause adverse public reaction to a structure because of aesthetics. The maximum crack width, as determined by aesthetics, is dependent on lighting conditions, the viewing distance and the crack length. In some cases 0.3 mm (0.010 in.) has been accepted as the smallest width of a crack that is unsightly and unacceptable.^(4.33) However, this value is subjective.

A debate entitled *Crack Width, Cover and Corrosion*^(4.34) was conducted at the ACI Fall Convention in New York in 1984. Two theories emerged regarding how cracks reduce the service life of structures. The first theory is that cracks significantly reduce the service life of a structure by permitting access of chloride ions, moisture and oxygen to the reinforcing steel, not only accelerating the onset of corrosion, but providing space for the deposition of corrosion products. The second theory is that while the cracks may accelerate the onset of corrosion, such

corrosion is localized and confined to the intersected reinforcing bars. After a few years of service there is little difference between the cracked and uncracked concrete. Factors that may influence corrosion were listed. These include: crack arrangement, crack width, crack depth, shape orientation of the crack with respect to reinforcing, intensity of the crack, origin of the crack, type of structure, and service environment.^(4.33)

Cracks greater than 0.05mm (0.002 in.) are approaching the width that may allow infiltration of water and salts. The configuration of the crack is also important as most cracks narrow quickly with depth. A very wide surface crack that quickly narrows with depth may not be as detrimental as a narrower surface crack that has parallel sides. The crack width at the bar is a function of the origin of the crack, amount of cover, stress in the steel, steel and concrete creep, reinforcement ratio, arrangement of the bars, diameter of bars, and profile of the stresses.

The orientation of exposed reinforcing steel with respect to a crack is important. Where a crack is generally transverse to the reinforcement, only localized corrosion may occur. It has been suggested that corrosion is limited to about three bar diameters away from the crack. However, recent laboratory studies by WJE on cracked No. 3 bar reinforced concrete beams found significant corrosion as far as 130 mm (5 in.) away from the crack location. Subsidence, common below reinforcing steel during casting, can provide a path along the bottom of the bar for chloride-containing water, and corrosion may occur even though the crack is transverse to the bar.

Generally, cracking tends to follow directly over the reinforcing, since the bars delineate a weakened plane. This increases the potential for corrosion of reinforcing steel along the length of the bars. Where the crack coincides with the bar, the passivity is lost at many locations and corrosion can proceed at an increased rate.

The rate of corrosion is related to the reactions at the cathodes, the resistivity of the concrete, and the size of the cathode. At early ages, wider cracks produce greater corrosion.^(4.34) For continued corrosion to occur, oxygen and moisture must be supplied to other parts of the same bar, or to bars that are in electrical contact with that bar. If the combination of density and the thickness of the concrete cover is adequate to restrict the flow of oxygen and moisture, and the resistivity of the concrete high enough, then the corrosion processes will be slowed.

Cracks may also cause reduced durability of epoxy coated reinforcement. Recent work by Perragaux and Brewster^(4.35) indicates that concrete cracking may result in the deterioration of epoxy coatings, and corrosion of the reinforcing steel may occur. Laboratory studies by Clear^(4.36) and Pfeifer^(4.37) indicate that while the corrosion resistance of reinforcing steel is greatly enhanced by the epoxy coating, corrosion can occur if breaks in the coating are present. Therefore, if cracks intersect the epoxy coated bar in an area containing a break or holiday in the coating, under film corrosion can occur.

The following table from ACI Committee 224 document *Control of Cracking*^(4.30) relates the tolerable crack width for reinforced concrete and the exposure condition:

Exposure condition	Tolerable crack width, mm (in.)
Dry air or protective membrane	0.41 (0.016)
Humidity, moist air, soil	0.30 (0.012)
Deicing chemicals	0.18 (0.007)
Seawater and seawater spray; wetting and drying	0.15 (0.006)
Water retaining structures	0.10 (0.004)

It is stated within the ACI 224 document that "values of crack width are not always a reliable indication of the corrosion and deterioration to be expected. In particular, a larger cover may be preferable for corrosion control in certain environments. Thus, the designer must exercise engineering judgement on the extent of crack control to be used."^(4.28)

Campbell-Allen and Hughes^(4.37) discuss crack width limitations. Their limits consider corrosion of reinforcement, maintenance of the structural function, and aesthetics. They suggest that the following crack widths are appropriate:

Condition of exposure	Maximum crack width, mm (in.)
Mild to moderate environment	0.3 (0.012)
Moderate to severe environment	0.2 (0.008)
Very aggressive environment	0.1 (0.004)

There are many contrary opinions as to limiting crack width for concrete structures to prevent corrosion. Some of these are shown in Table 4.4. Some of the research studies have found correlations between crack width and deterioration of concrete, while others have not. Beeby^(4.39) stated that there was no clear relationship between crack width and the amount of corrosion, although larger crack widths increase the probability of corrosion. Beeby also states that there seems to be no economical way of reducing crack widths by arrangement of reinforcement.

In general, the following values seem to be generally referenced for allowable crack widths: for severe exposure to deicing chemicals or for water tightness 0.1 mm (0.004 in.), for normal exterior exposures or interior exposures subjected to high humidities 0.2 mm (0.008 in.), and for internal protected structures 0.3 mm (0.012 in.). For structures containing chemicals or fluids where containment is essential, cracks widths greater than 0.05 mm (0.002 in.) may not be tolerable. Numerous investigations by WJE on cracked decks find water leakage through such cracks having surface widths of only 0.05 to 0.20 mm (0.002 to 0.008 in.). As discussed, the shape configuration of the crack is important, so that the surface width may not be a good indicator of the cracks effect on durability.

4.6.3 Selecting a Crack Repair Method

Before a crack is repaired, it is necessary to identify the cause of the crack to determine whether it will move. Conditions that may cause cracks are identified in Table 4.5. This table also includes a general guide to whether the crack will remain "active" or "dormant." Active cracks are those in which the mechanism causing the cracking is still at work; if one cannot verify the cause of the crack, it should be considered to be active. Dormant cracks are those that were caused by a condition that is not expected to recur.

The U. S. Corps of Engineers^(4.39) recommends that the following procedure be followed in order to identify the causes of cracks.

- Step 1 Examine the appearance and the depth of the cracking to establish the basic nature of the occurrence, such as, pattern or individual cracks, depth of cracks, open or closed cracks, and extent of cracking.
- Step 2 Determine when the cracking occurred. This step will require talking with the individuals who operate the structure and possibly those involved in the construction.
- Step 3 Determine if the cracks are active or dormant. This step may require monitoring the cracks for a period of time to determine if crack movement is taking place. Also, attempt to determine if the crack movement detected is growth or simply cyclical opening and closing such as caused by thermal expansion. Cracks that are moving but not growing should be treated as active cracks.
- Step 4 Determine the degree of restraint. This step will require a thorough examination of the structure and the construction drawings, if available. Both internal restraint (caused by reinforcing steel, embedded items, etc.) and external restraint caused by other elements may be present. A checklist for determining the cause of cracking is presented below. Using the checklist, eliminate as many potential causes as possible. If more than one potential cause remains, the final determination may require a laboratory analysis of concrete samples or a detailed stress analysis.
 - Check for major errors in design
 - Check easily identifiable causes:
 - corrosion of reinforcement
 - accidental or impact loading
 - poor design detailing
 - foundation movement
 - Check other possible causes, such as:
 - incidents during construction
 - shrinkage induced stresses
 - temperature induced stresses
 - volume changes
 - chemical reactions
 - moisture changes
 - freezing and thawing

Once the cause of the cracking has been established, the following questions should be answered.

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

Once these questions have been answered, potential repair methods can be selected as outlined in Figs. 4.5 and 4.6. Use of these figures may lead to consideration of several applicable crack repair methods. Final selection of a method and a repair material should take into account ease of application, durability, life-cycle cost, available labor skills and equipment, and appearance of the final product.

Once the reason for the cracking has been determined and repair is necessary, it is important to determine methods for repair. Reports on repair of cracks, such as those by ACI Committee 224^(4.30), should be reviewed. Usual methods for crack repair include epoxy injection, high molecular weight methacrylate (HMWM) topical treatment, silane and siloxane sealers, routing and sealing, and stitching. According to ACI 224,^(4.30) cracks as narrow as 0.05 mm (0.002 in.) may be bound using epoxy injection. HMWM resin and silane or siloxane sealers will work on cracks of even finer dimensions. Recent research by WJE indicates that silane sealers effectively prevent infiltration of static water into crack widths up to 0.5 mm (0.02 in.) based on laboratory ponding tests. HMWM resins have been effective, when properly applied, in bonding and preventing infiltration of deicing solutions into both wide and hairline cracks.

The need for strengthening should be considered prior to selecting a repair. If the element requires strengthening, the cracks can be rebonded with HMWM or epoxy resin, or the member can be strengthened by stitching or external stressing. If moisture is a concern, sealers may also be applied.

4.6.4 Dormant Cracks

Crazing is a dormant cracking caused by poor or inadequate curing practices. It appears similar to mud cracking but extends through only a thin top layer of cement paste. Early application of curing eliminates it. While crazing does not affect the structural integrity of slabs, it is unsightly and sometimes suggests to the owner that he has an inferior product. Repair is rarely indicated for such an essentially cosmetic problem.

Plastic shrinkage cracks are typically wide, short, and discontinuous and are easily repaired. They are dormant and typically are not a structural problem but may expose the interior of the concrete or local areas of embedded steel. If concrete durability or corrosion of embedded steel is a potential problem, the cracks should be filled. This is easily done on horizontal surfaces by laying a bead of HMWM or low-viscosity epoxy on top of the crack or flooding the surface and allowing gravity to fill the crack. These materials are capable of penetrating to a crack width of 0.1 mm (0.005 in.) or less, so they should fill nearly the full depth of a plastic crack. Trial sections are recommended and penetration can be verified by coring. Any excess material that hardens from the concrete surface is readily removed by grinding or abrasive blasting. Plastic cracks do not typically occur in vertical surfaces since the forms prevent drying.

Cracks in recently constructed structures can sometimes be repaired by autogenous healing. Healing of the crack can occur by continued hydration of cement across the crack when the concrete is maintained in a saturated condition. If the concrete is older or the cement is mostly hydrated, autogenous healing is difficult. A selection procedure for the repair of dormant cracks is shown in Fig. 4.6. The selection of a repair procedure is based on the strengthening requirements and the water condition.

Resin injection of dormant cracks — Epoxy injection has been successfully used in the repair of cracks in buildings, bridges, dams, and other types of concrete structures. However, unless the crack is dormant (or the cause of the crack movement is removed, thereby making the crack dormant), it will probably recur, somewhere else in the structure.

Epoxy injection generally consists of drilling holes at relatively close intervals along the cracks, in some cases installing entry ports, and injecting the epoxy under pressure. For more massive structures, an alternate procedure consists of drilling a series of holes, usually 22 mm ($\frac{7}{8}$ in.) in diameter, that intercept the crack at a number of locations. Typically, holes are spaced at 1.5 m (5 ft) intervals.

Pressure injection of cracks is labor intensive and time consuming. It is generally limited to structures containing a few large discrete cracks. Epoxy is the most commonly used resin for pressure injection applications, although other resins can be used. Cracks typically in excess of 0.05 mm (0.002 in.) wide can be sealed and injected using epoxy resins. Detailed information on epoxy injection is included in ACI Committee 503R^(4.12) and 224^(4.30) reports. Personnel experienced in epoxy injection should be used for this work.

Prior to injection, contaminated cracks should be cleaned by flushing with water or solvent. The water or solvent should then be blown out of the crack by compressed air and the crack allowed to dry. Moisture on the crack surface can reduce the bond of the epoxy to the crack faces. The surface of the crack should be sealed with a strippable plastic or polyester putty. Entry ports consist of fittings drilled into the crack or placed on the surface. The resin can also be injected through gaps in the cover, using a special gasket type of nozzle.

ASTM Standard C881,^(4.13) Type I, low viscosity grade epoxy is suitable for crack injection. Mixing can be done by batch or by continuous machine mixing. The maximum pressure of the injection must be chosen carefully, based on experience. Typically, the cracks are injected starting at the bottom of a vertical crack or at one end and moving toward the other, changing ports as the epoxy appears in the adjacent port.

Gravity feed techniques - HMWM resins were developed for topical treatment of bridge decks that contain numerous fine cracks. This technology was first developed by Caltrans in 1981.^(4.25) Due to the low viscosity of the HMWM resin, (8 to 20 cps, similar to diesel fuel), it readily flows into very fine cracks, even less than 0.02 mm (0.001 in.) wide, by gravity. It is not as effective in filling cracks wider than 0.25 mm (0.010 in.) due to lower capillary forces. Epoxy resins are also available to repair cracks by gravity feed. The epoxy resins typically do not penetrate fine cracks as deeply as the HMWM and they are slower curing. However, epoxies may be the best selection for gravity feed of cracks wider than about 0.25 mm (0.010 in.)

HMWM is best suited for horizontal surfaces, but it can be applied to vertical walls or injected under low pressures. The HMWM has a high solvent capacity which enables it to bond through lightly contaminated surfaces. Curing compounds and asphaltic materials should be removed prior to treatment since the resin will solvate them and thicken, causing poor final properties. The cracks must be dry, since water will prevent penetration and dilute the resin, also resulting in poor polymerization. HMWM has low volatility, so the resin will not readily evaporate. The standard MMA resins are not suitable for crack filling due to their high volatility.

The HMWM monomer is catalyzed with a metallic drier and peroxide to initiate polymerization. The resin is then swept, squeegeed, or sprayed on the cracked concrete at a rate of approximately 0.4 l/sq m (1 gal/100 sq ft). The resin flows into the cracks and polymerizes, filling and bonding them. Dry blasting sand should be broadcast into the resin, before the resin hardens, to improve skid resistance.

The performance of HMWM resins can vary. The product selection should be based on satisfactory use in similar applications. A trial application is recommended for large jobs. Generally, the resin may be expected to perform well if used when the concrete and air temperatures are between 7°C (45°F) and 32°C (90°F). Special formulations of HMWM resins are available for use during cold or hot weather. Over 11 years of satisfactory service has been achieved by sealing cracks with HMWM in bridge decks subjected to freezing and thawing and deicers by Caltrans.^(4.21) The cracks have remained bonded, and maintenance has been reduced.

HMWM resins tend to be brittle and abrade off the surface when subjected to traffic. They also do not function as a water repellent. HMWM resins are compatible with silane sealers, so if sealing the surface and bonding of cracks is required, silanes can be applied as a concrete surface sealer, followed by the HMWM to fill and bond the cracks.

4.6.5 Active Cracks

Active cracks must be treated as if they were control joints because, functionally, that is precisely what they are. It must be determined if it is necessary to restore the tensile or flexural strength across the crack. If the strength must be restored, it is best to install an expansion joint nearby prior to bonding the crack. If the crack is simply bonded, a new crack will occur adjacent to the old. A repair methodology for active cracks is shown in Fig. 4.5.

Active cracks that are leaking and must be bonded present the most difficult problem. First the leakage must be stopped and the crack dried. If the crack can not be dried prior to injection, other methods of strengthening should be considered, such as stitching or external stressing.

The crack can often be made water tight by injecting with urethane foam. If a leaking crack is dormant and does not need to be structurally bonded, urethane foams or joint sealants may be used without further treatments. This is discussed in detail in the seepage section of this report.

Active cracks that must be made watertight cannot be easily repaired, because they change in width in response to changes in temperature or humidity. The difficulty is that, in order to provide the resilience in the finished repair that is required to maintain a seal, it is not enough to specify a material that has the proper elongation. A proper shape factor must also be provided. ACI 504^(4.40) contains an excellent discussion of shape factors. A long, thin piece of material will stretch easier and with less stress than a short, thick piece. This is why a material having an "elongation at break" of 100 percent, when tested according to ASTM D688,^(4.13) cannot accommodate more than 3 or 4 percent elongation in a crack or a joint without the proper shape factor. The ASTM test uses a configuration of material that is similar to a dumbbell, with a long, thin center section over which elongation is determined. But a seal in a joint or crack has a short dimension in the direction it must stretch and must have a reasonably large dimension perpendicular to that direction, to provide an ample surface for bonding to the sides.

To provide a watertight seal in a crack, the crack must be routed out to a width of at least 10 mm ($\frac{3}{8}$ in.) to provide some stretching length. In addition, bond must be prevented to the bottom of the crack tip to achieve an unbonded length. This can normally be done easily with tape or wax placed in the sawcut or routed channel. However, if the routed crack has a triangular configuration, it must be partially backfilled with a soft, easily friable material, such as non-drying

caulk, to achieve the required stretching length. If all this is done, with due regard to maintaining scrupulously clean concrete sides in the channel, the repair can be considered long term. However, if exposed to sunlight, certain sealants may degrade in 6 to 8 years due to ultraviolet (UV) light exposure and require replacement. Careful selection of sealant materials are required.

Sealant Selection - Elastomeric sealants are used to prevent the infiltration of water and solids through cracks and joints. The sealants generally consist of urethanes, polysulfides, acrylics, silicones and epoxy. The selection of the sealant is based on the exposure conditions and the amount of movement of the crack or joint. Sealants such as asphalt mastics, general purpose latex sealants, and butyl-based sealants can be used to repair cracks or joints that are not moving and not continuously wet.

Polysulfide sealants and urethane sealants have been used successfully for many years for caulking joints in masonry, concrete and glazing.^(4.24) They will accommodate limited movement up to 25 percent and bond well to glass and metal. A primer should be used for bonding to concrete. They are supplied in either one-or two-component systems.

Silicone sealants can tolerate a wide range of temperatures; from -129 to 260°C (-200 to 500°F). They have excellent resistance to weathering and are the only sealants known to be resistant to nitric acid.^(4.24) They are well suited for a chemical environment that is also at elevated temperatures. Epoxy sealants have high strength and good chemical resistance but lack flexibility, tend to lose bond under wet conditions and are limited to temperature conditions less than 60°C (140°F).

The selection of the sealant should be based on the anticipated joint or crack movement and the exposure and moisture conditions. Closed cell polyethylene foam backer rods are compatible with all types of sealants. The backer rod should be 20 to 50 percent larger than the joint width. Certain types of backer rods, such as Rob-Seal, are bonded to the sides of the joint and act as a secondary seal. In a sawcut joint or wide crack, it is important that the sealant does not bond to the bottom surface. As discussed, the proper shape factor of the sealant is important for moving joints to maximize the tolerable movement.

4.7 Seepage and Grouting

Seepage of water through construction joints and cracks was reported as a problem by several plants responding to the survey. Water seepage through construction joints and cracks was reported. The water seepage may result in leaching, accelerated deterioration of the concrete or unacceptable moisture within the plant, causing safety concerns or corrosion of equipment. As discussed in an earlier section, deterioration of concrete rarely occurs if the concrete is maintained in a dry condition. Long term reactions such as efflorescence, sulfate attack or alkali-silica reactions all require moisture. Therefore, even though seepage has not resulted in deterioration, it is good policy to stop it to prevent possible future deterioration. Obviously, if ground water contamination by the loss of containment fluids is possible, the path of seepage must be identified and repaired. Chemical grouting is the most effective means to stop seepage and provide a durable repair. Hydraulic cement and other specialized coatings are generally not effective in preventing continued seepage but may be effective in slowing moisture movement through areas wetted intermittently.

Chemical grouts - Chemical grouts consist of solutions of two or more chemicals that react to form a gel or a solid precipitate, as opposed to cement or clay grouts that consist of suspensions of solid particles in a fluid. The reaction in the chemical grout may be purely chemical or physicochemical. It may involve only the constituents of the solution or may include the interaction of the constituents with water. This reaction causes a decrease in fluidity and a tendency to solidify

and form occlusions in channels or fill voids in the material into which the grout has been injected.^(4.41)

Chemical grouts provide increased versatility in terms of improved gel timing for injection control, extended injectivity limits, increases in the strength development of various systems, and increased bonding properties. Chemical grout may react to form either soft flexible, semirigid, or hard rigid gels. Caution should be exercised in the use of chemical grouts, since various systems have toxic and caustic constituents.

Most chemical grout systems are used in treatments for increasing the strength and consolidation of soils for subsurface water control, for fissure and crack repair, and for improving load-bearing capacities of foundations. Epoxy and polyester resins are used in shallow crack repairs in concrete and rock and also for the emplacement of rock bolts and anchors. Water based resins, somewhat higher in viscosity than acrylamide and silicate types but much lower than the viscosities common in portland cement grout, may be considered for grouting medium sands and coarse silts.

The viscosity of chemical grouts can be very low and, except for fillers that may sometimes be included, they contain no solid particles. For these reasons, chemical grouts can be injected into concrete containing voids that are too small to be penetrated by portland cement or other grouts containing suspended solid particles. Grouting (both portland cement and chemical) can be used successfully in a moist environment. Grouting operations require a relatively high degree of skill and expertise for satisfactory use. Chemical grouting has the added disadvantage of high cost, limited shelf life, lack of strength, more stringent safety precautions, and significant volume shrinkage if permitted to dry. Most of these grouts are used for soil consolidation or grouting of foundations.

