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# IN-SERVICE INSPECTION GUIDELINES FOR CONCRETE STRUCTURES IN NUCLEAR POWER PLANTS

C. J. Hookham
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2902 Eisenhower Parkway
Ann Arbor, Michigan 48108

December 1995

ORNL Program Manager: D. J. Naus
Work Performed under ORNL Contract No. 80X-SN811V

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OAK RIDGE NATIONAL LABORATORY
Oak Ridge, Tennessee 37831
managed by
LOCKHEED MARTIN ENERGY SYSTEMS, INC.
for the
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Prepared for the
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Office of Nuclear Regulatory Research
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# IN-SERVICE INSPECTION GUIDELINES FOR CONCRETE STRUCTURES IN NUCLEAR POWER PLANTS

Charles J. Hookham, P.E.
Engineering Consultant

## ABSTRACT

Assurance of the long-term safe performance of concrete structural components of nuclear power plants is highly desired. One means of increasing confidence that satisfactory performance is achieved is through the use of in-service inspection. In-service inspection consists of the periodic evaluation of the condition of a structure through visual inspection, non-destructive testing, destructive testing, and other methods. Early detection and treatment of degraded or damaged nuclear plant structures is especially attractive given their importance, size, and difficulty to repair. Should a concrete structure be observed in a degrading state, in-service inspection can be used to quantitatively define the extent of damage, to monitor for any continuation of the damage, and to assist in decision-making and mitigative efforts. This report examines available inspection methods, their application to nuclear plant concrete structures, and guidelines for establishing acceptance criteria, inspection schedules, and inspector qualifications. The relationship between degradation and its effect on structural integrity and performance is also addressed. The conclusions of this report include recommended guidelines and data for use in establishing in-service inspection programs for structures within the nation's nuclear power generating plants. Two appendices illustrate the development (and update) of an inspection program at an existing nuclear power plant, and recommendations for U.S. Nuclear Regulatory Commission structures inspection, respectively.

**Keywords:** acceptance criteria; aging; concrete (reinforced); degradation mechanism, durability; evaluation, in-service inspection; nondestructive testing; reliability; and structural engineering.

# 1. INTRODUCTION

The service lifetimes and performance characteristics of nuclear power plant (NPP) structures will be affected by the environment in which they exist, as well as by physical loadings to which they are subjected. It is anticipated that the ultimate response of these structures to the effects of environment and load in general will be limited to insignificant material and structural changes over their entire service life. This report examines the in-service inspection (ISI) requirements for NPP structures, with particular focus on those predominantly constructed of concrete. In-service inspection provides a mechanism for quantifying the condition and past performance of a structure at a specific moment in its service life, as well as a means to evaluate any degradation or damage present at that time. In-service inspection also supplies many benefits to the NPP owner (e.g., detection of damage at an early stage prior to necessity for extensive remedial measures, increased assurance that potential future functional and performance requirements will be met, and the associated economic benefits).

Since the advent of nuclear power generation in the United States (U.S.) and other countries, there has been a great amount of attention paid to active plant components that have failed to perform adequately under applied loads and environment. However, little attention has been paid to passive NPP structures. Few operating NPP's have formal inspection programs directed at monitoring the performance of structures, with the exception of containment vessels. Without periodic inspection to identify structural condition, it is difficult to evaluate the reliability of future performance (or lack thereof).

The purpose of this report is to examine current practices for the ISI of structures and cite those of greatest value in assessing the condition of NPP structures. The report also summarizes "available" limits or acceptance criteria for inspection observations in NPP structures. Because of the massive size of many NPP structures such as containment vessels, difficulty in gaining access to structure surfaces due to radiological constraints, existing plant terrain, and economic concerns, this report also necessarily examines inspection prudency and provides recommended contents of an ISI program for NPP structures. A conclusion of Mori and Ellingwood[58] is that an "optimum" inspection program strategy for NPP concrete structures, involving periodic inspections and repair of observed damage, is needed to maintain an appropriate safety factor against failure. Adoption of an optimal inspection program would minimize future costs and extent of inspection while maintaining all limit state probabilities (of failure) for these structures below an acceptable threshold. However, not all effects of the passage of time and service on a structure are negative. Successful past performance can be linked to a higher probability of future reliability. This report is sensitive to the need for <u>optimal</u> ISI guidelines as the cost for comprehensive inspection of all structures is substantial. Additional factors such as the manifestation of degradation mechanisms and their impact on performance, frequency of inspection, influence of repair, and decision-making are addressed.

The overall objectives of this task were to (1) develop a systematic methodology that could be used for making a quantitative assessment of the presence, magnitude, and significance of any environmental stressors or aging factors that could impact the durability of safety-related concrete structures in NPP's, and (2) provide recommended in-service inspection or sampling procedures that can be used to develop the data required for both evaluating the current structural condition as well as trending the performance of these components for use in continued service assessments. This report represents the capstone of Task S.3 of the Structural Aging (SAG) Program (Fig. 1.1), sponsored by the U.S. Nuclear Regulatory Commission (USNRC) and executed by Oak Ridge National Laboratory (ORNL). Previous reports issued under this task and forming the foundation for this report include those by Hookham;[45] Refai and Lim;[70] Sabnis and Kemngang;[73] Hartley and Bamforth;[41] Clifton;[29] Krauss;[49] and Snyder, Clifton, and Carino.[78] These

reports have identified the performance characteristics of typical NPP concrete structures and provided a guideline for ranking the criticality of structures based on degradation, safety significance and environmental exposure; cited current inspection technologies for reinforced and prestressed concrete structures; provided background for development of a damage classification system; developed statistical data for nondestructive examination (NDE) methods commonly used to indicate concrete compressive strength; evaluated techniques for estimating service life of existing concrete structures; and reviewed remedial measures (e.g., repair methods) utilized in the U.S. and Europe. The conclusions and findings of each of the prerequisite reports, as well as other industry documents, have been factored into this report.

As previously noted, the concentration of this report is on structures primarily comprised of reinforced concrete. Many of these structures also were integrally constructed with structural steel members and liner plates. In-service inspection of these metallic components, as well as the balance of safety-related and critical structural steel systems in NPPs, is also warranted. Guidelines for inspection of integral steel components is included herein as their performance also affects the parent concrete structure. However, detailed inspection procedures for structural steel systems is beyond the scope of this document.