The basic chemical grouts include the following:

- Various silicate systems
 - Silicate-chloride-amide system
 - Silicate-aluminate-amide system
 - Silicate-bicarbonate-amide system
 - Silicate salt of a weak acid (Malmberg System)
- Acrylamides
- Lignins
- Resins
 - Epoxy resins
 - Polyester resins
 - Urethane resins

Typical physical properties of an epoxy resin and a polyester resin system are shown in Table 4.6.

If a crack in concrete periodically dries, it can be injected using epoxy resins. If the crack is continuously wet, a water-activated polyurethane grout is often used. One-component, water-activated poly-urethane grouts can be injected to stop an active leak. If the crack or joint is not wet at the time of injection, water can be incorporated with the urethane injection resin to promote cure. The resin reacts with water and expands, resulting in a flexible foam material. Depending upon the amount of water, back pressure and condition of the crack or joint, several injections may be required to completely stop the seepage. If the crack or joint periodically dries, additional injections may be required, since the foam will contract when dried and may not expand to its original volume when wetted again. Therefore, periodic monitoring is required.

4.8 Sealers and Coatings

4.8.1 Sealers

A sealer is defined in ACI 116R^(4.20) as "a liquid that is applied as a coating to a surface of hardened concrete to either prevent or decrease the penetration of liquid or gaseous media, for example, water, aggressive solutions, and carbon dioxide, during service exposure." In many sections of this report we have talked about the damage that can be caused by the moisture content of concrete approaching the saturation point, for example, freezing and thawing, alkali-silica and alkali-carbonate reactions, corrosion, and general expansion. Moisture itself does not normally cause the damage, but the ambient environmental condition in conjunction with the moisture combine to result in the various modes of concrete deterioration. The best epoxy, methacrylate, and silane sealers usually outperform specialty concretes, such as those containing silica fume with high range water reducer and latex-modified concretes, in preventing the ingress of water and chlorides.

A large number of materials have been used over the years as coatings or sealers for concrete. Some have been effective and some have not. The first systematic study of sealers was done in the WJE laboratories, starting in 1979, and reported in NCHRP Report No. 244 *Concrete Sealers for Protection of Bridge Structures*.^(4.42) Before this study was done, reliance was placed primarily on subjective information and limited manufacturer's data. However, very little performance testing had been done, as the literature search for the NCHRP 244 project quickly revealed. The most popular time-honored material used by many state DOTs was boiled linseed oil. Linseed oil is a drying oil similar to a paint without pigment. The material is very inexpensive in large quantities and is easy to apply.

Other materials have been used to a lesser extent. These are sodium or potassium silicates, stearates, silicones, asphaltic emulsions, and cementitious formulations. The silicates are only marginally effective in preventing the penetration of water into concrete. They are better used as surface hardeners. High molecular weight silicones, although some are effective water repellents, do not penetrate the surface and, therefore, are subject to degradation by UV light and abrasion. Asphaltic and cementitious formulations, with or without latex additions, are generally ineffective as sealers. Stearates and stearate blends with hydrocarbon resins are partially effective water repellents.

Concrete sealers are being used with increasing frequency in both exterior and interior applications. However, only 5 of 29 plant surveys returned reported using concrete sealers. When designed for use in exterior applications, concrete sealers primarily function to protect concrete from chloride-induced corrosion of the reinforcing steel. If they are used in such a way that they prevent concrete from becoming saturated, they are very helpful in preventing freezing and thawing damage to the concrete. Such applications may be on the top surfaces of elevated parking slabs or exterior walls of buildings. For interior use, concrete sealers are applied to floor slabs to reduce or prevent dusting or absorption of liquid spills, and to make the surface easier to clean.

Five categories of sealers were found to be effective, in the NCHRP 244 study.^(4.42) These are polyurethanes, MMA, certain epoxy formulations, relatively low molecular weight siloxane oligomers, and silanes. Many of the polyurethanes currently sold have the limitation that they are not dependable when exposed to the UV rays of the sun. Epoxies, acrylics, and methacrylates are very effective sealers; however, at the viscosities normally used they do not penetrate into the concrete, but leave a continuous film on the surface. Also, they are not largely vapor permeable.

With the use of any type of sealer, surface preparation including cleanliness and moisture content of the surface is extremely important. With those that do not penetrate but only adhere to the surface, it is also important to provide some mechanical lock, or "tooth" for the material to adhere.

The sealers available today may be classified as either film forming or penetrating. Urethanes and epoxies are two of the most commonly used film forming sealers. Their penetrating ability is slight, with most of the material remaining on the surface to form a barrier. The ability of these products to effectively penetrate concrete is restricted by the relatively large molecular structure of the material. The use of solvents, therefore, will not appreciably enhance penetration capabilities. In addition to sealing out water, these materials also protect against mild chemicals and prevent the absorption of grease, oils, and other liquid spills. Film forming sealers will substantially reduce dusting under light traffic conditions.

When urethane applications are compared to epoxy applications, urethanes typically demonstrate greater abrasion resistance, especially impact, while epoxy sealers have shown superior chemical resistance. Both materials will darken and increase the gloss of the surface. Aliphatic urethanes will exhibit greater resistance and stability of hue in the cured film. Aromatic urethanes will yellow on exposure to UV.

The effectiveness of penetration of silane and siloxane sealers is a result of their very small molecular size, in comparison with the film forming sealers. These penetrating sealers are able to infiltrate and coat the micro-pores and capillary structure of the concrete. Penetrating sealers may achieve penetration depths of as much as 5 mm ($\frac{3}{16}$ in.) or more, depending on the density and finish of the concrete; the orientation of the surface, vertical or horizontal; and the moisture content of the concrete. Unlike film forming sealers, the effectiveness of the penetrating sealers is not dependent on the continuity of an exposed surface film.

The abrasion resistance of concrete containing the penetrating sealer is affected very little. It is slightly better than the concrete itself, but less resistant than concrete treated with film forming sealers. However, the penetrating sealers may still be effective when limited abrasion occurs, since the depth of penetration has not been exceeded.

The three most common penetrating sealers are silanes, siloxane oligomers, and methacrylate. The methyl methacrylate monomer is highly volatile and is rarely used as a sealer, especially with the recent air pollution regulations. Silanes and siloxanes are both derived from the silicone family. When catalyzed by moisture, these silicon materials react with the silica available in concrete to form a hydrophobic siloxane resin film that repels water without loss of vapor transmission properties.

Siloxane oligomers and silanes are not new materials, having been used in paint formulations for decades. However their use as concrete sealers is relatively new. Nevertheless, a great deal of research has been done on this application. Both penetrate the concrete to some degree, the silanes being better in this aspect due to their smaller molecular size. Newer products include 100 percent silanes and water-dispersed silanes, both of which penetrate as well or better than their predecessors, which were usually 20 or 40 percent solutions of silane in alcohol or petroleum distillates. The penetration characteristic is extremely important to the longevity of these materials. When they penetrate and react with the concrete, they are shielded from UV light, which otherwise would degrade them.

Despite being very closely related, the two silicon-based materials have significant performance differences. Silanes are monomers, very small molecules that have not yet reacted with other similar molecules to form polymer chains. Siloxane oligomers are produced when silane molecules are partially reacted to form a larger structure, linking up two or three silane molecules.

Because of their very small molecular size, silanes have the capability to penetrate more deeply. Silanes that have penetrated to a depth of 3 mm ($\frac{1}{8}$ in.) or more are less subject to loss of effectiveness caused by abrasion or weathering than siloxane or film forming sealer. Silanes, due

to their very small molecular size, are relatively volatile. The solids content (reactive component) in a furnished silane should be high enough to compensate for the loss of reactive material through evaporation during application and cure. Siloxanes, because they are less volatile, sometimes offer good hydrophobic performance at lower initial cost than do silanes. However, for concrete surfaces subject to abrasive wear, such as pavements and decks, treatment with a more deeply penetrating silane sealer should provide longer-lasting protection. Silane sealers typically cannot be visually detected on treated surfaces. Siloxane materials may slightly darken the treated surfaces. The silanes and siloxane oligomers allow water vapor to pass so the sealed concrete can periodically dry. Since these penetrating sealers do not block the movement of air or water vapor, carbonation of the concrete by ingress of carbon dioxide gas can still occur. In areas exposed to both moisture and carbonation, a penetrating silane material should be applied followed by a barrier coating such as an acrylic or epoxy coating.

4.8.2 Coatings and Membranes

For the sake of distinguishing between sealers and coatings, coatings are applied in some thickness measured in several hundredths of a millimeter (thousandth of an inch), and generally do not penetrate the concrete. The following should be considered when selecting either a sealer, coating or membrane:

- compatibility with concrete (new and old)
- compatibility with the joint sealant materials
- crack-bridging ability
- effective service life
- effect of outgassing
- abrasion resistance
- weatherability of the materials
- the overall structure and overall condition of the concrete considered for the surface treatment
- surface preparation

Types of coatings include epoxy resins, polyester resins, acrylics, vinyls, polyurethanes, and cementitious materials. Membranes include liquid applied acrylics, urethanes, neoprenes, vinyls, rubberized asphalts, silicones, and preformed membranes such as rubberized asphalts, neoprenes, and butyl rubbers, hypalons, vinyls, and EPDM. The general characteristics of some coating materials are discussed in the following sections.

Acrylics – Acrylics include polymers of acrylic acid, methacrylic acid, esters of those acids, or acrylonitrile. The resins range from soft, sticky semifluid materials to hard solids. They have:

- good resistance to outdoor weathering
- good resistance to chemicals at service temperatures
- good resistance to water and aqueous salt solutions
- good resistance to moderate concentrations of acids and bases
- poor resistance to certain organic solvents

Acrylics have shown to improve substantially the mechanical properties and durability of porous materials such as concrete. Some lower cost acrylic copolymers such as styrene acrylic and vinylacetate acrylic resins have limitations that make them less durable than straight acrylic resins. For example, styrene acrylic resins yellow and are vulnerable to degradation on extended exposure to UV, and vinyl acetate acrylic copolymers are vulnerable to alkaline hydrolysis on moist concrete substrates.

Chlorinated rubbers – Chlorinated rubbers are usually produced by chlorinating isoprene rubber. Normally chlorinated rubber coatings dry by solvent evaporation. The resulting product no longer possesses the resiliency and elasticity of the rubber from which it was produced. General characteristics are:

- excellent resistance to water, salts, acids and alkalies
- good weatherability when they include UV blocking pigments or UV absorbers
- easy application when recoating
- poor resistance to certain solvents
- limited resistance to heat (>65°C [150°F])
- good adhesion to concrete

Epoxies – An epoxy-based polymer consists of an epoxy resin and a polymerization or curing agent. Mixing the epoxy resin with the curing agent converts it into a hard, thermosetting cross-linked polymer. The most commonly used epoxy resins are derived from diphenylolpropane (bisphenol A) and epichlorohydrin. Since such resins are too viscous to penetrate concrete they are usually diluted with organic solvents. Characteristics of epoxy polymers resins are that they have:

- excellent adhesion to stone and concrete
- excellent resistance to water, solvents, and chemicals
- high dependence upon proper proportioning and adequate mixing
- poor resistance to chalking caused by UV
- excellent resistance to abrasion
- good resistance to mildew and bacteria growth
- high dependence on temperature for rate of curing
- poor recoatability after curing
- excellent toughness and durability

Polyesters – Polyester resins include a large group of synthetic resins, almost all produced by reaction of dibasic acids with dihydric alcohols. General characteristics include:

- excellent resistance to many solvents
- excellent resistance to moisture transmittance (impermeable)
- excellent temperature tolerance (up to 204°C [400°F])
- good resistance to impact
- high dependence upon application requirements
- high dependence upon clean, dry, and alkali-neutralized concrete to chemicals
- moderate dependence upon curing temperature (preferably above 10°C [50°F])

As with most polymer coatings, polyesters may form an impervious layer which prevents the passage of moisture through the concrete surface (not breathable). Common liquid resins are converted to solids using a peroxide catalyst such as benzoyl peroxide and an accelerator such as dimethyl aniline. Two conditions can prevent polyester resins from hardening completely.

- Water will inhibit hardening; therefore, a primer that is not sensitive to water should be applied to the concrete first.
- Air inhibits the curing of some polyesters; on these, a final top coat of the resin containing about 1 to 2 percent paraffin needs to be applied. The paraffin rises to the surface and protects the resin from the atmosphere. Without paraffin in the top coat, the resin may remain tacky.

Polyurethane – Polyurethanes are synthetic polymers based on isocyanate resins that may be either thermoplastic or thermosetting. They are used for flexible or rigid coatings or linings. A

polyurethane coating can be either a one-component or two-component system. The one-component urethane is sometimes referred to as urethane polymer. Two-component systems typically consist of an isocyanate resin and most commonly a polyether-based alcohol. Some general characteristics are:

- hard films exhibit excellent toughness and hardness
- excellent mar-resistant surface
- membrane formulations exhibit excellent flexibility
- excellent resistance to chemicals and alkalinity
- excellent adhesion to dry concrete
- excellent resistance to solvents
- excellent weathering properties; however, aromatic-based polyurethanes exhibit poor resistance to yellowing with age
- poor recoatability once cured
- highly dependent on dry surfaces; excessive humidity or excessive moisture in a concrete substrate can cause foaming in urethane membranes
- Multiple coat application requires specific timing between coats.

Vinyls – Vinyl chloride and vinylidene chloride, polymers and copolymers have been used for conservation and consolidation of stone. If used in penetrating formulations or in cementitious coatings for steel-reinforced concrete, liberated chloride ions could adversely affect and cause corrosion of the steel reinforcement. If the vinyl polymers are not carefully applied and if not sufficiently diluted, an impervious layer may be formed that would entrap moisture. Poly vinyl acetate based coatings are subject to degradation by alkaline hydrolysis on moist concrete substrates.

Polyvinyl chloride-based elastomeric membranes are typically plasticized with phthalate ester-based plasticizers. These plasticizers are susceptible to migration and alteration on exposure to moisture and UV. This results in hardening or embrittlement and cracking of membranes particularly at low temperatures. Plasticizer migration can also affect the adhesion of solvent welded seams of PVC membrane sheets.

Silicones – Silicone elastomeric membrane coatings have excellent UV resistance and very high water vapor transmission rates, i.e., greater than four times the vapor transmission rates of competitive membrane coatings. This would appear to be a significant advantage for some applications. However, this is a relatively new type of elastomeric coating, and applications in the United States are still somewhat limited. Some typical physical properties of the various materials used in coatings are shown in Tables 4.7, 4.8, and 4.9.

Liquid applied membranes: Liquid applied membranes include rubberized asphalt, liquid neoprene and polyurethane. They are well suited for both horizontal and vertical surfaces. Some formulations may be sprinkled with sand to support pedestrian traffic but they must be protected from heavy traffic. They are elastic materials applied with either a spray system, roller or squeegee.

Preformed membranes: Preformed membranes are typically 1.25 to 2.4 mm(30 to 60 mils) thick. They can be bonded with adhesive, mechanically fastened or loose laid. Sheet membranes include vulcanized rubbers, such as Butyl and Neoprene, thermoplastics, such as PVC, CPE, HDPE, and Hypalon, and impregnated asphalt composites. Vulcanized rubbers have very low water vapor and gas permeability. They have adhesion problems at the seams. Thermoplastics such as PVC can be heat sealed at the joints to provide a good barrier. High density polyethylene (HDPE) is a good nonbreathable membrane, but breaks down when exposed to UV radiation, so must be protected from the sun. Rubberized asphalt sheeting is typically a composite of asphalt bonded to a layer of polyethylene plastic or reinforced plastic. They are bonded with hot asphalt or by heating with a torch. Water ingress sometimes causes blistering.

4.8.3 Coatings and Sealer Selection

Generally speaking, penetrating sealers are simpler to apply than film forming coatings or membranes. However, long-term effectiveness of penetrating sealers, such as silane or siloxane oligomers is dependent on degree of penetration. When concrete is cold and damp, effective penetration of these types of sealers can be considerably reduced. Unfortunately, the only way to determine the depth of penetration is to core or chisel off a section of the surface region of the concrete and examine it in cross section. This type of an examination should be an integral part of the quality assurance procedures for a penetrating sealer application.

A membrane should only be selected as a protective coating if the user is convinced that the architectural details or configuration of the installation do not allow water access to the substrate below the membrane. Even membranes that are relatively breathable will tend to fail if appreciable quantities of water get below them.

Because the long-term performance of film forming coatings and membranes is strongly affected by the moisture content of the concrete, measurement of the moisture content of the concrete prior to application of the coating is very important. A number of techniques are used for this purpose. Some of the more popular techniques are based on resistance or capacitance meters, calcium carbide generators, hygrometers, moisture emission test kits with desiccants (CaCl_2) that absorb emitted water vapor, and a simple technique where a 2 ft by 2 ft sheet of polyethylene is taped to the concrete for 24 hours to see if moisture condenses below the sheet. The moisture emission test kit has been found to be reliable and relatively inexpensive. It was developed by the Rubber Tile Manufacturers Association. Essentially, a chamber containing a petri dish with calcium chloride desiccant is sealed to the concrete surface for 60 hours. After this 60-hour exposure, the water absorbed by the desiccant is determined by weighing the container of desiccant. Calculations are made in weight of water emitted per area per 24 hours. Moisture emission rates of less than 1.5 to 1.9 kg/10 sq m (3 to 4 lbs/100 sq ft) per 24 hours, are generally recognized as suitable for coating application. Thicker, less permeable membrane coatings are particularly vulnerable to premature failure when moisture emission rates exceed this level.

Evaluation of coating and sealing systems — Potential sealing and coating systems should be evaluated in mock-up before full scale installation. The mock-up should be installed using the actual equipment that will be used for full scale application. After the system has had a chance to cure, it should be evaluated for its effectiveness. For penetrating sealers this evaluation will include determination of depth of penetration as described above, but it should also include a measure of water absorption using mini stand pipe technique, or it could include diking a specimen and cyclic ponding with salt solution.^(4.42)

For film forming coatings, the evaluation should include adhesion measurements by tape adhesion, ASTM D3359 *Adhesion*^(4.13) by direct tension (elcometer), or by direct peel. For membrane coatings, 25.4 mm (1 in.) wide strips are cut into the membrane, and these are pulled by clamping to a force gage and pulling at 180 degrees to the surface.

Adhesion measurements should be repeated after the coating has been sprayed with water and covered for 2 to 4 hours. After this water exposure, the coated surface should be wiped dry, and the adhesion tests should be performed immediately. If adhesion strength has been compromised, alternative systems should be considered, or surface preparation procedures should be improved.

4.9 Concrete Repair Case Studies

Concrete structures in nuclear power plants have performed well with minimal concrete deterioration. Most repairs have been related to cracking problems, construction defects,

seepage, inadequate cover, cosmetic repairs, or corrosion damage. The following are short descriptions of a few of the larger repair projects specific to nuclear power plants provided by Sargent & Lundy Engineers in response to our survey request.

1. Acid Attack on Sump Walls and Floor

Description: The interior wall surfaces in a turbine building sump were attacked by acids that entered the sump by discharge from a water treatment facility. Reinforcing steel was exposed in some areas.

Repair: The areas of exposed steel were covered with an epoxy grout filler. Then, all walls and the floor were lined with a fiberglass reinforced epoxy, approximately 3.2 mm (1/8 in.) thick.

Performance: During application of the lining, problems were encountered in maintaining ideal environmental conditions for coating work. High humidity and damp substrate, plus possible acid residue in the concrete surface, made the application very difficult. The linings on some sections of the sump walls were repaired about 18 months after the original installation. Examinations made at 2½ and 4 years after the initial application revealed many areas where the lining had debonded from the substrate. In spite of the loss of adhesion, the lining appeared to still be providing protection to the concrete.

2. Surface Spalling Near Beam Support Pockets

Description: There is a 200 mm (8-in.) concrete slab, at the turbine deck level supported on steel wide-flange beams, spanning between the two heavy shielding structures below the turbines of a two-unit BWR plant. Support for the steel beams is provided by pockets in the end walls of each shield structure. The walls and top slab of the shield structure are 1.2 m (4 ft) thick.

Anchor bolts, embedded in the concrete below the support pockets, extend through the base plates and the bottom flanges of the steel beams. L-shaped dowels, though not shown on design drawings, were found connecting the edge of the 200 mm (8 in.) slab to the heavy end wall on one shield structure.

Cracking and delamination were observed below several beam support pockets; hollow sounds, indicating delamination were noted in several areas on the end shield wall, just below the slab. After removal of loose concrete, some reinforcing steel was exposed below the support pockets. No exposure of steel was found in the delaminated areas below the floor slab, except for the dowels mentioned above. It appeared that relative separation between the end walls of the respective shield structures had occurred. The restraint forces developed by the anchor bolts and dowels were judged to be the cause of the observed damage.

Repair: The areas where delaminated concrete was removed were repaired with a prepackaged cementitious patch material, with light wire mesh attached to the wall to aid in holding the material in place. The anchor bolts at the beam support locations and the L-shaped dowels between the slab and the shield wall were cut to allow future freedom of movement between the elements. Finally, steel plates were attached to the wall below the support pockets, after placement of the concrete repair material, with expansion anchors.