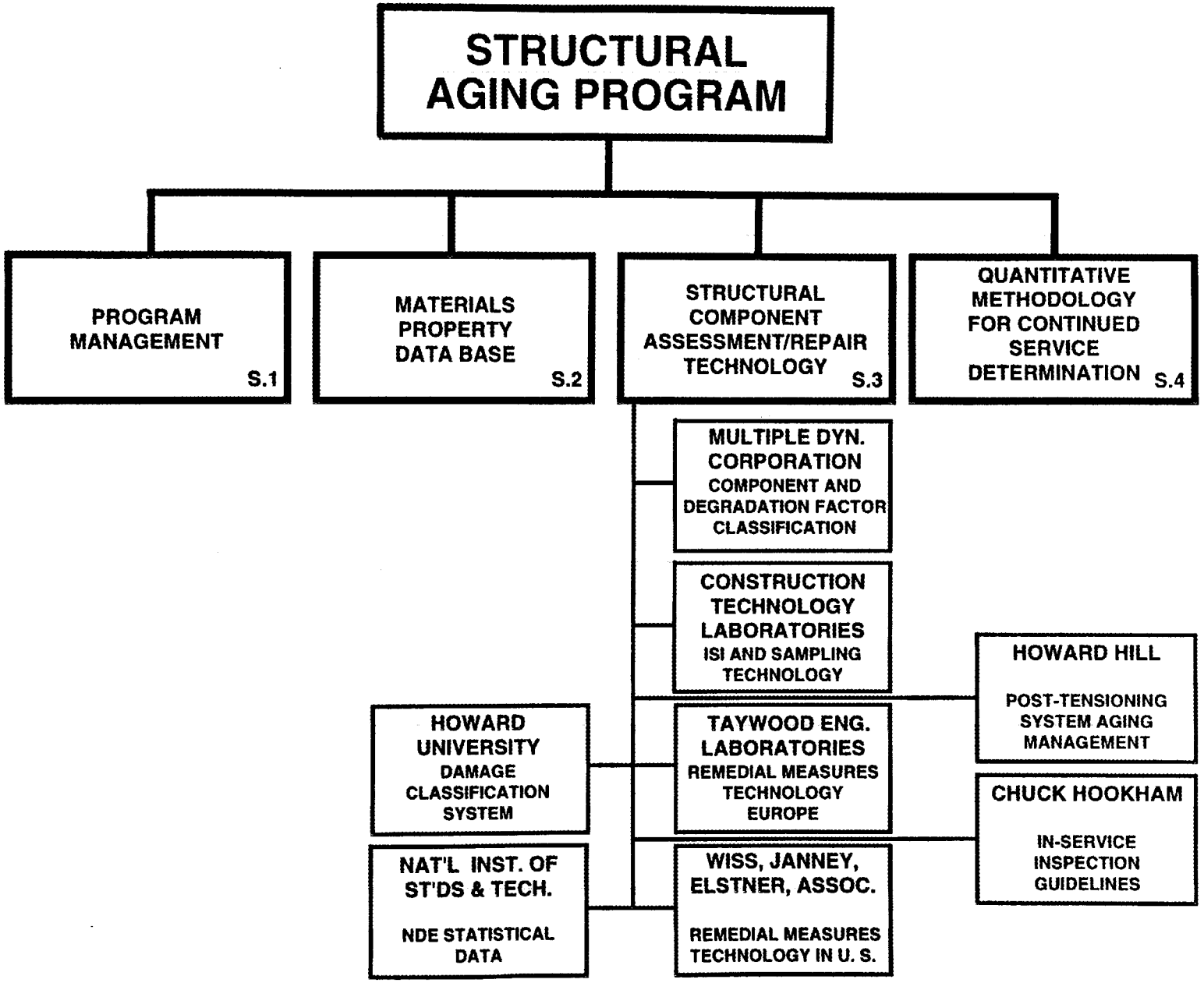



Fig. 1.1 Structural Aging Program overview.

# 2. SCOPE AND INTERFACES

## 2.1 REPORT SCOPE AND PURPOSE

The purpose of this report is to highlight available in-service inspection techniques, their application to nuclear power plant (NPP) structures, appropriate acceptance criteria, inspection scheduling, and qualifications of inspection personnel. Current inspection programs used in industry, including those mandated by the Code of Federal Regulations (10CFR50), are examined in terms of effectiveness and capacity for managing aging and degradation effects. By requirement, this report also visits the effects of existing damage on the load-resisting capacity and performance of structures and attempts to establish threshold levels upon which additional actions (e.g., further evaluation or repair) are required. By nature of its topic, this report parallels current American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code (B&PVC), Section XI[12] and U.S. Nuclear Regulatory Commission (USNRC) Regulatory Guide 1.35 and 1.35.1[82,83] procedures and methods to meet 10CFR50 requirements for the in-service inspection of NPP components. This report also parallels current efforts by American Concrete Institute (ACI) Committee 349,[1] that is preparing a report on structural evaluation and inspection of concrete safety-related structures in NPPs, other than containment structures.

This report assumes that the reader has access to previous Structural Aging (SAG) Program reports and guides and other industry documents, as well as a general understanding of NPP structure configuration, inspection and testing techniques, and material degradation processes. Reference to other sources containing more detailed descriptions of specific test methods, degradation processes, or other information are identified in this report to assist in the establishment of an inspection program and treatment of observed conditions.

This report is <u>not</u> intended to serve as a required inspection guideline, nor should its contents or criteria be used to substitute for current licensing commitments or plant procedures. Because of differences in structural design, plant environmental conditions and other phenomena, use of the information contained herein should not be without review of plant-specific conditions. However, it is anticipated that the technical information contained in this report will be used to establish an inspection procedure and guideline, especially where none currently exist.

The scope of the report includes discussion of relevant methods that may be implemented as part of in-service inspection, acceptance criteria that exist and are applicable to NPP concrete structures; and general guidelines for inspection personnel qualifications and frequency and scope of inspection. This report does not address the aspect of repair procedures; other SAG Program documents provide a description and assessment of materials and techniques that are available for the repair of degraded concrete structures.[49, 69] Similarly, this report focusses on structures predominantly constructed of reinforced concrete and preservation of their function for the remainder of NPP service life.

A paralleling study within the SAG Program focussed on the aging of concrete containment post-tensioning systems.[43] Information relevant to the in-service inspection of these systems was integrated into this report.

## 2.2 OVERVIEW OF CURRENT INSPECTION REQUIREMENTS

The primary goal of an in-service inspection (ISI) program is to ensure that NPP systems, structures, and components are maintained in suitable condition and perform reliably and safely in

future operating periods. Secondarily, ISI provides a means for NPP owners to identify degradation and aging before the severity of the condition impedes overall plant safety or repair becomes an economic issue. Inspection also serves as a means to demonstrate suitable maintenance practices and assurance that structural performance is sustained.

At the present time, only limited in-service inspection requirements exist for NPP structures. Contained in Appendix A of Part 50, "Domestic Licensing of Production and Utilization Facilities," Title 10, *Code of Federal Regulations* (10CFR50), is the requirement that the containment vessel be designed to permit inspection of all important areas and that an appropriate surveillance program be developed (General Design Criterion 53). USNRC Regulatory Guides 1.35[82] and 1.90[84] were prepared to give procedures and methods for in-service inspection of reactor containment prestressed tendon (ungrouted and grouted) systems to meet the 10CFR50 intentions. With the exception of periodic containment leakage testing also specified in 10CFR50 (Appendix J), no other formal requirements exist for in-service inspection of NPP concrete structures.

Subsection IWL of Section XI of the ASME B&PVC[12] provides the rules and requirements for pre-service examination, in-service inspection, and repair of the reinforced concrete and post-tensioning systems of concrete containments (i.e., Class CC). Rules and requirements for similar examinations and inspections of metal containments (i.e., Class MC) and metal liners of Class CC containments are provided in Subsection IWE of Section XI of the ASME B&PVC.[12] At present, neither Subsection IWL nor Subsection IWE has been endorsed by the NRC, but a rule covering these subsections is presently under review. These subsections will likely be endorsed within the next two years, either in partial or complete form, for use and adoption by NPP owners. No requirements exist with respect to in-service inspection of the balance of NPP concrete structures.

Similar to other subsections of the ASME Section XI code in use, both Subsections IWE and IWL define the scope of inspection, techniques, frequency, acceptance criteria, and guidance on repair and replacement. Both subsections also address the containment leakage testing requirements of 10CFR50, Appendix J. Subsection IWL also incorporates current (Regulatory Guide 1.35) containment tendon test requirements. When adopted in whole or part by the USNRC, these subsections will be used by plant owners in performing concrete containment inspections. In the absence of code requirements, few other guidelines exist to aid in establishing an ISI program for concrete structures. Because of this condition, the SAG Program (and this report) has the mission of preparing guidelines to allow <u>prudent</u> implementation of ISI for NPP structures.

An informal survey of existing U.S. plant owners was conducted by the author to determine (1) if plant-specific ISI programs for concrete structures existed; and (2) if such programs existed, how comprehensive were they and were there any common aspects between plants. This informal survey identified that few NPP owners had any type of periodic inspection program for structures, with the exception of procedures to address previously noted containment testing. For those plants having procedures, it was determined that no specific schedule for inspections was maintained and that often the use of such procedures was "triggered" by some upset condition or observed degradation. Most NPP owners concentrated their inspection efforts on <u>active</u> NPP components that had operational problems or a higher degree of failure consequence.