Performance: Work on this project is just being completed at this time. Therefore, no information on effectiveness of the materials and methods used is available.

3. Delamination of Concrete Cover at Beam Ends

Description: In the cooling sections of two cross flow natural draft cooling towers, the principal structural support is provided by two-level, radial structural bents connected at both levels by inner and outer tangential beams. There are a total of 15 bents in each, thus the connecting beams form the sides a 15-sided polygon.

The connecting beams are made of two sections; a short section of only a few feet and a longer section covering the rest of the distance between bents, each cast integrally with adjacent bents. To accommodate movements, each beam contains a sliding joint. The stub beams are cantilevered from the columns of the bents and configured with ledges to support the free ends of the long sections. Relative movement between bents, along the beam axes is thus possible.

Light precast fill support beams, normal to the beams between successive bents, are supported on the tangential connecting beams. Pockets in the main beams form the support points for the precast elements. Finally, mist eliminator panels are fastened to the interior main beams with anchor bolts into the beams.

During examinations of the towers, cracking and delamination were observed at the sliding joints, at the beam support pockets and in the vicinity of the mist eliminator attachments. Structural capacity was not affected by the damage, but in some cases reinforcing steel was exposed. In order to provide corrosion protection to the exposed steel, repairs to the damaged areas were made.

Repair: The general repair scheme consisted of removing delaminated concrete and replacing it with patch material. Repairs were made to the first tower examined during the winter months; temperatures at the time were above freezing, but generally about 4°C (40°F) or less. In addition, the outage schedule required repairs to be made very quickly. In view of these two constraints, the patch material used was a chemically activated magnesium phosphate based concrete. The companion tower was examined and repaired in essentially the same way as the first. However, because the work was completed in more favorable weather, an ordinary cementitious nonshrink grout was used as the repair material.

Performance: An examination made after approximately 18 months of service showed that several of the magnesium phosphate concrete repairs were no longer adhered to the base concrete.

On the basis of the observed behavior of these repairs, it appears that more thorough preparation of the area to be patched, including elimination of sloping surfaces, might result in longer lasting repairs. Repairs should be scheduled for periods of moderate temperature, if possible.

4. Spalled Concrete Due to Restraint of Thermal Expansion

Description: A large heater is supported at one end by concrete pedestals and the other by rollers. The pedestals are considered "fixed" and thermal expansion is intended to be accommodated by the rollers. Examination showed that considerable spalling of the pedestals had taken place. Structural damage appeared to be minimal. The cause was assumed to be lack of freedom of the rollers to function as designed.

Repair: Steps were taken to ensure that the rollers would allow the necessary thermal movement. Loose concrete was removed from the damaged areas of the pedestals and replaced with a prepackaged cementitious repair mix containing pea gravel aggregate. In some of the pedestals, a complete jacket of the repair material was placed because of the nature of the spalling. A few areas, that were small and were not suited for use of the pea gravel aggregate material were patched with nonshrink grout.

Performance: These repairs were made approximately seven years ago. Since then, additional spalling and some delamination has occurred. In our opinion, the damage is due to excessive stresses caused by restraint forces rather than failure of the repairs themselves.

5. Fill in Cavities Left by Removal of Temporary Supports

Description: During construction of a containment structure, short angles were welded to the liner of the dome for various attachments during placement of the dome concrete. The area around the members was boxed out and after completion of the concrete work the angles were cut off the cavities filled with a material of unknown composition. Subsequently, the filler material began to pop out and a replacement program was initiated.

Repair: The original materials was cleaned out and the edges cut approximately normal and parallel to the dome surface. A cementitious patch material with pea gravel aggregate was used to refill the cavities.

Performance: The repairs have been in place for five years and are performing satisfactorily.

6. Acid Leakage on Floor Supporting Tank

Description: The floor slab supporting a tank containing boric acid had a number of shrinkage cracks. Leakage from the tank was observed dripping from the underside of the slab. To determine the extent of any internal damage to the floor slab, concrete was chipped away from the top surface to the depth of the top reinforcing steel in two or three places. No corrosion of the steel was observed nor was there any damage to the concrete itself.

Repair: The chipped areas were patched with a material of unknown composition. The entire area of slab affected by the leakage was coated with a material, also of unknown composition. While the exact coating material used is not known, it may be assumed that it was chosen to be compatible with concrete and to be acid resistant.

Performance: This repair was made approximately 10 years ago. No further problems have been reported.

7. Miscellaneous Concrete Cracks

Description: The vast majority of the cracks have been due to shrinkage and are located in dry areas and have no structural significance. However, in exterior below grade walls and in basement floors, groundwater infiltration has been observed in several cases. Most such leaks result in minor nuisances and do not affect plant operation. But as long as leakage continues, there is some possibility that deterioration of the concrete or reinforcing steel in the affected element may occur. For that reason, it is generally desirable to stop the leakage if possible.

Repair: The crack repair method that is usually recommended is the injection with polyurethane sealers.

Performance: Overall, it appears that the polyurethane sealers have performed quite well. Total elimination of groundwater infiltration is extremely difficult. It has been observed that a crack will begin leaking, in a region previously dry, following sealing of a portion of the same crack, or even after sealing of other cracks in a given area. Further leakage is normally not observed in those cracks where the sealant has been injected properly.

8. Surface Blemishes, Core Holes and Other Penetrations

Description: Some honeycomb can be expected on any project. Generally, it does not affect the structural capacity of the element, but if unrepaired may lead to deterioration. In any case, it is unsightly. Core holes are required to provide passage of pipes, conduits or other carriers through concrete elements. Abandoned anchor bolt holes, incorrectly located holes and similar situations usually require filling.

Repair: Recommendations include filling such voids with cementitious grouts, either prepackaged or field proportioned with cement, aggregate and water. Generally, we do not recommend polymer based materials for such applications.

Performance: In our experience, the cementitious based patches have not resulted in any problems.

Overall, the repairs to nuclear power plants have performed well. The problems have been minor and the repairs have not been extensive. The durability of repairs has not been a concern and repair performance data are not routinely collected. Most repairs to nuclear power plants have utilized common repair techniques. There appears to be a need for dissemination of information on state-of-the-art repair methods as well as the importance of condition surveys and monitoring of structures and repairs.

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**Table 4.1 – Average Bond Strength for Curing and Test Age Indicated, MPa (psi)
(average of 4 or 5 measurements)**

Bonding agent	Concrete surface preparation	Continuous underwater cure				10 days in water then air dry	28 days in water then air dry
		4 d	7 d	28 d	98 d	98 d	98 d
None	Wet brushed	2.12 (307)	3.16 (458)	2.74 (398)	2.06 (299)	0.05 (8)	**
None	Sandblasted	2.07 (300)	2.12 (308)	2.50 (362)	2.75 (399)	1.38 (200)	2.68 (389)
Latex - neat, tacky	Wet brushed	0.09 (13)	0.44 (64)	0.36 (53)	**	**	**
Latex - dilute, tacky	Wet brushed	0.14 (21)	1.38 (200)	1.11 (161)	1.11 (161)	**	**
Latex grout - lean	Wet brushed	1.43 (207)	1.36 (198)	2.37 (344)	2.71 (394)	1.88 (273)	3.26 (473)

** Not tested – Coring operation dislodged all topping sections.

Table 4.2 — Typical Properties of Rapid Set Patching Materials by Generic Family (SI)^(4,17)

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

E = Modulus of elasticity in compression

α = Thermal coefficient of expansion

*High exotherm

Table 4.2(a) — Typical Properties of Rapid Set Patching Materials by Generic Family^(4.17)

MATERIAL	Approx. working time (min.)	Approx. time to traffic (min.)	Compressive strength (psi)		Abrasion loss (g)	Flexural strength (psi)	Bond strength (psi)		E (10 ⁶ psi)	α (10 ⁻⁷ /°F)	Linear shrinkage (%)
			3 hr	24 hr			Dry PCC	Wet PCC			
Inorganic											
PCC w/accelerator	120+	300+	—	3100	22	425	300	350	2.0 - 6.0	4 - 11	0.02 - 0.08
Magnesium phosphate	15	60	4000	6000	25	800	475	160	3.0 - 5.0	6 - 8	0.10 - 0.15
High alumina cement	15	60	5000	6500	20	600	400	375	3.0 - 5.0	4 - 11	0.02 - 0.08
Gypsum based	20	60	3500	6000	18	400	300	375	2.0 - 4.0	4 - 11	0.02 - 0.08
Organic											
Epoxy	30 - 60	90 - 200	2000	8000 - 12000	0 - 1	2400 - 3000	Failed in PCC	Failed in PCC	0.1 - 6.0	15 - 40	0.02 - 0.2
Methacrylate	20 - 40	60 - 120	7000	8000 - 9500	10	2000 - 3000	Failed in PCC	Failed in PCC	1.0 - 3.5	7 - 15	1.5 - 5.0
Polyester-styrene	15 - 40	60 - 120	2000	3000 - 5000	3	1500 - 2000	Failed in PCC	Failed in PCC	1.0 - 5.0	18 - 30	0.3 - 3.0
Urethane	5 - 45	30 - 90	500 - 2000	500 - 5000	3	1500 - 4000	Failed in PCC	500	0.1 - 4.0	30 - 70	0.02 - 0.2

E = Modulus of elasticity in compression

α = Thermal coefficient of expansion

*High exotherm

Table 4.3 – Comparison of Advantages of Dry-Mix and Wet-Mix Processes*

Dry-Mix Process	Wet-Mix Process
1. Instantaneous control over mixing water and consistency of mixture at the nozzle to meet variable field conditions.	1. Mixing water is controlled and can be accurately measured.
2. Capable of longer hose lengths.	2. Better assurance that the mixing water is thoroughly mixed with other ingredients, resulting in less rebound and waste.
3. Compatible with all ordinary admixtures including appropriate accelerators, except those for air entrainment.	3. Less cement loss and dusting accompanies the gunning operation.
4. Better suited for placing mixtures containing lightweight porous aggregates.	4. Better adapted to ready-mixed concrete procedures.
5. Maintenance costs tend to be lower of shot concrete or equipment.	5. Certain wet-mix equipment permits mixing within the delivery equipment.
	6. Compatible with all ordinary admixtures. Special dispensers for certain accelerators are under development.

*After ACI Committee 506^(4.25)

Table 4.4 — Tolerable Crack Widths

Author	Environmental factors	Permissible width, mm (in.)
Rengers	Dangerous crack width	1.0 to 2.0 (0.04 to 0.08)
	Crack width allowing corrosion within ½ year saline environment	0.3 (0.012)
Abeled	Structures not exposed to chemical influences	0.3 to 0.4 (0.012 to 0.016)
Tremper	Found no direct relation between crack width and corrosion	
Boscard	Structures exposed to a marine environment	0.4 (0.016)
de Bruyn	Found no direct relation between crack width and corrosion	
Engel and Leeuwen	Unprotected structures (external)	0.2 (0.008)
	Protected structures (internal)	0.3 (0.012)
Voellmy	Safe crack width	up to 0.2 (0.008)
	Crack allowing slight corrosion	0.2 to 0.5 (0.008 to 0.020)
	Dangerous crack width	over 0.5 (0.020)
Bertero	Indoor structures	0.25 to 0.35 (0.010 to 0.014)
	Normal outdoor exposure	0.15 to 0.25 (0.006 to 0.010)
	Exposure to seawater	0.025 to 0.15 (0.001 to 0.006)
Haas	Protected structures (interior)	0.3 (0.012)
	Exposed structures (exterior)	0.2 (0.008)
Brice	Fairly harmless crack width	0.1 (0.004)
	Harmful crack width	0.2 (0.008)
	Very harmful crack width	0.3 (0.012)
Salinger	For all structures under normal conditions	0.2 (0.008)
	Structures exposed to humidity or to harmful chemical influences	0.1 (0.004)
Wastlund	Structures subjected to dead load plus half the live load for which they are designed	0.4 (0.016)
	Structures subject to dead load only	0.3 (0.012)
Efsen	Exterior (outdoor) structures exposed to attack by seawater and fumes	0.05 to 0.25 (0.002 to 0.010)
	Exterior (outdoor) structures under normal conditions	0.15 to 0.25 (0.006 to 0.010)
	Interior (indoor) structures	0.25 to 0.35 (0.010 to 0.014)
Rusch	Ordinary structures	0.3 (0.012)
	Structures subjected to the action of fumes and sea environment	0.2 (0.008)

Table 4.5 – Causes of Cracking

Cause	Type of crack		Comment
	Active	Dormant	
Accidental loading		X	
Design error (inadequate reinforcement)	X		Limit loading according to current capacity and repair, or redesign and repair as indicated by the redesign
Temperature stresses (excessive expansion due to elevated temperature and inadequate expansion joints)	X		It may be desirable to redesign to include adequate expansion joints
Corrosion of reinforcing steel	X		Simple crack repair methods should not be used as the steel will continue to corrode and crack the concrete
Foundation settlement	X	X	Measurements must be made to determine if the foundation is still settling
Alkali-aggregate reaction	X		Concrete will continue to deteriorate as long as moisture is present. Crack repair methods will be ineffective
Poor construction procedures (inadequate curing, formwork, etc.)		X	
Design faults <ul style="list-style-type: none"> • use of exposed rigidly connected material to concrete which has a much different modulus of expansion • stress concentrations • faulty joint systems 	X		
<p>NOTE: This listing is intended to serve as a general guide only. It should be recognized that there will be exceptions to all of the items listed.</p>			

Source: Corps of Engineers, Washington, DC

Table 4.6 — Properties of an Epoxy and Polyester Chemical Grout^(4.41)

Property	Epoxy resin system	Polyester resin system*
Tensile strength, MPa (psi)	35.2 (5100)	55.8 (8100)
Tensile modulus, MPa (psi)	1.17 x 10 ³ (0.17 x 10 ⁶)	—
Elongation, %	15.8	2.6
Flexural strength, MPa (psi)	60.0 (8700)	84.8 (12,300)
Flexural modulus, MPa (psi)	1.45 x 10 ³ (0.21 x 10 ⁶)	4.2 x 10 ³ (0.62 x 10 ⁶)
Deflection, mm (in.)	13.5 (0.53)	—
Compressive strength, MPa (psi)	52 (7500)	—
Compressive modulus, MPa (psi)	1.3 x 10 ³ (0.19 x 10 ⁶)	—
Deflection at yield, mm (in.)	4.6 (0.18)	—
Izod impact strength, J/m (ft-lb/in. of notch)	55 (0.82)	—
Hardness at 25°C (77°F) (Shore D)	79	38-40**
Water absorption, %	0.21	0.15
Shrinkage, % (volume)	0.001***	6.0

* Typical values

** BARCOL

*** Effective (after gel formation)

Table 4.7 — Water Absorption and Vapor Transmission of Selected Coating Materials

Generic type	No.	7-day water absorption, %			7-day water vapor transmission, %			
		Average	Mean	Range	Average	Mean	Range	Application rate range sq ft/gal
Control	1	4.79	-	-	3.53	-	-	-
Acrylics	9	0.82	1.84	0.50 - 3.08	1.56	1.47	0.83 - 2.81	75 - 110
Cementitious	5	2.75	2.94	0.49 - 4.56	1.80	2.18	1.02 - 2.28	-
Epoxy	7	0.31	0.16	0.06 - 0.37	0.20	0.21	0.06 - 0.46	60 - 125
Hypalon	2	0.36	-	0.29 - 0.42	0.23	-	0.18 - 0.37	75 - 95
Neoprene	1	1.06	-	-	0.37	-	-	70
Polyurethane	16	0.55	0.20	0.04 - 3.79	0.60	0.54	0.15 - 1.54	70 - 175
Polyester	1	0.12	-	-	0.17	-	-	90
Silicone	1	1.76	-	-	2.76	-	-	80

Table 4.8 — Accelerated Weathering*

Generic type	Appearance after 1600-hr testing
Acrylic	Slight yellowing of coating No effect
Polyurethane	Slight yellowing of coating Slight discoloration Some chalking and loss of gloss
Silicone	No effect

* ASTM G 53^(4.13)

Table 4.9 — Bond Strength of Concrete to Coating (Applied to Dry Surface)

Generic type	Average bond, MPa (psi)	Generic type	Average bond, MPa (psi)
Acrylic (6)	1.3 (190)	Hypalon (2)	1.6 (240)
Epoxy (7)	2.3 (340)*	Neoprene (1)	0.8 (120)
Polyurethane (13)	2.1 (306)*	Silicone (1)	0.7 (110)

* All breaks exceeding 270 psi were concrete breaks

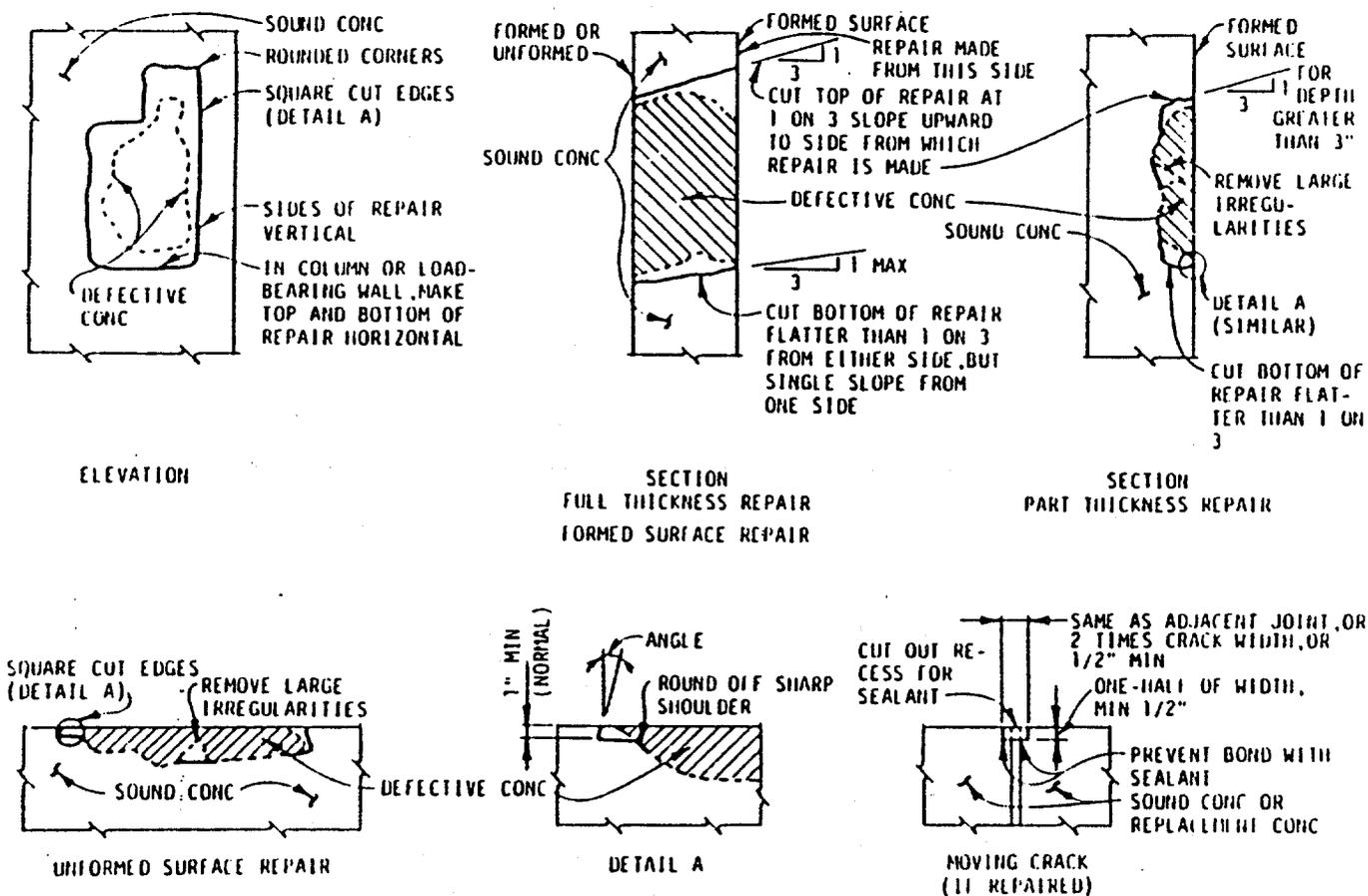


Fig. 4.1 — Repair practice utilized by TVA for concrete replacement in vertical walls or columns [Ref.— General Engineering Specification G-37, Requirements For Repair of Concrete During Construction, Modification and Maintenance, Tennessee Valley Authority, Rev. 5, 1990]