The USNRC also conducts periodic informal inspections of NPPs and concrete structures therein. This document will assist the NRC in prioritizing inspection efforts for concrete structural elements.

# 3. INSPECTION METHODS

Methods or techniques available for the in-service inspection (ISI) of nuclear power plant (NPP) structures range from the relatively "low technology" visual condition survey and inspection to nondestructive testing procedures requiring sophisticated test and data acquisition equipment and highly trained personnel. Although simple in principle, probably the most valuable inspection method for the evaluation of existing NPP concrete structures is the visual condition survey. This nondestructive technique can provide significant quantitative and qualitative data regarding past structural performance and presence of degradation, if the structure in question is accessible. Many aging and degradation mechanisms produce visible indications or discontinuities on exposed concrete surfaces. These indications can be located and evaluated using standard visual inspection techniques, supplemented with enhanced condition survey methods such as crack mapping, and further evaluated with other destructive and non-destructive testing and analytical methods as needed. Using established acceptance criteria, observed indications may be classified as discontinuities (acceptable condition) or defects (conditions requiring further attention). Traditionally, standardization of testing procedures for concrete materials has been conducted by the American Society for Testing and Materials (ASTM)[13] after the test method has been successfully utilized, approved by a consensus process, and verified as producing a specific result.

The long-term durability of nuclear plant concrete structures is related to original quality of materials and construction methods employed; the adequacy of the design; the effects of applied loads from construction, operations, and short-term accidents; the aggressiveness of the surrounding environment; and effects of aging. In general, concrete structures in NPPs were designed to remain elastic and ductile under a wide range of significant short-term load events. As such, the primary concern for their service lifetime is the ability to withstand structural loads (as opposed to other serviceability concerns such as deflection and vibration response). The goal of ISI is to form an opinion regarding condition and performance based on examination of exposed surfaces and within the structure for physical "effects" or conditions leading to a reduction in strength or ductility by degradation factors. Applied inspection methods must be able to address each of these factors and quantify the effects, either at a specific time or over a period of time.

Inspection and testing methods are typically divided into four specific categories:

1. Visual Condition Survey and Inspection;
2. Nondestructive Testing,
   a. Structure-specific, and
   b. Environment-specific;
3. Destructive Testing; and
4. Analytical Methods.

For the purposes of this report, nondestructive testing consists of all methods in which very limited or no physical alteration of the existing structure is needed to properly carry out the test. Destructive testing includes all methods where a sample of the existing structure is removed for either field or laboratory testing, or where significant repair is needed following testing. Analytical methods include advanced calculations and analysis and enhanced testing methods used to assess the adequacy of an existing structure.

Previous Structural Aging (SAG) Program research efforts by Krauss[49] and Refai and Lim[70] reported on available testing technologies for concrete, and to a certain extent, their potential use with respect to NPP structures. Other literature, including ASCE Standard 11-90,[11] have listed available nondestructive examination (NDE) and destructive test methods and their potential use in examining concrete structures. Methods having applications specific to NPP

concrete structures and successful application histories are summarized in this report, as well as those useful in quantifying the aggressiveness of the surrounding environment(s). Analytical tools such as enhanced structural analysis and their potential application in assessing structural condition and impact of existing damage are also addressed.