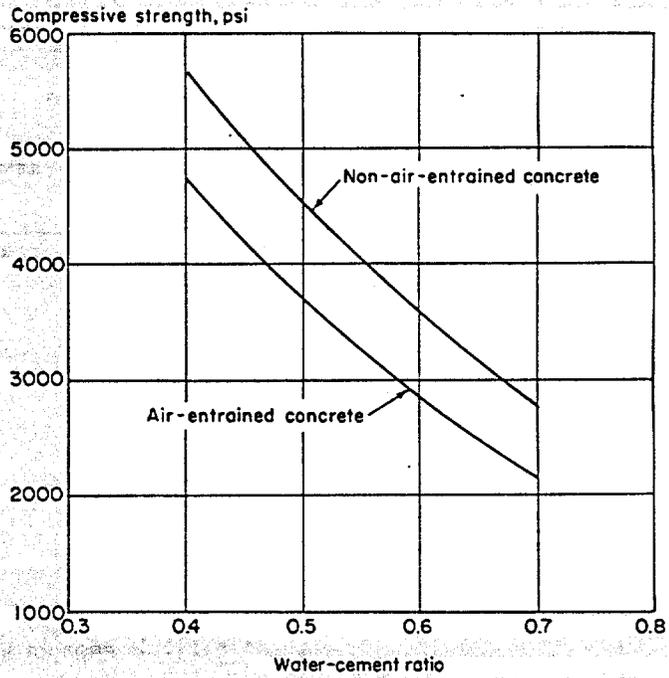


Fig. 4.2 — Typical trial mixture on field data strength curves (PCA Fig. 7-1(4.14))

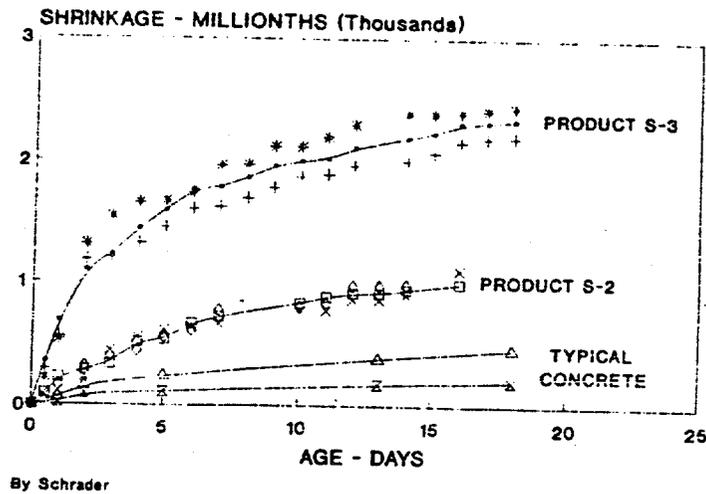
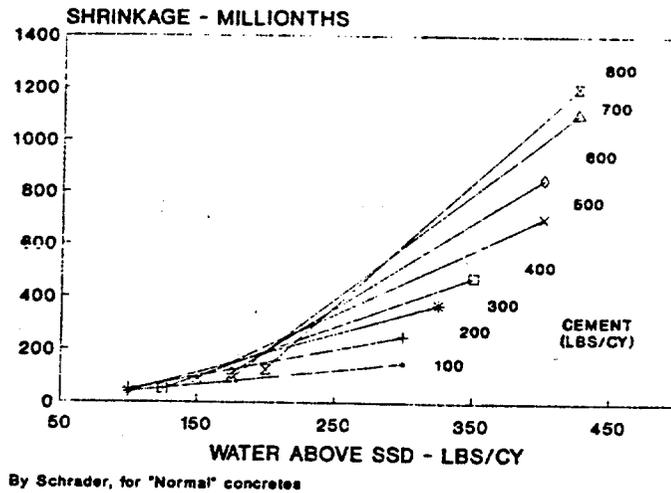


Fig. 4.3 — Typical effect of water and cement on shrinkage (top graph) and shrinkage of two commercial repair materials compared to concrete (bottom graph) (Schrader^(4.15))

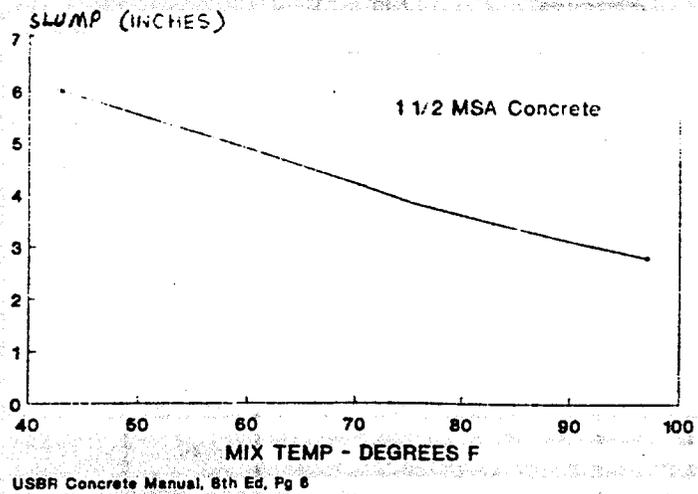


Fig. 4.4 — Effect of mix temperature on slump (Schrader^(4.15))

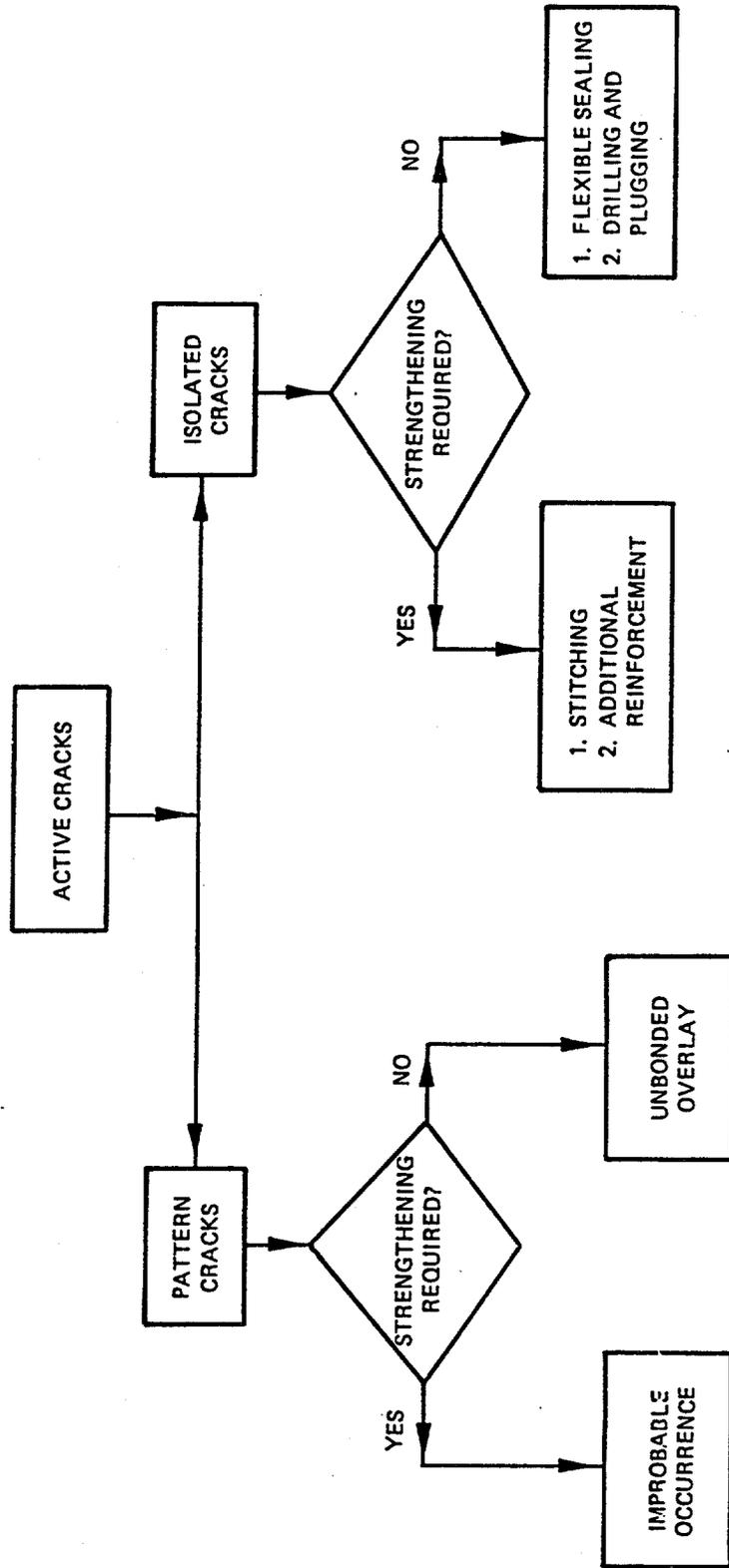


Fig. 4.5 — Selection of repair method for active cracks^(4.5)

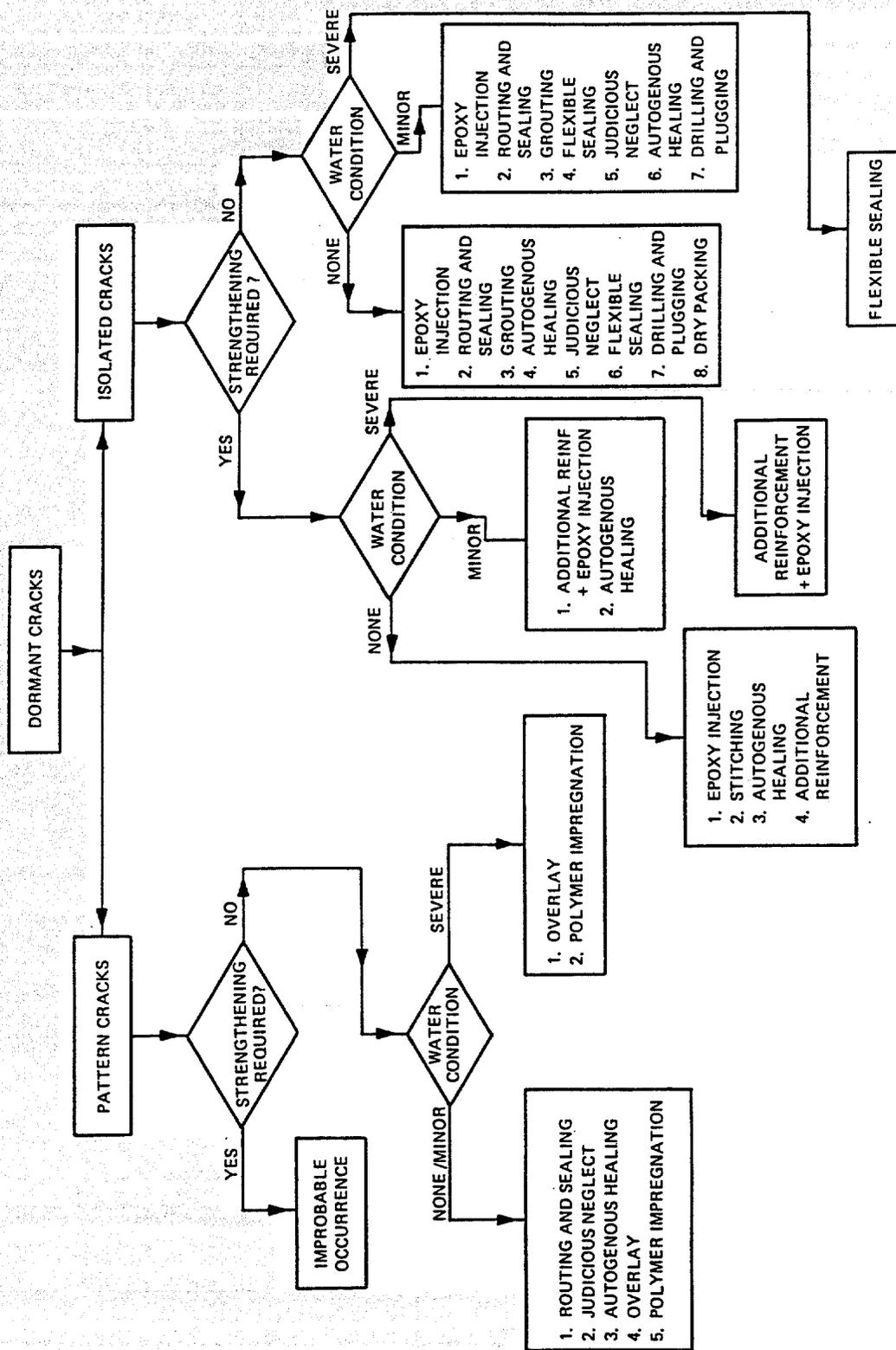


Fig. 4.6 — Selection of repair method for dormant cracks^(4.5)

5. EVALUATION TECHNIQUES

The following discussion provides an overview of methods available to evaluate concrete members or structures. It includes nondestructive as well as semi-destructive methods. Selection of the most appropriate and effective method to assess the concrete requires good judgment, based on the needed information and the cost of evaluation. The procedures for doing a complete condition survey of a structure or element were discussed in Chapter 2, Subsection 2.7 of this report.

5.1 Overview of Nondestructive Techniques

Nondestructive techniques that have been used on concrete structures include acoustic, ultrasonic, electrical resistivity, electromagnetic, vibration, strain gage, dye-penetrant, radar, magnetic particle, and radiography techniques. The use of any of these methods depends largely upon the sensitivity desired, adaptability to field use, cost of the instrumentation, and survey requirements. In general, most methods detect defects by the observation of changes in the response of the interrogating medium. Interrogating media in nondestructive tests include electromagnetic wave, stress wave, electricity, magnetism, and charged particles.

In nondestructive testing using electromagnetic waves, the practical range of wavelengths is from about 300 mm (12 in.) [microwaves] to less than 10^{-14} mm (4.0×10^{-17} in.) [gamma rays]. Microwaves are mostly used to determine the moisture content of porous materials like wood, plaster, clay products and concrete. X-rays and gamma rays make up the shortest wavelength and have the greatest penetrating power. Thousands of X-ray radiographs taken from all sides of an object can be used to examine various properties of engineering materials. This new technique is called a computed tomography. Lasers with wavelengths in the visible and infrared ranges are mostly used in precision measuring devices in the construction engineering field.

Stress waves propagate through solid materials with a frequency range from hypersonic (10^{10} Hz) to sub-audible (0.01 Hz). Flaws or discontinuities in a material disturb the passage of stress waves and therefore can be detected. There are two measurement methods of sonic testing techniques: resonant and ultrasonic pulse. The resonant methods involve the measurement of the natural frequencies of vibration of a component while ultrasonic methods use high-frequency sound wave technology (above the hearing range, 16 kHz).

Other interrogating media include electricity, magnetism, and charged particles. Electrical resistance measurements are most often associated with evaluation of moisture content, as in concrete and timber. Magnetic methods are used primarily for examination of steel structures, but are used extensively in concrete construction to locate embedded steel items.

Selection of a testing method depends on the property to be evaluated, the structural material and cost. The most common forms of defects and deterioration that occur in concrete components include cracking, scaling of the surface, corrosion of reinforcement or prestressing tendons, honeycombing or air pockets, popouts (usually related to frost-susceptible aggregates), surface deposits, chemical attack, wear, and erosion. On the other hand, in steel structures the most common forms of deterioration include corrosion (uniform, pitting, galvanic, or dissimilar metal) and cracking (fatigue or embrittlement). The presence of defects and extent of deterioration is usually determined visually, with the aid of some of the available testing procedures.

Visual inspection often provides a good overview of the condition of the structure. In particular, it allows the identification of cracks, corrosion, or surface defects. Special equipment such as borescope or periscope can be used for visual inspection of inaccessible areas. However, visual inspection is inefficient in the case of many serious defects such as corrosion of reinforcing and prestressing steel, pitting corrosion, fatigue damage accumulation, structural cracking, and others.

Hardness and strength of concrete can be measured by rebound and penetration methods (Swiss hammer and Windsor probe). Other methods using the elastic properties of concrete (modulus of elasticity) include sonic and ultrasonic pulse-velocity methods. These methods are based on the measurement of the travel time of waves generated by electronically-induced mechanical pulses.

Magnetic methods are mostly used for concrete structures to determine the position of reinforcement (cover meters or pachometers). Standard electrical methods are used to determine moisture content and corrosion rate. Chemical test methods are routinely used to evaluate the depth of carbonation and chloride ion content. Concrete delaminations can be located using infrared thermography, by using the thermal conductivity of the material in measuring differences in the surface temperature. Radar has been used to detect deterioration in concrete bridge decks. Two other techniques, radiography and radiometry, are available. In radiography, the radiation is detected by a photographic emulsion (film), and in radiometry, variations in the gamma intensity are detected by radiation detectors such as Geiger or scintillation counters.

Some defects on the surface of the structure, primarily steel structures, can be detected by dye-penetrant methods, magnetic particles (particles align themselves in the magnetic field following the outline of the crack) or eddy currents (defect is detected by perturbation in an electrical field).

Several of the more promising and versatile techniques reviewed above are described in more detail below.

5.1.1 Visual Inspection

Visual inspection is one of the most widely used methods for evaluating the surface condition of concrete. It can be performed with or without the use of optical aids such as low-power magnifiers. Defects such as missing components, cracks, erosion, corrosion, misaligned joints, etc., can often be detected with the unaided eye. With the use of 5 to 10 power magnifiers, even surface flaws as small as a few microns wide can be detected.

The most obvious advantage of using the visual method to evaluate surface flaws in concrete is that it is inexpensive and requires little or no special equipment. It sometimes can be used to find defects that otherwise cannot be found. However, the visual inspection method can be time consuming and is completely dependent upon the visual acuity and the experience and training of the inspector. Obviously, no subsurface information can be obtained using this technique and even for surface conditions, a correlation already must have been established between surface conditions and serviceability of the concrete. With the use of fiberscopes and borescopes in small diameter access holes, internal areas of the concrete also can be visually examined.

5.1.2 Rebound Test

The rebound test hammer can be used to estimate the compressive strength of concrete by measuring the height of rebound of a hardened steel hammer dropped on concrete. The rebound hammer consists of a spring-actuated hammer and plunger mounted within a tubular housing. A schematic of the workings of the hammer is shown in Fig. 5.1. To operate the hammer, the impact plunger is placed against the surface of the concrete specimen and the hammer housing is pressed down until the plunger almost disappears into the housing. The hammer will then release. A built-in scale measures the rebound value, in percent of the forward movement of the hammer mass, after it strikes the concrete surface. The amount of rebound is affected by many factors such as the composition of the concrete, aggregate properties, surface texture and hardness, moisture content, and mass of the concrete specimen. User's charts (provided with the equipment) are based on empirical correlations that have been established between strength properties, hardness, and the rebound number. The rebound hammer test results are very sensitive to the condition of the top several

inches of concrete. The surface condition is an important factor; values for surfaces cast against different form materials should not be compared. Surface grinding of the concrete should only be done when necessary such as if the surface is soft or textured. More reproducible values can be obtained from formed surfaces than finished surfaces. Therefore, structural slabs should be tested from the underside. The surface of the concrete should be in a uniform moisture condition since the moisture content of the concrete will affect the rebound hammer results. Frozen concrete should not be tested. To increase the accuracy beyond about ± 30 percent, a chart must be developed for the materials in the concrete being evaluated. ASTM Standard Test Method C805 *Test for Rebound Number of Hardened Concrete*^(5.1) describes this test method.

5.1.3 The R-Meter

The R-meter is a solid state magnetic detector that operates on the principle of change in magnetic flux. The meter is shown in Fig. 5.2. The unit supplies a constant current to the drive windings of the probe and a reference inductor, both being connected in a bridge configuration to the meter. With no metallic material present, the bridge output is null. If magnetic material is encountered in the field of the probe, an unbalance of the bridge occurs due to a shift in the probe inductance. The unit can be calibrated in the laboratory to evaluate bar cover and size of reinforcing. If one of the parameters is known, that is, the size of the bar or the cover over the bar, the other parameter is read directly from the meter. The unit has an accuracy of ± 1 bar size ($\frac{1}{8}$ in.).

5.1.4 Through-Transmission Method

The through-transmission method involves measurement of the transit time and an assessment of the resulting signals from a pulsed stress wave through a concrete member as represented in Fig. 5.3. Detected changes in arrival time, amplitude and characteristics of the propagated waves can indicate corresponding changes in the internal makeup of concrete. The testing technique is based on procedures outlined in ASTM C597, *Standard Test Method for Pulse Velocity Through Concrete*.^(5.1)

Ultrasonic testing of concrete applies fundamental theories of vibrating elastic mediums to measure changes in material density and elastic modulus. The measurement of pulse velocity of a concrete member has been used to determine the following:

- Uniformity and relative quality of concrete
- Strength of concrete
- Regions of internal cracking
- Extent of internal voids
- Severity of deterioration

The testing equipment consists of a transmitting and a receiving transducer and a time-measuring meter. The meter supplies a train of pulses to a piezoelectric transmitter that is in acoustical contact with the concrete surface. A receiving transducer, at a known distance from the transmitter senses these pulses and converts them into electrical signals. The meter, with an accurate time delay circuitry, calculates the transit time and displays it digitally. A digital storage oscilloscope can be used with the meter to view and store the received signals. The pulse velocity, or V-meter, was developed in the 1960s.^(5.2)

Current equipment is lightweight and battery-operated. Small differences in pulse velocity typically means a relatively large difference in the quality of the concrete. A minimum path length of 150 mm (6 in.) is recommended. The least dimension perpendicular to the pulse should not be less than the wave length of the pulse or 100 mm (4 in.) for a 54 kHz frequency transducer. The transducers must be securely coupled to the concrete surface. Rough areas should be ground and a coupling material such as thin grease or seal should be used. The moisture content of the concrete

has a small effect on the pulse velocity and saturated concrete may read up to 2 percent higher than dry concrete.

Pulse velocity is most commonly used to determine the uniformity of the concrete, the presence of defects such as voids or honeycombed concrete, the extent of fire damage to the concrete, or to indicate the strength of the concrete. Correlation data between the pulse velocity and the concrete strength must be developed before strength properties can be determined in-situ.

Pulse velocity is best utilized to identify voids within concrete members that have access to both faces. The transducers are placed on each side of the member and readings are taken in many locations. A void lying between the transducers prevents the direct passage of the pulse since there is negligible transmission across the air void. The pulse must diffract around the edge of the void, thereby increasing the transit time. By comparing the data, areas having higher transit times can be associated with voids. Pulse velocity can also be used to determine the effectiveness of repairs of the voids because once the voids are filled with resin or grout, the transit time will decrease.

The effect of embedded reinforcing steel can be significant if the bars are aligned in the direction of the pulse and if the bar diameter is greater than 10 mm ($\frac{3}{8}$ in.). The wave velocity along a steel bar is much higher than through the concrete; therefore, accurate readings cannot be achieved. Equations have been developed to account for parallel reinforcing steel but the location of the steel must be accurately known. It is best to avoid areas of high steel content parallel to the measurement. Transverse bars do not seem to affect the readings.

Fire damage can be estimated by using the transducers in an indirect mode, with both transducers placed on the exposed surface. One transducer is fixed and the other is moved in incremental distances away from the fixed transducer. The changes in transit time can be related to the depth of deterioration. The evaluation of fire damage is discussed in detail in the Fire Damage section in Chapter 3. Verification of the damaged areas should be performed by destructive methods such as coring.

5.1.5 Impact-Echo Method

The impact-echo is a nondestructive method for concrete evaluation that involves introducing mechanical energy, in the form of a short pulse, into a structure. A transducer mounted on the surface of a structure receives the reflected input waves or echoes from the discontinuities (flaws) within the material as shown in Fig. 5.4. By determining a propagation velocity, reflected waves can be analyzed with a Fast Fourier Transform (FFT) analyzer to determine internal characteristics of the concrete.

The impact-echo method can detect the size and location of significant flaws in concrete such as honeycombing, debonding, delaminations, cracks (planar or otherwise), mud pockets and cold joints. In addition, the method can measure the thickness of concrete pavements, slabs, beams or columns and can verify grout continuity in post-tensioning ducts.

The major advantage of this method is that it requires access from one side only. The method can also identify critical areas where cores should be taken rather than drilling and removing cores at random locations.

5.1.6 Impulse Response Method

Impulse response is a testing technique in which the force magnitude of an impact is compared to the vibration response of the structure being impacted, as depicted in Fig. 5.5. The

technique is very helpful in understanding the physical interactions taking place in a complex system. An important basic assumption is that the structural system is linear at all times; that is, the motion response at any point is always proportional to the magnitude of the applied force. This assumption can easily be satisfied for the small magnitudes of force and displacements in concrete or masonry structures.

There are two general types of responses that are considered. One is "driving point response," which is the response at the point where the excitation force acts. The other is "transfer response," which is the response motion at one point when the force is applied at another point (some distance away). In the Impulse Response method, the "driving point response" is used. The ratio measurement (transfer function) is obtained by measuring the impact force and the corresponding response function, then dividing the Fourier transform of the response by the transform of the input force. The ratio is a complex number because of damping in the system.

The response of a system can be measured in units of velocity, acceleration or displacement. The respective ratios (transfer functions) of response to input force are called mobility, inertance and compliance. The reciprocals of these ratios are designated as impedance, apparent mass and apparent stiffness, respectively. All of the six ratios are related to each other, and the response spectrum shows peaks or valleys at the structure resonant frequencies. By knowing one of six response functions, other ratios can be calculated, if necessary. Significant differences in the measurements at different locations in a given uniform structure can be indicative of internal variations or flaws.

5.1.7 Spectral Analysis of Surface Waves (SASW) Method

The SASW method evolved from the steady-state Rayleigh Wave technique that was used in the 1950s and 1960s for measuring the elastic properties of pavements. The propagation of surface waves in layered systems varies with frequency or wavelength. By measuring the propagation velocities of surface waves over a wide range of wavelengths, the elastic stiffness and layer thicknesses in the system can be determined. The SASW method is shown schematically in Fig. 5.6.

The propagation velocities of surface waves at various frequencies can be measured with accelerometers (or V-meter transducers) and a Fast Fourier Transform (FFT) analyzer. Two or more accelerometers are mounted vertically on the concrete slab. The spacing between the accelerometers is varied according to the wavelengths being measured. A multichannel FFT analyzer is used to record and analyze the response of the accelerometers when the concrete slab is impacted with a suitable instrument

The phase of the cross spectrum represents the phase difference of the motion at the two accelerometers. The surface wave velocity and wavelength can be measured by using the following expressions:

$$L_R = \frac{360^\circ}{(\text{phase angle})} \Delta X$$

$$V_R = f L_R$$

Where

L_R = Wave length

ΔX = Spacing between the accelerometers

f = frequency

By analyzing the phase information obtained from the cross spectrum of the accelerometer responses, the condition of the subgrade support is determined.

5.1.8 Radar Profiling

In radar profiling, echoes from a pulsed electromagnetic wave are received by an antenna as shown in Fig. 5.7. The penetration and resolution of the signal are a function of the frequency of the electromagnetic pulse. A high frequency signal provides high resolution, but has shallow penetration. Lower frequency signals have greater penetration. Two or possibly three different radar antennas, ranging from a few hundred to a thousand or more megahertz, may be utilized. The data recording rate is chosen to yield high resolution continuous profiles over multiple traverses. Field evaluation and preliminary interpretation of data are generally made from a group of parallel profiles. Radar profilings detect the presence of delaminated conditions and may indicate deterioration caused by cracking. Chemical deterioration of concrete and consequent changes in the dielectric constant may be detected if radar is used in conjunction with other methods, particularly electrical resistivity.

5.1.9 Acoustic Emissions

Acoustic emission evaluation consists of monitoring and evaluating high frequency acoustic signals (stress waves or pulse) that are produced naturally by the test materials themselves, when placed under stress. These acoustic signals are related to the internal physical changes taking place within the specimen being tested. Detection of the stress waves is accomplished by affixing sensors to the specimen, amplifying the acoustic emissions, and recording them on a tape recorder or processing them through a computer for recording and analysis.

Application of the technique is based on the Kaiser Effect—the principle that states that under a load, almost all materials produce a significant level of small-amplitude elastic stress waves above the base level of stress waves produced without a test load. A high emission rate above the base level of emission at operating load, design load, or test load level indicates that a flaw is growing or inelastic action is occurring.

Acoustic emissions are generally high frequency signals (>200 Hz) emitted by local imperfections such as jointing, separations, cracking in concrete or steel members, or movement of soil particles in the sub-base. These are generated by intermittent stress variations, caused by a number of external sources such as thermal effects and variations in water level on dams, traffic and other induced loads on concrete.

This technique can also be used to detect rebar/concrete bond deterioration, by following the generation of small cracks in the vicinity of the rebar. The results also suggest that the bond deterioration associated with corrosion-induced cracking should also be detectable.

5.1.10 Magnetic Field Disturbance

The basic function of magnetic field disturbance (MD) equipment is to provide maps of the magnetic field across the concrete surface. An electromagnet on the inspection cart provides a magnetic field that induces magnetization in certain elements in its vicinity, such as steel rebars, stands, stirrups, etc. An array of magnetic field sensors distributed across the bottom and sides of the beam measures the field produced by magnetized elements within the structure. If a flaw is present in a magnetized element, it will produce a disturbance of the normal magnetic field pattern associated with the unflawed element. Thus, the basic idea behind the MD system is to search the surface of the structure for magnetic field anomalies that indicate the presence of flaws in reinforcing steel within the structure.

5.1.11 Infrared Thermography

Thermography is the science of evaluating materials by measurement of their surface temperature. Infrared thermography is a particular implementation of thermography in which an infrared camera is used as the means for making surface temperature measurements. An infrared camera detects the infrared radiation emitted by a material surface. With appropriate calibration for material properties and background radiation, this radiation can be converted into a direct measurement of temperature of the material surface.

Infrared thermography can detect surface delaminations in concrete elements. The basis for this capability is the fact that the delamination introduces a thermal discontinuity. Under the influence of solar radiation or heat, the surface areas above a delamination will become hotter during the day, and cooler at night. This "thermal anomaly" can be directly seen on an infrared image. This capability requires the presence of solar input or heat (and therefore will not work on cloudy days) and relatively dry surface conditions, since wet surfaces will result in a uniform surface temperature. The presence of moisture in a material will influence the amount of infrared absorption and radiation. It is possible that this property can be used to detect moisture in asphalt. A similar capability has been developed for remote sensing of near-surface moisture in soils.

The presence of fine vertical cracks in pavement produces localized heat transfer anomalies. Experience with infrared has shown that these cracks can appear more prominent in an infrared image than they might in a visual image, particularly under moist conditions.

5.1.12 Radiography

The principal difficulty in applications of conventional radiography to the inspection of large structures is that the object-to-image plane distances can be no shorter than the thickness of concrete cover. As this distance increases, the "shadow" cast by the steel reinforcement on the image plane, which leads to the formation of the radiographic image, becomes more diffused and resolution is lost.

Radiography is being used to image much larger features inside concrete structures up to 1.2 m (48 in.) thick. The French system "Scorpion" accomplishes this by using a linear accelerator, rather than conventional X-ray tubes or an artificial gamma ray source, to produce an intense beam of high energy radiation. Imaging is accomplished in real time, and radiographic views of the structure are transmitted to the inspector via a television link. Though this is a very sophisticated system for inspection of large structures, image quality is, as one would expect, too poor for the detection of small amounts of corrosion.

5.1.13 Tomography

Computerized tomography is an advanced radiographic imaging method that makes use of multiple views of an object to produce a computer-generated image of high resolution. In fact, it has been demonstrated that resolution better than 1 mm (0.04 in.) can be achieved through 150 mm (6 in.) of concrete. The main difficulty with this method is cost, which would probably approach \$1 million per system. An additional problem is that in imaging some structures it may not be possible to position source and detectors in such a way as to obtain a sufficient number of different views of the object.

A recent variation of the tomographic method employs backscattered radiation, with source detectors on the same side of the object. Although this approach would seem to offer advantages in ease of deployment, a preliminary study indicates that resolution may be much poorer than with the more conventional through-transmission tomography systems. In any case, the cost should be about the same, which would prohibit widespread application.

5.2 Semi-destructive Tests

In contrast to the numerous NDT methods available to assess the concrete members, there are fewer destructive methods developed for this purpose. The methods include penetration tests, core drilling and petrography, which can be used together to determine the strength and other properties of concrete.

5.2.1 Penetration Test (Windsor Probe)

The Windsor Probe device is a gun that uses a powder charge to drive a high-strength steel probe into the concrete to be tested. Generally, three probes are driven in a triangular pattern, and the penetration is determined by measuring the length of the probe extending from the surface of the concrete. Then the depth of penetration average can be converted to concrete strength by using calibration curves supplied by the manufacturer, or preferably by correlation to core samples.

The Windsor Probe equipment is simple, durable, requires little maintenance, and can be used by laymen in the field with little training. It is fast: three tests are made in approximately 5 minutes. ASTM C803 *Standard Test for Penetration Resistance of Hardened Concrete*^(5.1) describes this test method.

The Windsor Probe primarily measures subsurface hardness and does not yield precise measurements of the in situ strength of concrete. The probe test, however, is useful in assessing the quality and relative strengths of concrete. As with the Schmidt hammer (rebound) test, interpretation of test results depends on other known factors, which are based on correlation plots. When used on concrete which is 40 to 50 years old, this test method may yield a higher strength than actually exists.

The Windsor Probe test does minor damage to the concrete over an area of approximately 25 to 50 mm (1 to 2 in.) in diameter, leaving holes about 8 mm (0.3 in.) in diameter for the depth of the probe. It may also cause minor cracking, necessitating superficial repairs.

5.2.2 Core Drilling

A widely accepted semi-destructive test method to check for quality of concrete is core drilling. Generally, cores are drilled and removed to represent the overall concrete in place or to display specific features of interest such as cracks or small voids. Subsequent tests performed on the samples vary according to the needs of the engineer. The basic testing includes visual inspection, compressive strength, modulus of elasticity, Poisson's ratio, and petrography.

5.2.3 Petrography

Petrographic examination reveals the properties of aggregates, including physical condition, petrographic identity and chemical stability, and the properties of cement paste. Using microscopes, X-ray diffraction, and differential thermal analysis, experienced petrographer can evaluate the following features of the concrete:

- Density of cement paste
- Homogeneity of the concrete
- Occurrence of settlement and bleeding of fresh concrete
- Depth and extent of carbonation
- Occurrence and distribution of fractures
- Characteristics and distribution of voids
- Presence of contaminating substances
- Evidence of cement-aggregate reaction

- Proportion of unhydrated granules of cement
- Presence of mineral admixtures
- Volumetric proportions of aggregate, cement paste, and air voids
- Air content and various parameters of the air void system
- Composition in terms of aggregate, hydrated lime, masonry cement, portland cement and other components, and proportions of each
- Admixtures such as air-entraining agents, accelerators, waterproofers, pigments, etc.
- Hydration of the portland cement component, desirable and undesirable hydration reactions, and w/c
- Textural features related to original consistency and workmanship
- Migration paths of water
- The chemical and physical soundness or unsoundness of the components
- Features of the concrete that reflect causes for failure

Table 5.1^(5.3) presents a summary of the information obtainable from examination of hardened concrete by petrographic analysis. Petrography generally includes the description of concrete components and physical properties such as permeability, density, absorptions and cracking patterns. The standard light microscopy can also be supplemented with X-ray diffraction, chemical analysis, scanning electron microscopy, infrared spectroscopy, and many other chemical and physical test techniques. The use of petrographic techniques and chemical methods is extremely valuable for evaluating failures or for determining the potential for future deterioration, and estimating durability. It is very important to utilize consulting firms with petrographers and chemists that are experienced in the analysis of concrete and concrete materials. Concrete petrography is a specialized area, and experienced personnel are essential.

5.3 Strength Evaluation Testing

If the safety of a structure or member is suspected due to low strength concrete, missing or misplaced reinforcement, or deterioration of concrete, load tests are conducted to evaluate the strength. A procedure for making a strength evaluation of an existing structure is published by ACI.^(5.4) The test loads can be applied with various materials such as lead, concrete or steel blocks, or water. The deflections and strains at critical locations are monitored with dial gages and strain gages. The structure is shored so that in case of a failure during the test, complete collapse does not take place.

Hydraulic rams can also be used to apply the load provided that the system against which the rams must react can be designed. The advantage of using rams is that the load on the structure can be removed very quickly.

Acceptance criterion for the structure under load test is that it should not show visible evidence of cracking or spalling. If there is no visible failure, then recovery of deflection criterion is used to determine if the structural strength is adequate.

5.4 Corrosion Testing

Several reliable techniques are available to monitor corrosion of embedded steel in concrete structures. These include half-cell potentials, corrosion currents to reference electrodes, and linear polarization (LP) techniques. Other techniques such as AC impedance are also available but are not well suited for field evaluations. The analysis of the chloride content of the concrete is useful to determine the corrosion of the embedded steel.

5.4.1 Half-Cell Potential

Copper-copper sulfate half-cell tests are performed in accordance with the ASTM Standard Test Method C876, *Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete*.^(5.1) The method involves making one electrical connection between the reinforcing steel and the positive terminal of a high-impedance voltmeter, and another between the negative terminal of the voltmeter and the half-cell. The half-cell consists of a copper electrode, enclosed in a plastic housing, surrounded by a saturated copper sulfate solution. The end of the cell consists of a porous disk, through which the copper sulfate solution can make electrical contact with the concrete.

Copper-copper sulfate half-cells are the most commonly used half-cell for measuring potentials in concrete structures. Other half-cells are available such as calomel and silver-silver chloride. The silver-silver chloride half-cell is useful in measuring potentials of structures exposed to saltwater because the chlorides will not contaminate the silver chloride solution as readily as they would the copper sulfate solution. Silver-silver chloride half-cells should be used any time measurements are made under seawater, or adjacent to saltwater such as in tidal areas.

By taking potential readings at multiple locations, an evaluation of the corrosion activity of embedded reinforcing steel (or other metals) can be made. Work on laboratory slabs at WJE^(5.5) has defined the threshold of corrosion to be more negative than -0.23 volts. Readings more negative than this indicate active corrosion while those less negative indicate no corrosion activity. This threshold value may not be applicable to field structures, especially those in contact with soil. If sufficient readings are taken on a grid pattern, a diagram can be prepared that resembles a contour map. On such a diagram, points of equal electrical potential are connected by isopotential lines, permitting areas of high potentials to be readily identified.

The moisture content of the structure is an important factor when measuring half-cell potentials. Interior concrete protected from moisture becomes very dry and accurate potentials cannot be measured without wetting the concrete. The grid locations should be repeatedly wetted prior to measuring potentials, and wet sponges should be placed at the grid points. The half cell-readings should be stable. If they are not, additional wetting time is required. The dryness of the concrete and accuracy of the reading can also be monitored by reducing the circuit resistance and monitoring the stability of the potential. If the potential does not change over a broad range of circuit resistance, the reading should be accurate.

Interpretation of half-cell potential measurements of structures in contact with the ground must be done with caution. Soils can have very high potentials due to ferric components in the soil. Corrosion in the structure may not be occurring even though the potentials are very high, even more negative than 0.7 volts. Differences in the values should be studied, and areas of highest potential or steepest potential gradients should be investigated further. Exposing the steel for visual observation or measuring the chloride content in the concrete at the level of the steel are the best verification methods.

For direct measurement of corrosion currents, sections of bars can be isolated by sawcutting and monitored. The isolated section of bar or bars is reconnected to the surrounding steel through a measuring circuit. The corrosion current of the isolated section can then be measured in relationship to the structure. If the isolated section is in a high chloride environment relative to the structure it will become anodic to the structure and corrode. The corrosion current can then be measured using a resistance-less ammeter, or by measuring the voltage drop across a 10-ohm resistor.

5.4.2 Linear Polarization Measurements

Cyclic potentiodynamic polarization measurements or linear polarization (LP) provide a means of determining the susceptibility to localized corrosion of steel in concrete. The method is a refinement of the half-cell potential technique. The main disadvantage to LP measurements is that each measurement may take between 5 to 20 minutes, whereas half-cell potential measurements are almost instantaneous. Also, specialized equipment is needed for LP measurements.

The apparatus includes a potentiostat, high impedance voltmeter, current measuring device, and electrodes. Counter and reference electrodes are placed on the concrete surface. An electrical connection is made to the steel in a similar manner to that required for half-cell potential measurements. The initial potential is measured, then a current is impressed to shift the potential a predetermined amount. The amount of current required to shift the potential is recorded. This process is continued for several steps of potential and a polarization curve is developed. The amount of corrosion is indicated by the slope of the curve. LP can also be used to determine the susceptibility of metals to corrode. This is measured as the potential at which the anodic current increases rapidly and a breakdown in potential is seen. The more positive the potential, the less susceptible the metal is to corrosion.

5.5 Case Studies Using Nondestructive Evaluation Techniques

5.5.1 Investigation of a Concrete Foundation Mat

During placement of a rectangular foundation mat, concerns were expressed regarding concrete quality and potential internal honeycombing due to insufficient vibrator penetration and placement of lifts greater than 0.46 m (18 in.). The foundation slab measured 15.2 m x 21.3 m (50 ft x 70 ft), with a thickness of 2.4 m (8 ft). It contained No. 11 reinforcing bars spaced 0.15 m (6 in.) on center in both directions, top and bottom.

Three vertical sides and the top surface of the foundation mat were accessible for the investigation. The north and east vertical sides were fluted as a result of the use of corrugated metal for formwork, to provide for shear transfer to the adjacent slabs (Fig. 5.8). The west vertical side had a smooth surface. A procedure was established to take pulse velocity readings on paths between the three exposed vertical sides (north, east, and west) and the top surface. On the vertical sides, the grid points on each line were spaced 0.6 m (2 ft) apart and on the top surface they were 3.0 m (10 ft) apart.

Straight shots were made at each grid point on the vertical sides to an adjacent location on the same grid lines on the top surface (Fig. 5.9). In addition, diagonal shots were made typically at each grid point on the vertical sides to adjacent points on the parallel grid lines on the top surface, as shown in Fig. 5.10. Approximately 400 readings were obtained through the concrete mat.

The summary of data is presented in Table 5.2. The measured velocities were consistent and uniform. Low coefficients of variations are indicative of reliable data. Velocities above 3,660 m/sec (12,000 ft/sec) are indicative of good quality concrete.

5.5.2 Testing of a Discharge Tunnel Slab

The south end of a discharge tunnel slab showed signs of deterioration, which included surface delamination and visible cracks. Some of these cracks penetrated through the nominal 0.9 m (3 ft) thickness of the slab, causing water leakage problems during periods of high tide. Because of the visible deterioration of the concrete on the surface, there was concern about the possible spalling of concrete on the underside of the slab. It was decided to survey the affected area (9.1 m x 12.2 m [30 ft x 40 ft]) using the impact-echo technique.

The impact-echo tests were performed in two areas. Area No. 1, which did not show evidence of concrete deterioration, was tested and the results were used as a reference for comparison with the results of the tests on the Area No. 2—the deteriorated area.

In Area No. 1, 20 tests were conducted at grid points located 1.5 m (5 ft) apart. The frequencies associated with the nominal 0.91 m (36-in.) thick slab varied between 2000 Hz and 2375 Hz as shown in Fig. 5.11. In Area No. 2, a total of 60 impact-echo tests were conducted at the grid points, which were also 1.5 m (5 ft) apart. The test results showed that frequencies associated with the slab thickness varied between 2000 Hz and 2500 Hz. The Area No. 2 showed a number of locations with surface delamination. Delaminated areas generally show frequencies below 2000 Hz. A representative spectrum plot of a delaminated area with a primary frequency of 1250 Hz is shown in Fig. 5.12.

In the impact-echo method, the response of the concrete structure to mechanical impact is monitored and analyzed. Concrete members of uniform thickness are generally characterized by one primary peak due to reflections of longitudinal waves on the back side of the slab. In the case of the discharge tunnel, the nominal thickness of the slab is 0.9 m (36 in.). The spectra of the impact echo tests conducted in Area Nos. 1 and 2 showed frequencies associated with the nominal thickness and surface delaminations. The observed frequency variation between 2000 Hz and 2500 Hz in non-delaminated areas is within the limitations of the impact echo technique. The frequency variation represents a thickness variation of 102 mm (± 4 in.).

Based on the impact echo test results, it was concluded that the thickness variation between Areas No. 1 and No. 2 was insignificant. Therefore, it was unlikely that Area No. 2 contained deep spalled-off concrete on the underside of the slab as compared to the Area No. 1. Our findings were verified by visual observations by a diver.

5.5.3 Testing of a Column of a Turbine/Generator Pedestal

Upon removal of formwork, poorly consolidated concrete was observed on the north face of two columns of a turbine/generator pedestal (Fig. 5.13). Concern was expressed about the overall concrete quality and its potential for honeycombing, at locations other than on the surface of the concrete. The suspected areas were tested using impact-echo as well as pulse velocity techniques.

Impact-echo testing was performed on the north and south faces of columns, using grid lines spaced at 0.3 m (1 ft) on center. The tests were performed at each grid point by making an impact near the transducer and monitoring the signal response of the structure on an analyzer. The impact-analysis procedure was repeated several times at each grid location to assure that a consistent, repeatable response was produced. If the grid locations were chipped or had a very irregular surface, examination of the concrete was carried out at the nearest smooth surface.

A representative spectrum is shown in Fig. 5.14. A frequency peak at 650 Hz is associated with the column depth of approximately 3 m (10 ft). Figure 5.15 represents a spectrum plot of the patched area where unconsolidated honeycombed concrete was discovered. None of the results showed any frequencies associated with significant internal flaws.

Propagation velocity measurements were also taken at the same grid points as used in impact-echo testing. Based on a nominal column thickness of 3.0 m (10 ft), the average propagation velocity in the concrete was calculated as 3,844 m/sec (12,610 ft/sec) with a standard deviation of 79.5 m/sec (261 ft/sec). This velocity is indicative of good concrete. None of the test locations showed significant variations in the velocity readings, indicating uniform quality concrete.

To verify the nondestructive testing results, three concrete cores were removed from the column for petrographic inspection. The examination revealed that the paste-aggregate bond was

tight to moderately tight and no aggregate contaminants or coatings on the aggregate were observed. Normal entrapped air voids were observed in all three cores.

5.5.4 Inspection of Cooling Towers

The owners of a power plant wanted to inspect the cooling towers (Fig. 5.16) to assess their condition. The condition survey consisted of visual inspection, core samples, pachometer readings and condition of the concrete. The cooling towers are about 98 m (323 ft) high. Veil thickness for the bottom 24 m (80 ft) varies from 0.15 m (6 in.) at the 24 m (80 ft) level to 0.45 m (18 in.) at the bottom. Since the access to the veil is only from side; that is, from outside, "through transmission" method could not be applied. Hence, the impact-echo technique was used to make thickness measurements.

A standard rebound hammer was used as an impact source. The FFT analyzer used was a portable compact unit with a large video display and a micro floppy disk. The transducer used was a piezoelectric element of PZT-4 material, housed in a stainless steel case. It is normally used in a through-transmission method.

An average propagation velocity of 4,035 m/sec (158,851 in./sec.) in the veil concrete was determined by the impact-echo technique. Measurements were taken at 0.9 (3 ft) intervals from the 21.3 m (70 ft) elevation (see Table 5.3). Theoretical thickness values were determined from the drawings of the cooling tower. Figure 5.16 shows a spectrum plot with a peak at 13,200 Hz, which relates to a veil thickness of 6 in. Impact-echo testing did not indicate significant defects in the concrete.

5.6 References

- 5.1 Annual Book of ASTM Standards, American Society for Testing and Materials, Philadelphia, Penn., 1992
- 5.2 Chung and Law, *Diagnosing In-Situ Concrete by Ultrasonic Pulse Technique*, Concrete International, Oct. 1983.
- 5.3 Bernard Erlin, *Methods Used in Petrographic Studies of Concrete*, Analytical Techniques for Hydraulic Cement and Concrete, ASTM STP No. 395, American Society for Testing and Materials, 1966, pp. 3-17.
- 5.4 American Concrete Institute Committee 437, *ACI Manual of Concrete Practice, Part 3*, Detroit, Mich., 1992
- 5.5 Pfeifer, D., Landgren, R., and Zoob, A., *Protective Systems for New Prestressed and Sub-structure Concrete*, FHWA/RD-86/193, National Technical Information Service, Springfield, Va., Apr. 1987.

Table 5.1 - Information Obtainable from Hardened Concrete by Petrographic Analysis^(5.3)

TABLE 1—Information Obtainable from Hardened Concrete.

Technique or Method	Information Obtainable	Possible Interpretations	Limitations
Light Microscopy	<p>ASTM Recommended Practice C457 air-void system</p> <p>Aggregate-composition, texture, classification, other properties given in ASTM Recommended Practice C295, reaction rims</p> <p>Proportions of aggregate and paste</p> <p>Cracking patterns</p> <p>Identification of solid admixtures</p> <p>Extent of cement hydration</p> <p>Composition, fineness, and dispersment of relic cement particles</p> <p>Identification of hydrated cement compounds</p> <p>Identification and location of secondary compounds</p> <p>Detection of "unaccommodative" chemical reactions</p> <p>Physical properties of the paste such as hardness, granularity, porosity, density</p>	<p>Potential effectiveness of the air-void system</p> <p>Aggregate unsoundness</p> <p>Type of cement-aggregate reaction</p> <p>Effect of cement composition on concrete behavior</p> <p>Curtailment of cement hydration</p> <p>Relative cement content</p> <p>Relative water-cement ratio</p> <p>Distribution of stress and probable causative agency</p> <p>Type of distress and whether caused by internal or external agency</p> <p>Deviation from specification</p>	<p>Many interpretations are subject to personal judgement based upon experience.</p>
X-ray Diffractometry	<p>Aggregate mineralogy</p> <p>Identification of secondary compounds</p> <p>Identification of hydrated and unhydrated cement compounds—best determined after crushing and removal of aggregate</p>	<p>Type of aggregate</p> <p>Type of distress</p> <p>Extent of cement hydration</p>	<p>Amorphous compounds are undetectable.</p> <p>Compounds present in low concentrations (i.e. 5% or less) may be difficult or impossible to identify.</p>

Table 5.1 - (Continued)

X-ray Spectroscopy	Identification and relative proportion of elements present in aggregate and paste	Estimate of cement content Presence of elements such as zinc and lead that may indicate the presence of compounds responsible for abnormal set High concentrations of certain elements that may indicate types of distress	Supplementary microscopy and X-ray diffractometry are necessary to identify the combination of elements as specific compounds.
X-ray Radiography	Cracking patterns	Distribution of stress	Some difficulty exists in resolving cracking in the mortar fraction of concrete.
Wet-Chemical Analysis	ASTM Method C85 cement content Chemical composition of aggregate Chemical composition of paste Chemical composition of secondary compounds Detection of some organic substances	Excessive quantities of certain substances that may be indicative of specific types of deterioration Quantitative estimate of cement content Deviation from specification	Supplementary microscopy and X-ray diffractometry are necessary to identify specific compounds present.
Electron Microscopy and Diffractometry	Identification of compounds in aggregate and paste Identification of secondary substances	Type of aggregate Type of distress	Certain substances are subject to alteration during examination.
Infrared Spectroscopy	Identification of many organic groups	Presence of organic admixtures Deviation from specification	Some organic groups are undetectable.

Table 5.2 – Summary of Pulse Velocity Test Results

Shot Designation and Location	Total Number of Shots	Mean Velocity, fps	Standard Deviation, fps	Coefficient of Variation, %
Straight Shots				
East Side	54	14,450	700	4.8
West Side	51	14,950	450	3.0
North Side	69	14,970	690	4.8
Diagonal Shots				
East Side	54	14,370	820	5.7
West Side	48	14,640	930	6.3
North Side	75	14,780	770	5.2

Table 5.3 - Thickness Data on the Cooling Tower

Elevation, ft	Thickness, in.	
	Measured	Theoretical
30.0	17.2	16.1
32.8	14.7	13.7
35.7	12.4	11.3
38.5	9.2	8.9
41.4	6.6	6.5
44.2	6.2	6.0
47.1	6.3	6.0
50.0	6.7	6.0
52.8	6.5	6.0
55.7	5.9	6.0
58.6	6.3	6.0
61.4	6.0	6.0
64.3	6.0	6.0
67.2	6.2	5.9
70.0	6.1	5.7

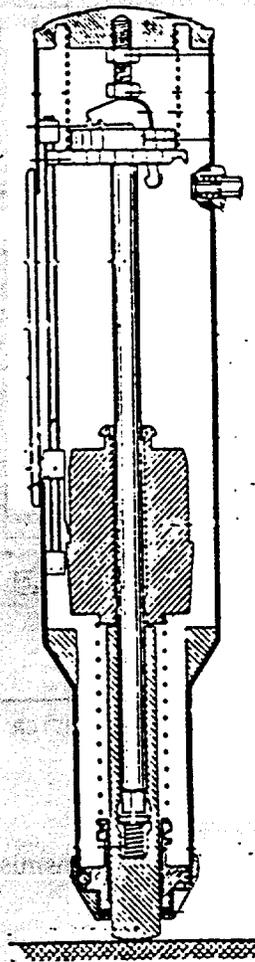


Fig. 5.1 - Rebound hammer

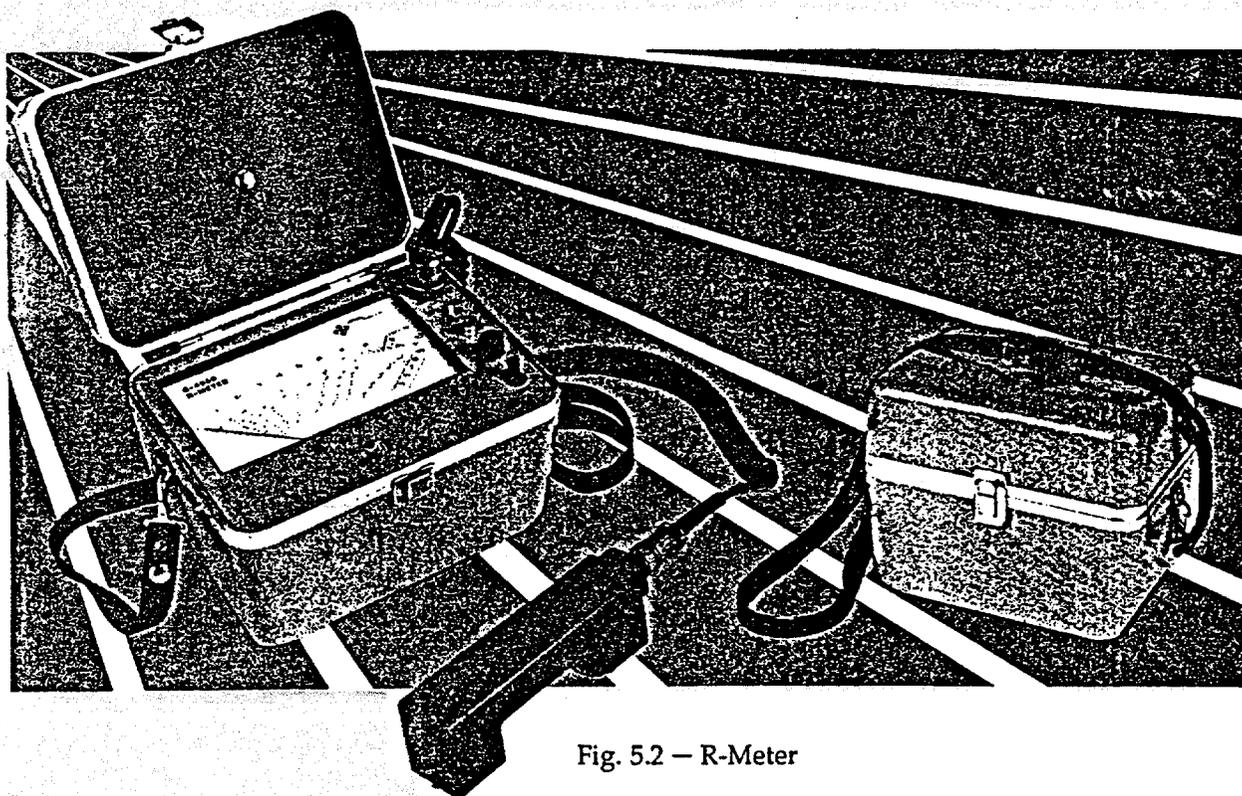


Fig. 5.2 - R-Meter

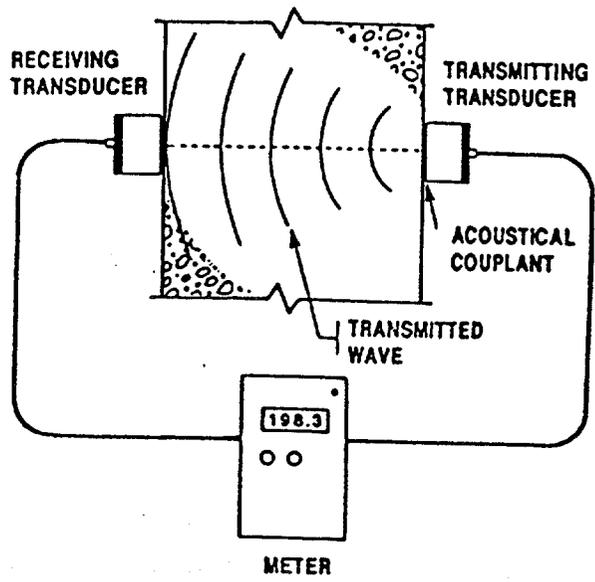


Fig. 5.3 – Through-transmission method

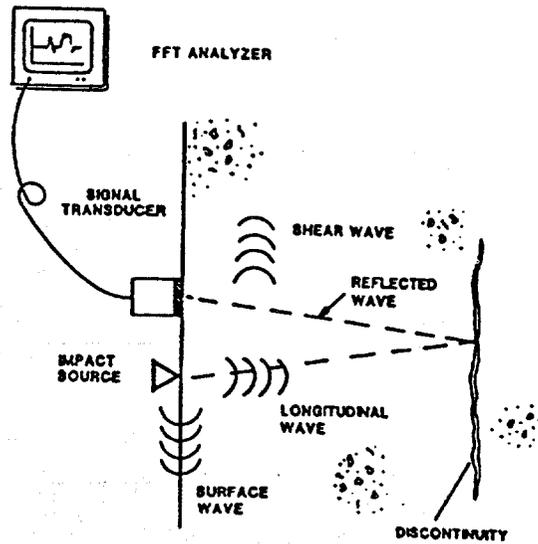


Fig. 5.4 – Impact-echo method

IMPULSE RESPONSE METHOD

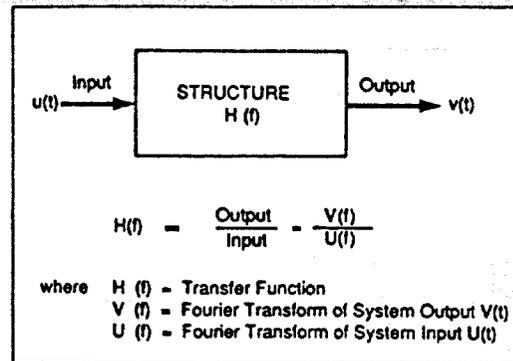
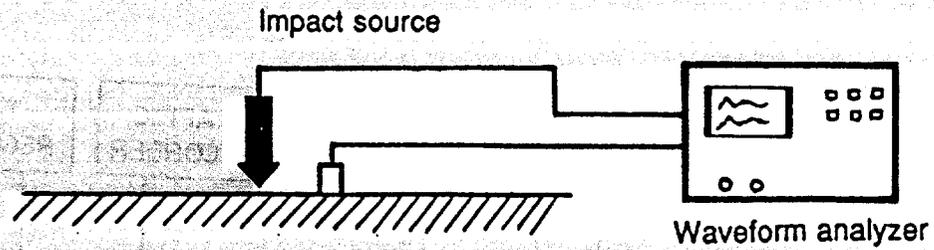


Fig. 5.5 - Impulse response method

Spectral Analysis of Surface Waves (SASW) Method

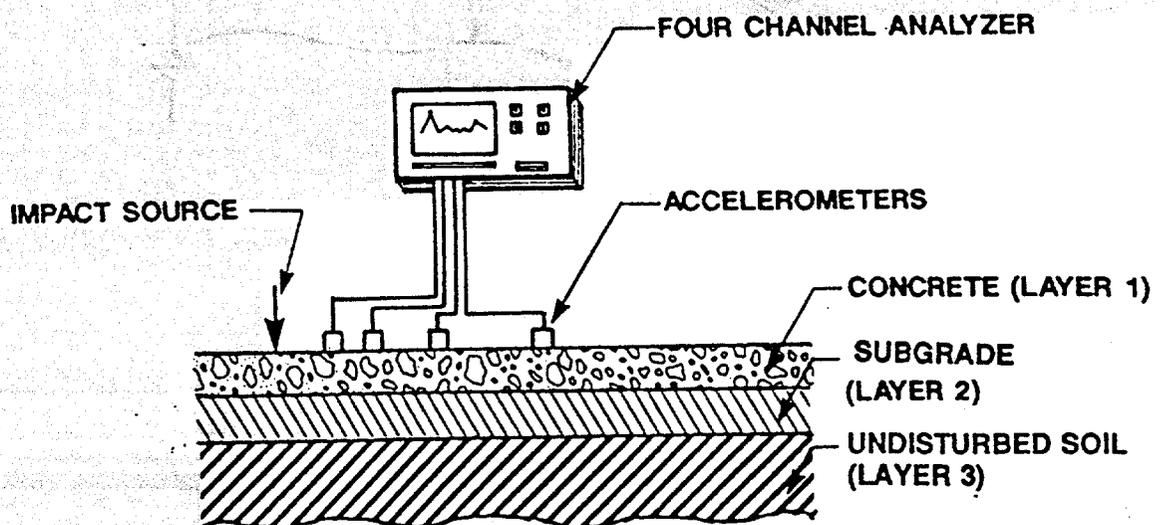


Fig. 5.6 - Spectral analysis of surface waves (SASW) method

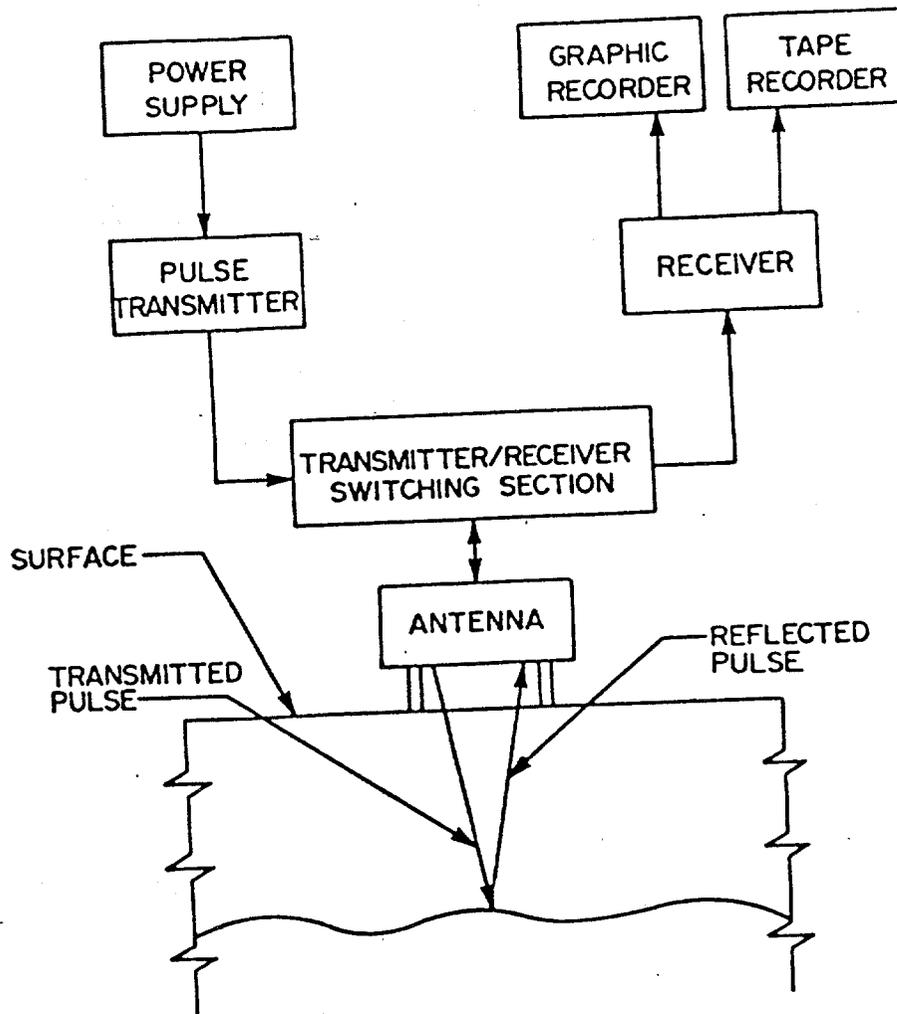


Fig. 5.7 – Radar profiling technique

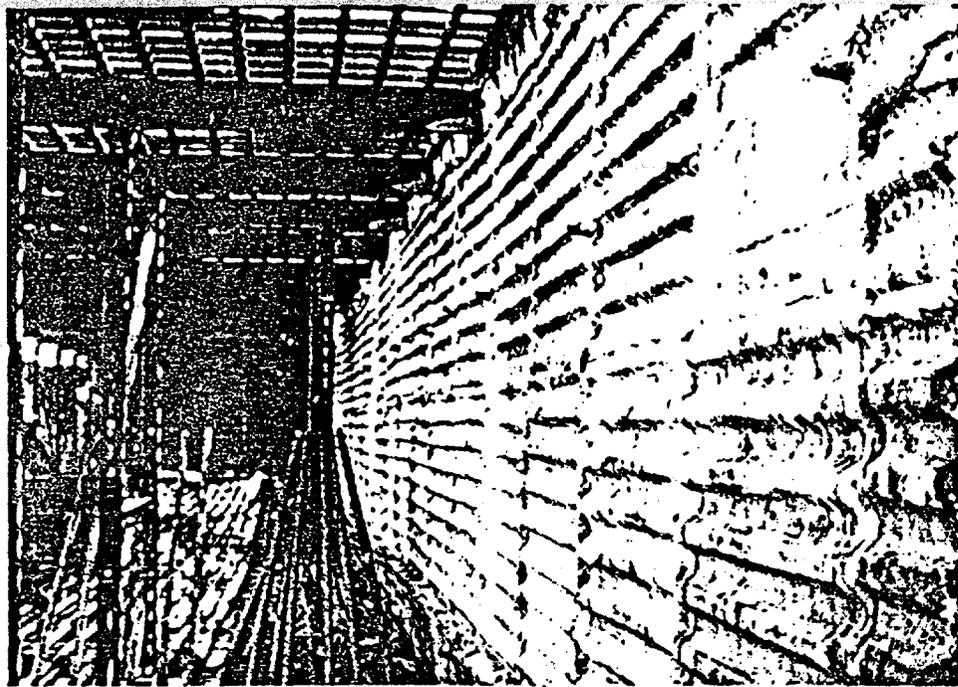


Fig. 5.8 — View of east vertical side

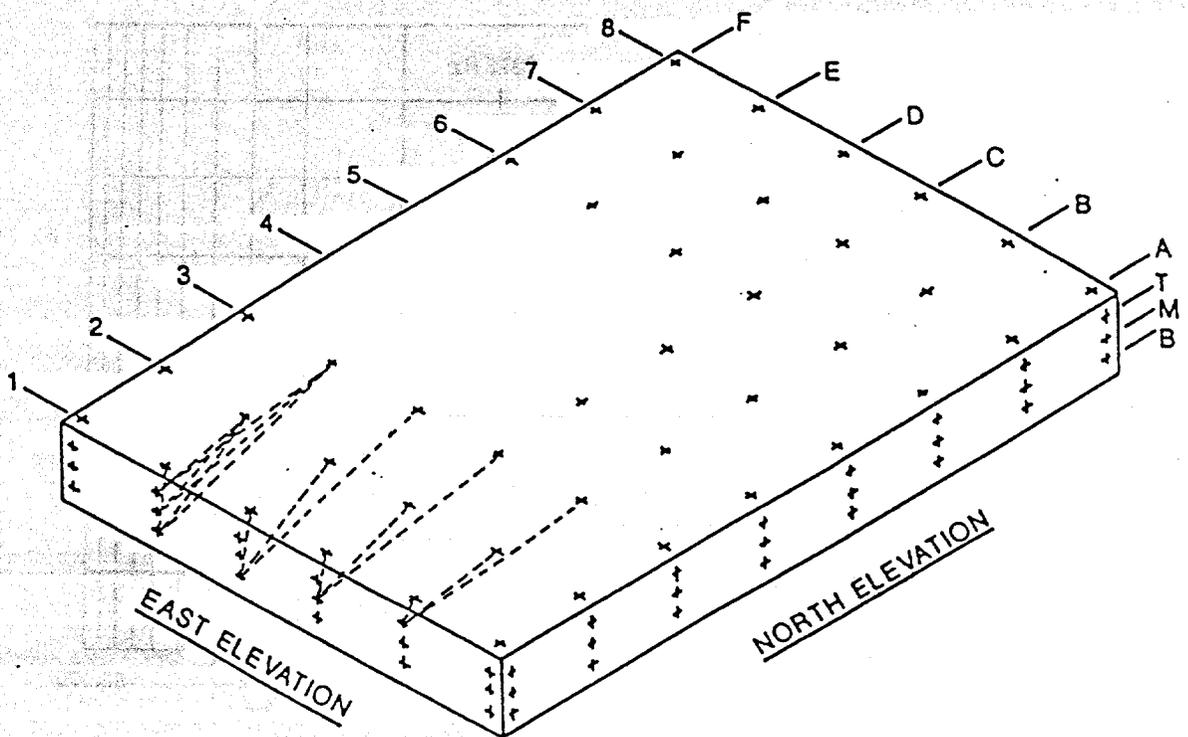


Fig. 5.9 — Schematic of straight shot coverage

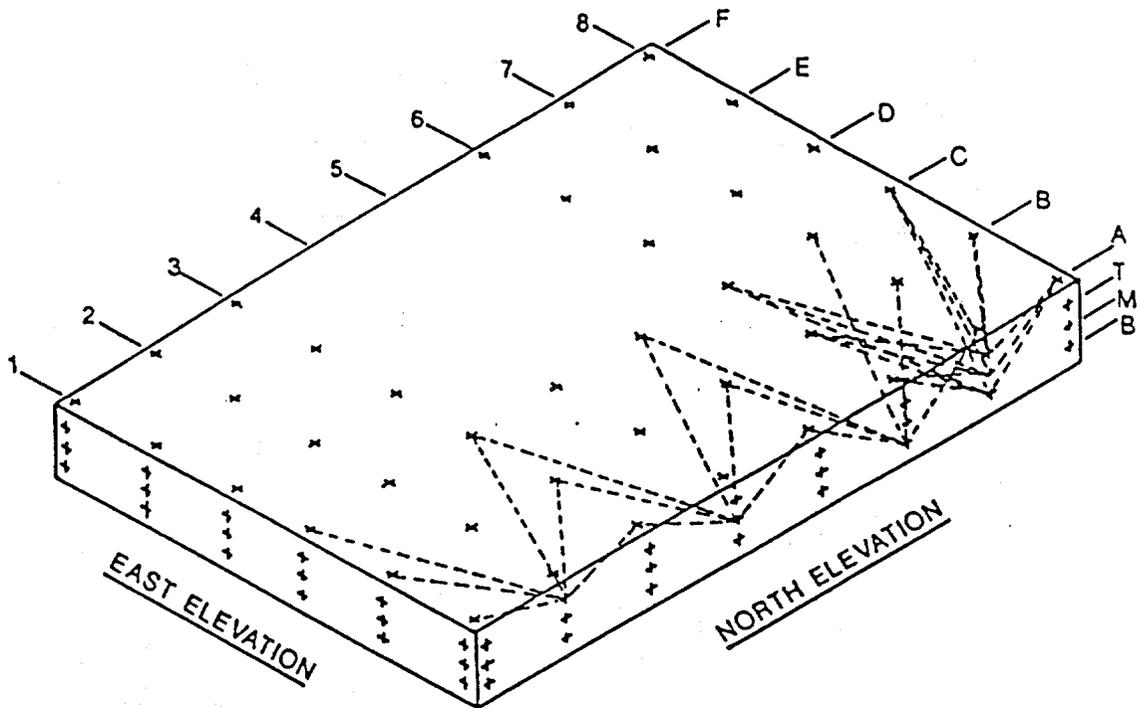


Fig. 5.10 – Schematic of diagonal shot coverage

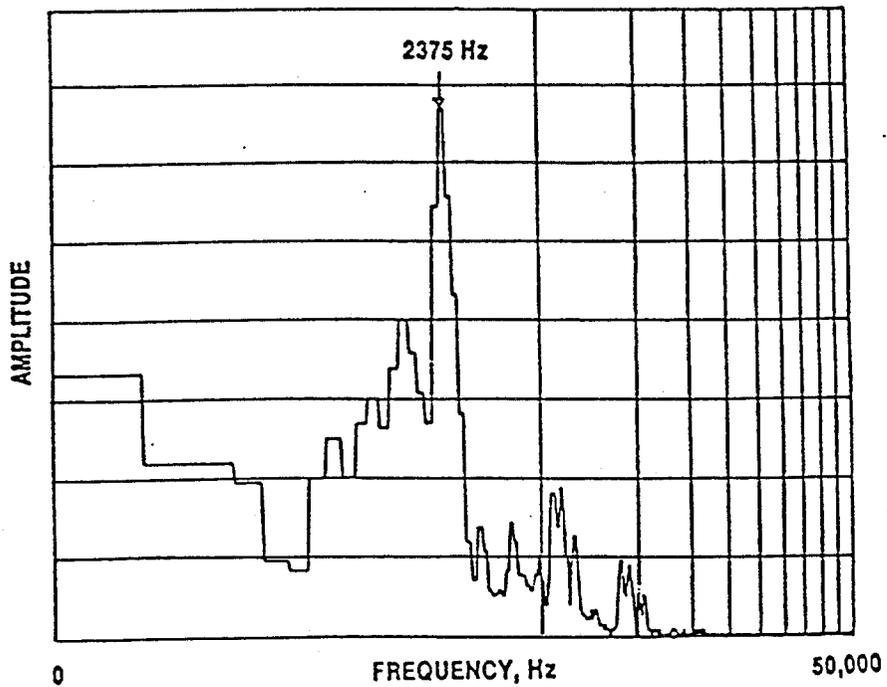


Fig. 5.11 – Representative spectrum plot

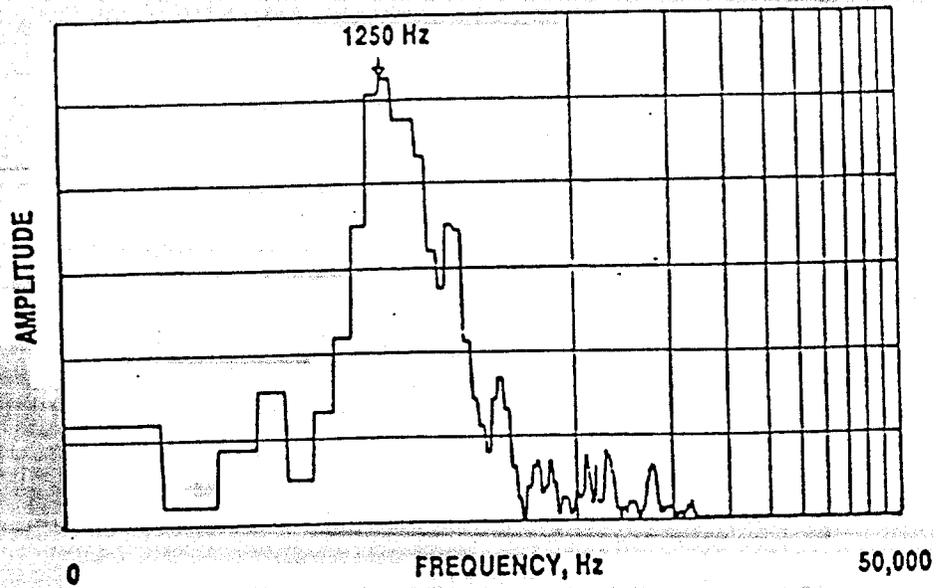


Fig. 5.12 - Spectrum plot of a test conducted in a delaminated area

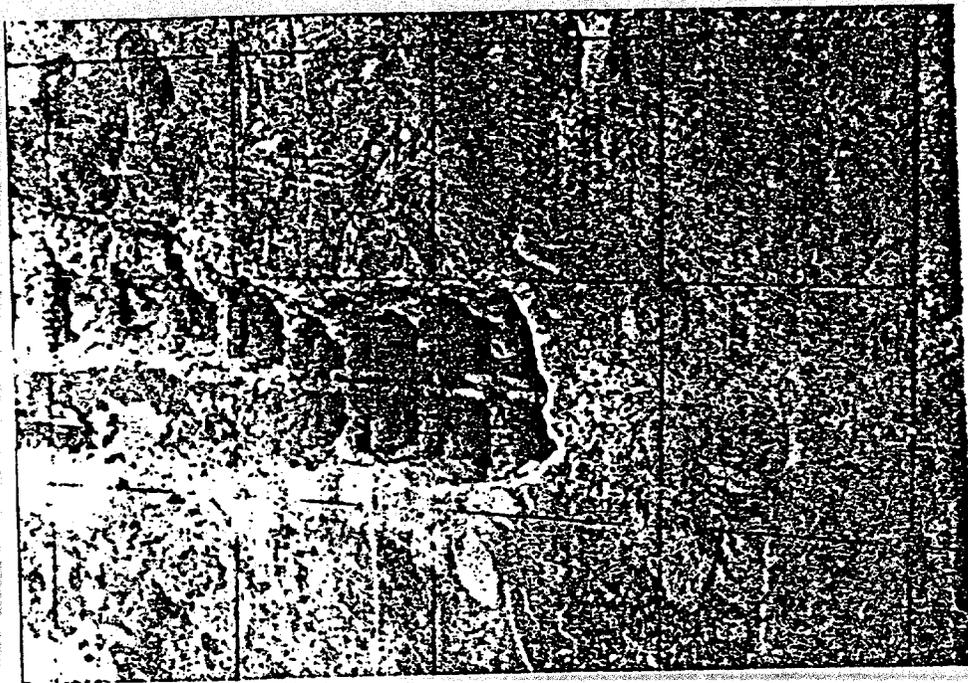


Fig. 5.13 - View of the column showing a surface void

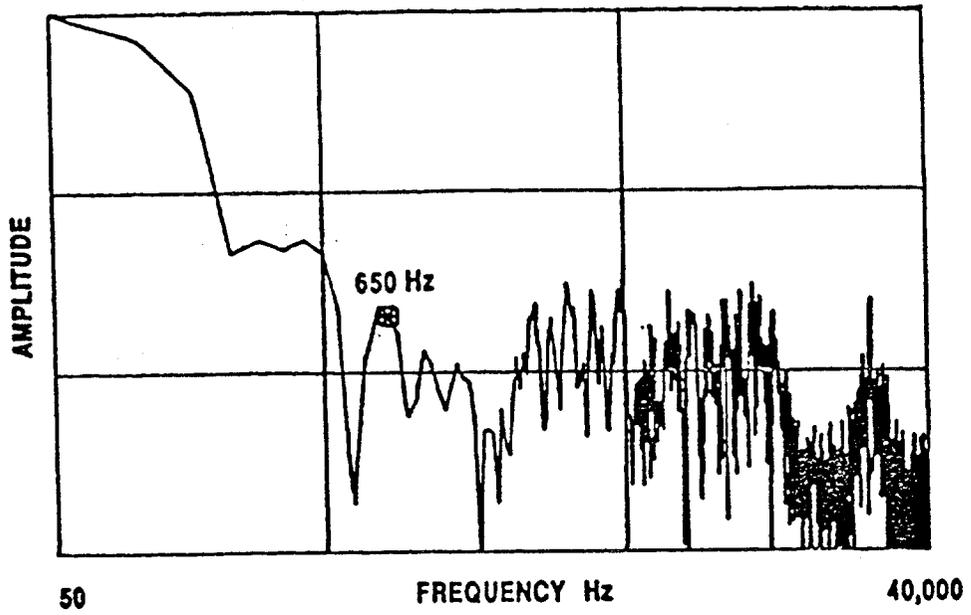


Fig. 5.14 - Representative spectrum plot

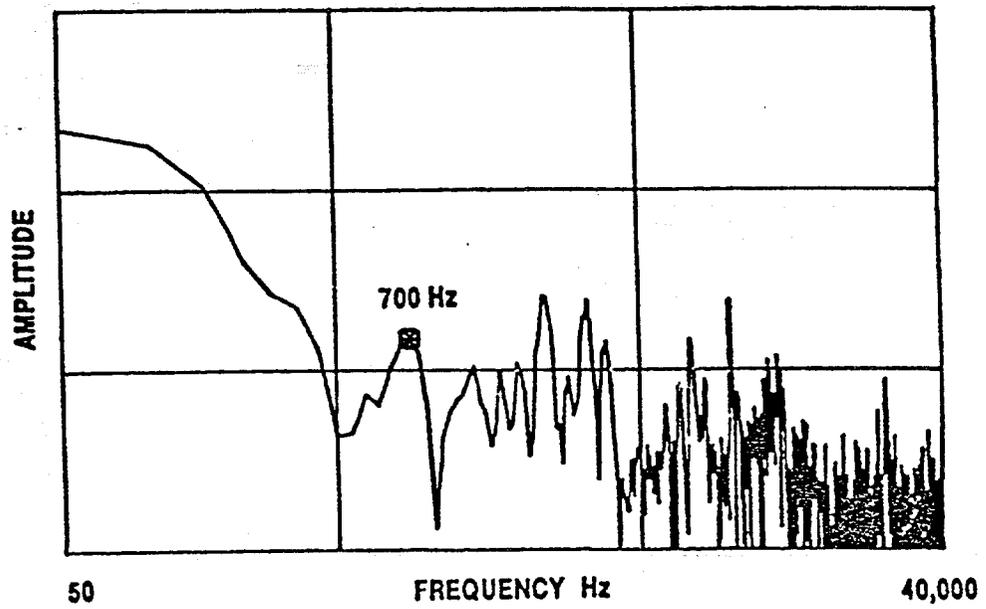
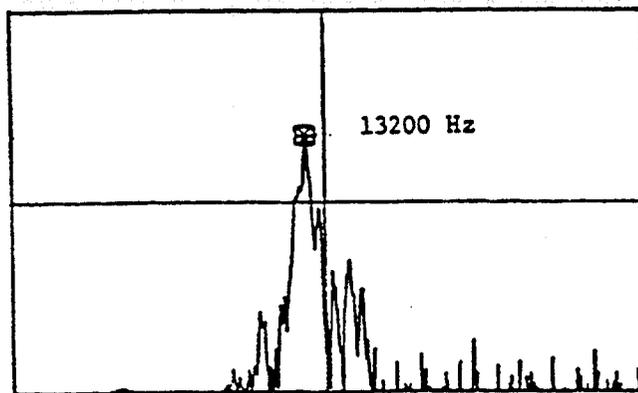
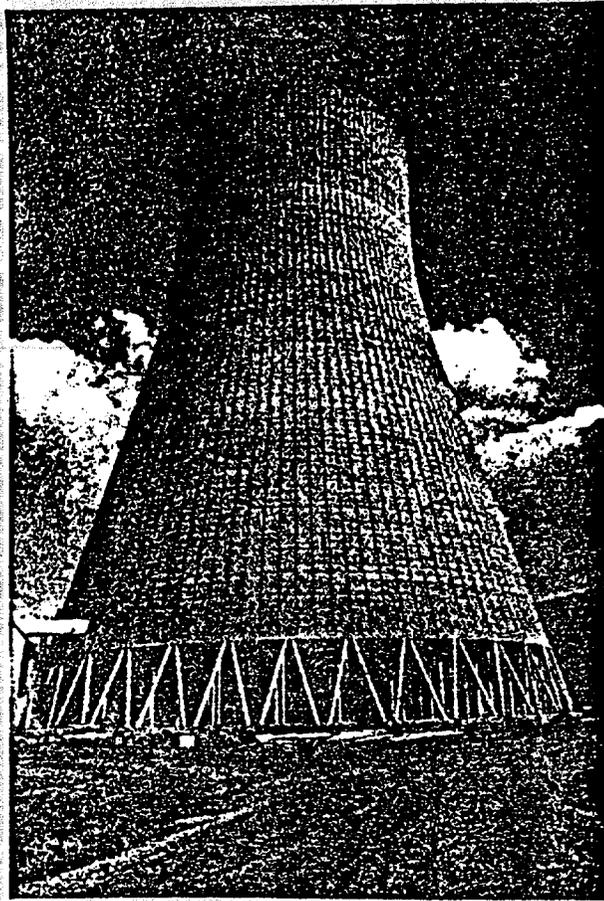


Fig. 5.15 - Spectrum plot of a test conducted in a patched area



Frequency spectrum obtained over a 6-in. thick veil

Fig. 5.16 - Cooling tower

6. GENERAL SUMMARY

This report discusses many aspects of concrete deterioration and repair of concrete structure in nuclear power plants. The emphasis of this report is on improving the durability of repairs and providing groundwork for personnel involved with repairs of concrete structures in nuclear power plants. A plant questionnaire identified the most common types of distress and typical locations of distress. The results of the questionnaire provided an overall idea on where and what type of problems have occurred in nuclear power plants. Very few problems were identified and most were minor. However, due to the relatively young age of most of the plants other types of distress may become more prevalent as the plants age.

This report describes the types of concrete deterioration that occur to concrete structures with emphasis on the most common types that affect long-term durability of nuclear power plants. The proper identification of the deterioration and knowledge of the cause of the deterioration is essential before repair procedures can be selected. The discussions concerning deterioration are also helpful for engineers involved with new structures since the best way to achieve durable structures is to properly construct them with durability in mind. Procedures for conditions surveys of structures and evaluation techniques are also described.

Repair methods for various types of distress are discussed. Little information is available on the long-term durability of repairs. The success of a repair is largely dependant on workmanship during installation. If the repair is properly designed using the best materials, failure can still occur if it is not properly installed. Experienced personnel should always be used to design and install repairs.

The most common distress in nuclear power plants was cracking caused by shrinkage and thermal movements. It is important that the cause of the cracking be determined and whether the crack is active or dormant. It also must be decided if the crack needs repair. Many fine cracks may not need repair if the concrete is not in an aggressive environment. Cracks less than 0.10 mm (0.004 in.) rarely require repair unless they are leaking. Table 6.1 shows a general guide to the recommended repair options for various types of cracks. A perceived durability index is also listed. The techniques are rated based on a random scale of 1 to 5, with 1 being the most durable. These ratings are based on general experience and opinions of engineers familiar with repairs. The tables in this chapter are provided for general reference only and should not be relied upon solely for decision-making purposes. Other references should be reviewed and site conditions considered. Careful review of all options should be performed prior to the selection of a repair technique. Often a combination of techniques are best suited.

Spalling of concrete was the next most common form of distress in the plant survey. Concrete spalling is most commonly caused by reinforcing steel corrosion, impact, chemical or acid attack, abrasion, thermal movements, and cyclic freezing. Repair methods for various types of spalling are shown in Table 6.2. This table is provided for general reference, and the perceived durability rating should not be used solely for repair selection.

Table 6.3 shows the perceived durability of various bonding agents for portland cement concrete patches and overlays. Attention to proper surface preparation and moisture of the substrate is essential for a successful repair. Table 6.4 ranks various means of surface preparation for overlays and spall repairs. The concrete should be saturated surface dry or drier for most cementitious overlays and as dry as practical for polymer concrete overlays.

Reinforcing steel corrosion was a problem cited by many plants. The concrete should always be removed at least 1 in. behind the corroded steel. The bar and excavations should be abrasively blasted. Workmanship is important for a successful repair. Generally the bar should be coated with

two coats of epoxy, then patched with a low w/c concrete. The use of secondary means of protection such as calcium nitrite, membranes, sealers, or polymer concrete is also suggested. Cathodic protection may be an option for large areas and is a means to stop corrosion. Several methods for repairing corroded steel in spalled areas are listed in Table 6.5.

Table 6.6 shows various surface sealers and the related durability ranking. This table should be used with caution, as the performance of materials within each group can vary widely. Silane and siloxane sealers tend to be best for water and chloride resistance, if the structure periodically dries. They penetrate the concrete and do not change its appearance. If carbonation is a concern, a barrier coating such as an acrylic or epoxy is best. A combination of sealers may be best and silanes can be followed by most barrier coatings. Sealers are not commonly used by the plants, but are a valuable tool to extend the life of structures subjected to carbonation, water, or chlorides.

Deterioration due to alkali-silica reactions or sulfate attack were not reported by the plants. These require special mention since they do not exhibit distress early in the structure's life and are very difficult to repair after deterioration starts. Periodic inspections should be implemented by the plants to identify and monitor any areas that develop pattern cracking. Special attention should be given to areas that are continuously or periodically moist and at elevated temperatures. Cooling tower structures, water intake structures, and the tops of exposed foundations are particularly susceptible.

The relatively excellent performance of concrete structures in nuclear power plants reflect the initial quality control, the young age, and the generally benign environment within a plant. Most distress has been construction related or associated with exterior structures such as seawater intakes or discharges. As the plants age, it becomes more important to monitor the aging of the concrete structures to identify potentially deleterious reactions that may adversely effect the continued serviceability of the plant.

Table 6.1 – Crack Repairs

		Perceived durability rating (1-5*)	Comments
Dormant pattern or fine cracking	Judicious neglect	4	Only for fine cracks
	Autogenous healing	3	Only on new concrete
	Penetrating sealers	2	Use penetrating sealer for H ₂ O, Cl resistance
	Coatings	3	Use coating for abrasion & chemical resistance
	HMWM or epoxy treatment	2	Topical application, bonds cracks
	Overlay or membrane	2	For severely cracked areas
Dormant isolated large cracking	Epoxy injection	1	Needs experienced applicator
	Rout and seal	3	Requires maintenance
	Flexible sealing	4	Requires maintenance
	Drilling and plugging	3	
	Grout injection or dry packing	4	
	Stitching	5	
	Additional reinforcing	4	
	Strengthening	3	
Active cracks	Penetrating sealer	3	Cracks less than 0.5 mm (0.020 in.)
	Flexible sealing	3	Requires maintenance
	Rout and seal	3	Use for wide cracks
	Install expansion joint	2	Expensive
	Drilling and plugging	4	May cause new cracks
	Stitching	4	May cause new cracks
	Additional reinforcing	3	May cause new cracks
Seepage	Eliminate moisture source	1	Usually not possible
	Chemical grouting	2	Several applications may be necessary
	Coatings	4	May have continued seepage
	Hydraulic cement dry packing	4	May have continued seepage

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

Table 6.2 — Spall Repairs

	Perceived durability rating (1-5*)	Comments
Shallow Spalling		
Portland cement grouts	3	Not good for acid attack
Polymer-modified grout	2	Different thermal coefficient
Coatings	4	Limited to shallow areas
Membranes	3	Acids— epoxy, methacrylate, butyl, neoprene
Polymer grouts	2	Acids— use polyester grout
Deep Spalling		
Portland cement concrete	2	Inexpensive
Expansive cements	3	Unreliable expansion
Gypsum-based concrete	5	Do not use in moist environments
High alumina (modified)	3	Bonds best to dry concrete
Magnesium phosphate	2	Base concrete must be dry
Polymer-modified	2	Thermal stress can be high
Polymer patching materials	3	Less than 40 mm (1½ in.) thick
Polymer overlays	2	25 to 50 mm (1 to 2 in.) thickness
LMC overlays	2	Greater than 30 mm (1¼ in.) thickness
PCC overlays	3	Use low w/c and HRWR
Silica fume overlays	3	High strength
Pre-placed aggregate	2	Low shrinkage
Shotcrete	3	Good for large areas

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

Table 6.3 — Bonding Agents for Overlays and Spall Repairs

	Perceived durability ratings (1-5*)	Comments
PCC Overlays and patches		
No bond agent	2	Scrub concrete into surface
Cement and water	4	High shrinkage
Cement, water and sand	2	Sand reduces shrinkage
Acrylic latex (SBR) with sand and cement	2	Requires proper formulation
PVA latex	5	Looses bond in moist environments
Epoxy	4	Need proper formulation
Epoxy emulsions	4	New technology (no experience)
Neat latex (SBR)	5	Poor bond

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

Table 6.4 — Surface Preparation for Overlays and Spall Repairs

	Perceived durability ratings (1-5*)	Comments
Steel shotblasting	1	Limited use on vertical or very irregular surfaces— not practical for spall repairs
Hydrodemolition	2	Concrete surface left wet, debris clean up
Abrasive blasting	2	Operator sensitive, can be non-uniform
Impact hammers	3	Fractures base concrete
Acid etching	4	Acid must be thoroughly rinsed
Air sweep	5	Does not provide roughening

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

Table 6.5 — Repair to Corroded Steel

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

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

Table 6.6 — Concrete Surface Sealers

	Perceived durability ratings (1-5*)	Comments
Silane	1	Clear penetrating
Siloxane	1	Clear penetrating
Epoxy	2	Barrier coating
Methacrylate	2	Barrier coating
Urethane	3	Barrier coating
Butadiene	3	
Silicate	4	
Siliconate	4	
Boiled linseed oil	4	Needs repeated application

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

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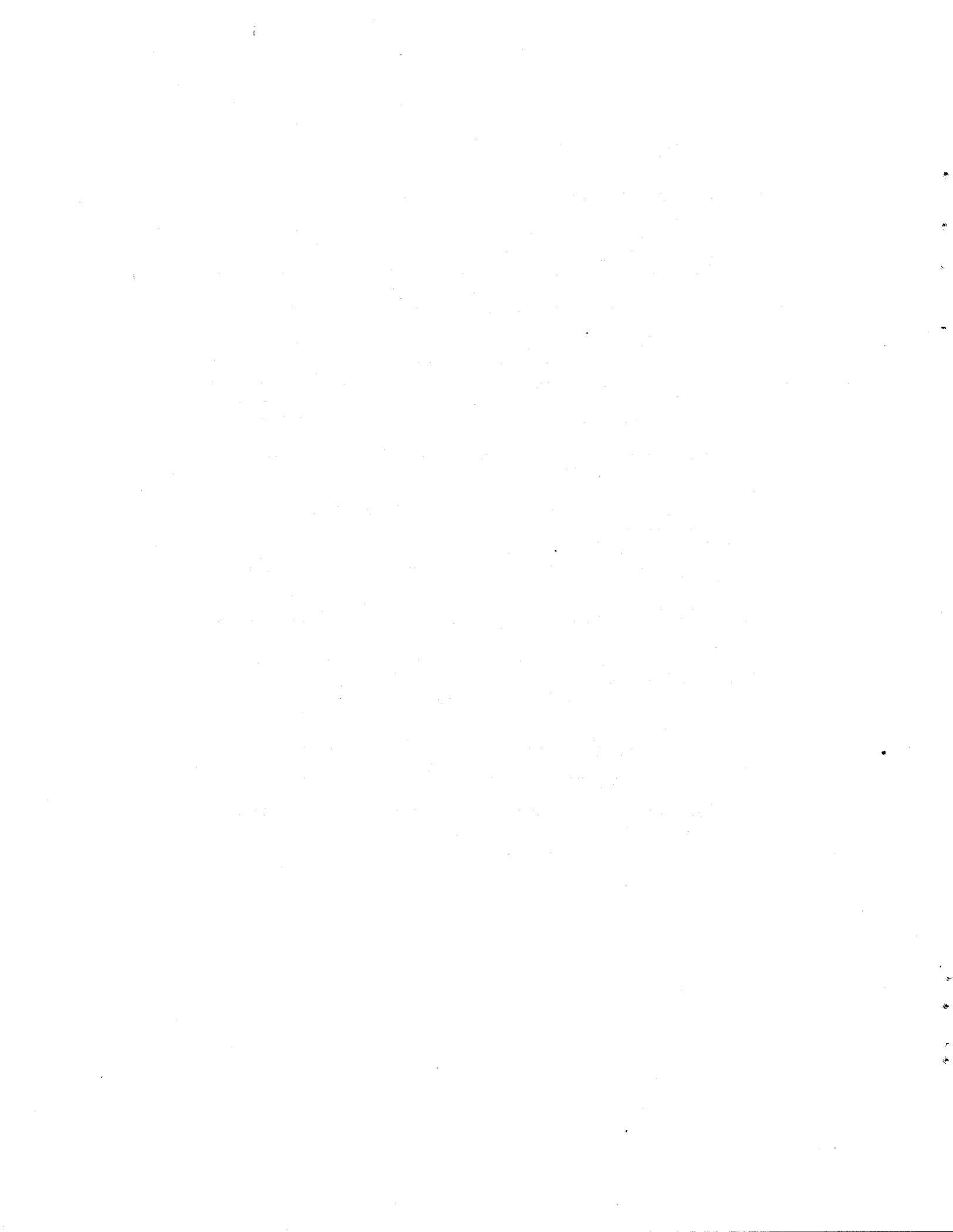
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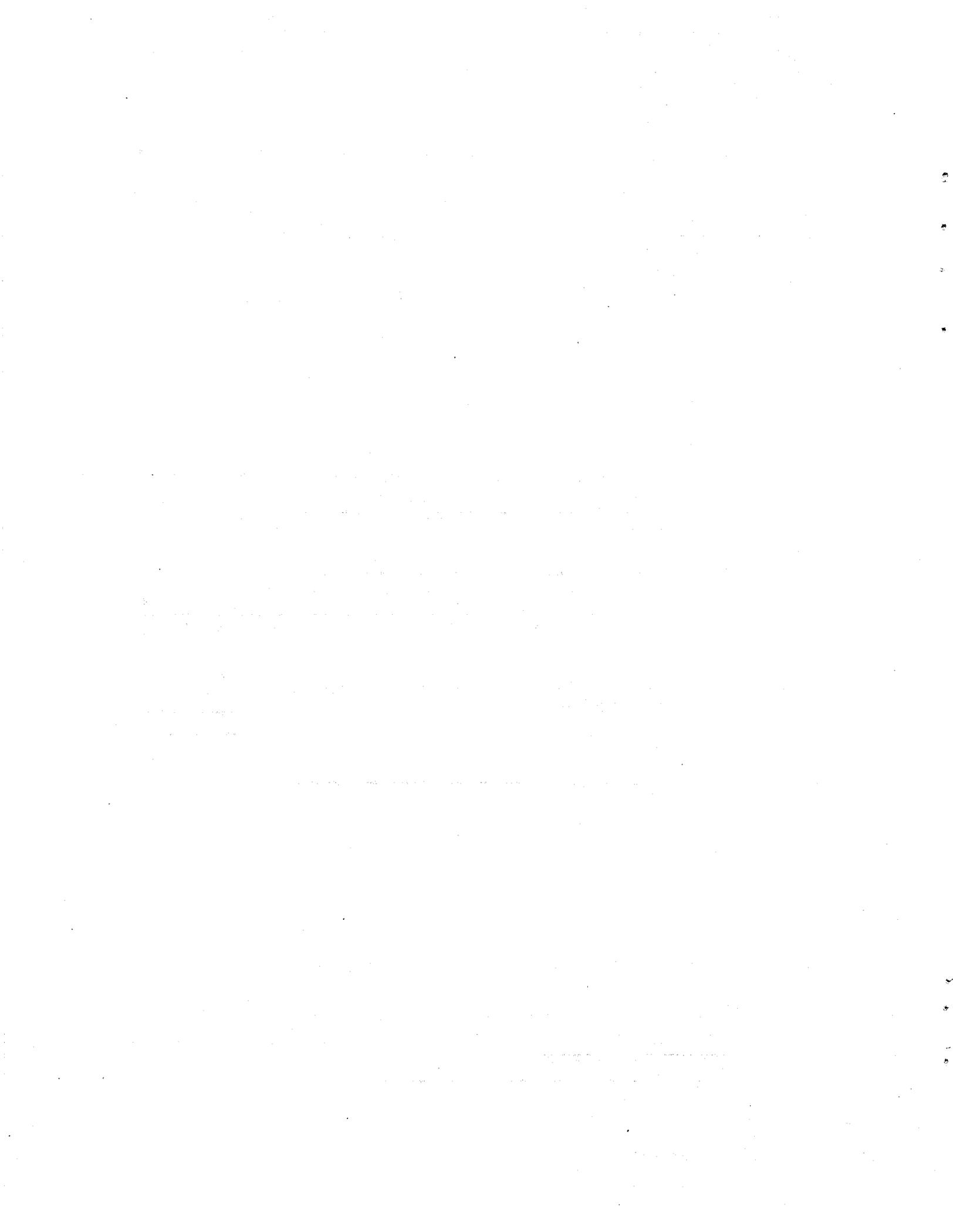
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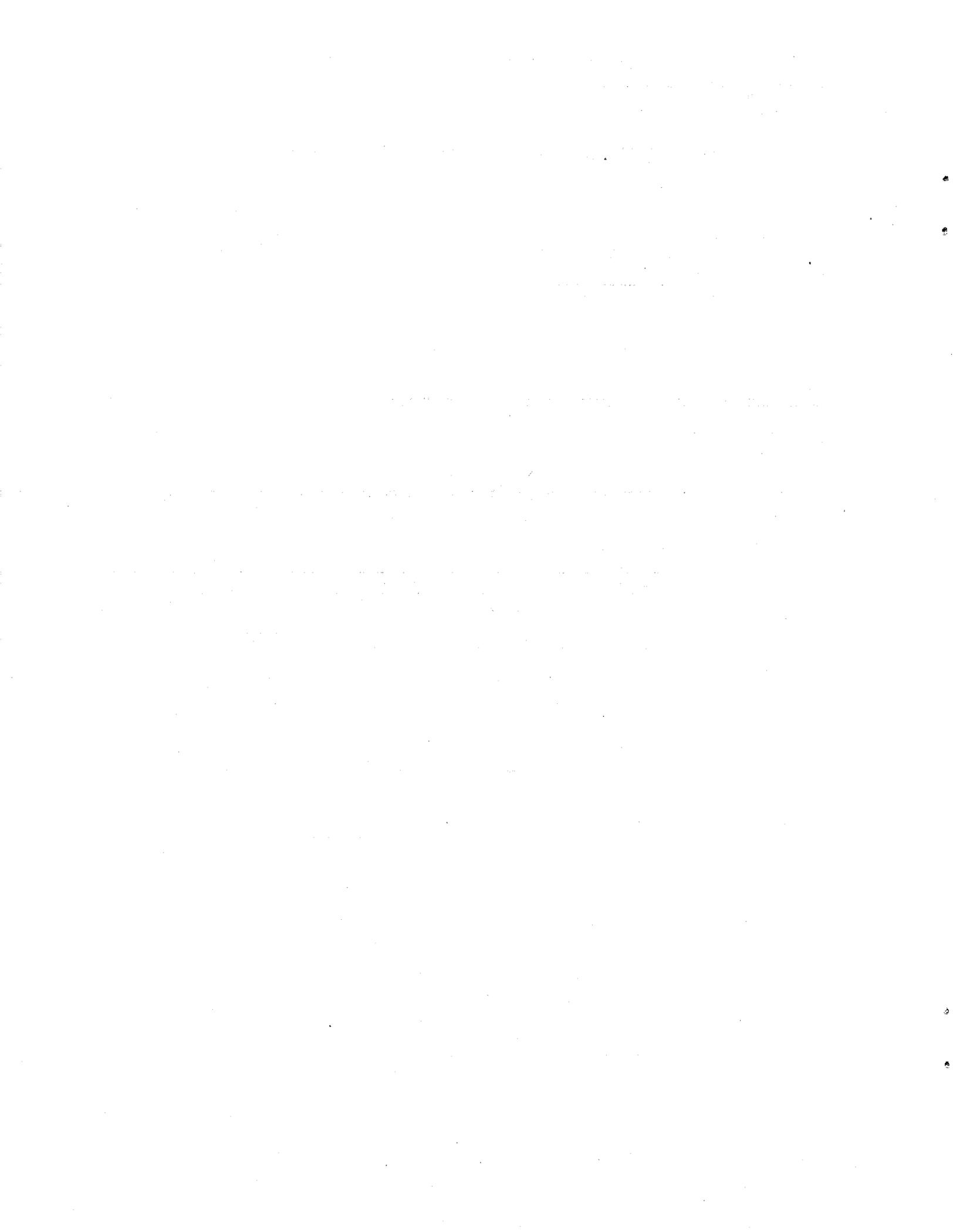
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