This report also assesses the "detectability" of each available inspection and test method. That is, the ability and accuracy of each method to detect the presence of discontinuities and damage that may affect structural performance are briefly addressed. Detectability is defined by describing each method's capability and usefulness (probability) to identify the presence of specific damage types in reinforced concrete structures. Comparing the relative detectability of several possible inspection methods for a specific damage type (e.g., delaminations, voids, reinforcing steel corrosion), allows prudent selection of a method for implementation. From a statistical perspective, the detection of damage through inspection and testing may be defined as a non-decreasing function of damage intensity with a range between 0 (no detection) and 1 (perfect detection).[58] This is shown graphically in Fig. 3.1 where several curves represent possible detection functions for methods and damage intensities. In general, none of the available methods have the ability to perfectly detect damage. However, for specific discontinuity or damage types, a method may have significant sensitivity to detect defect presence. For example, the presence of shallow delaminations in a flat floor slab may be of concern. Possible methods of detection include chain drag, impact-echo, infrared thermography, tomography, and concrete core sampling. In terms of detectability, the chain drag, impact-echo and core sampling methods all provide accurate information. However, the impact-echo method allows precise definition of delamination planar boundaries (e.g., this method has a detectability function similar to curve "E" in Fig. 3.1, with $X_{min}$ approaching 0 for significant delamination depths). In addition, other factors such as cost of equipment and testing and improved accuracy support selection of the impact-echo method over other methods for detecting this form of damage.

The capability of a certain method to detect damage is related to the minimum size, orientation, and type of damage present and method sensitivity. As noted in Chap. 4, the detectability is also affected by the size and complexity of NPP structures. Where the minimum level of detection (defect size and orientation) has been documented, it is noted. Acceptance criteria for several inspection/testing methods are proposed in Chap. 5 as a means of limiting degradation and damage to levels below that of concern.

## 3.1 VISUAL CONDITION SURVEY AND INSPECTION

Visual condition survey and inspection encompasses a variety of techniques including direct and indirect inspection of exposed surfaces, crack and discontinuity mapping, physical dimensioning, environmental surveying, and protective coatings review. This technique may provide significant information on the current condition of a concrete structure having accessible surfaces, including the absence or presence and cause of degradation; material deficiencies; performance of metallic liners, coatings and cover concrete; and response of a structure under load or operating status (deflection, vibration, strain, or similar). Typically, visual inspection is the "initial" technique employed to gain a general knowledge of the overall condition. Structures, or components thereof, that are primarily inaccessible without removal of soil or neighboring structures, may require other initial efforts to locally characterize current physical condition. These other techniques may include qualification of the surrounding environment by sampling the air, soil, and groundwater chemistry; examination using other test methods (e.g., nondestructive testing); inspection after local disassembly (concrete or tendon removal); or sampling via concrete coring and laboratory testing. For all structures, additional inspection, testing, and evaluation methods and practices may also be implemented after use of initial methods and review of results.

In terms of detectability, this method has significant sensitivity for most surface-related damage types, but is limited in sensitivity for below-surface concerns (e.g., for surface concerns, the visual method would have a detection probability similar to curves A or E in Fig. 3.1).

Commonly used practices and checklists for the visual inspection and condition survey of existing concrete structures are contained in ACI Report 201.1R-92, *Guide for Making a Condition Survey of Concrete in Service*;[8] ACI 207.3R-94, *Practices for Evaluation of Concrete in Massive Structures for Service Conditions*;[10] and ASCE Standard 11-90, *Guideline for Structural Condition Assessment of Existing Buildings*.[11] Much of the information in these standards also applies to inspection of NPP structures.

Components such as metallic and non-metallic liners, waterproofing materials, protective coatings and membranes, and joint sealants are provided to protect concrete from aggressive fluid and other environmental exposure. Inspection of a structure should characterize the condition of these components as well, with maintenance performed as required to preserve their function. For metallic liners, visual inspection should be augmented with ultrasonic measurement of remaining wall thickness and local NDE of any discontinuities (i.e., damage) observed. For sealants, coatings and membranes, visual inspection will likely provide the best indication of system performance, condition, adhesion, and remaining service life. Should any concerns be identified by the inspection team, vendor representative(s) for the particular material should be contacted for further input.

The scope of the visual inspection should include <u>all</u> exposed surfaces of the structure, joints and joint material; interfacing structures and materials (e.g. abutting soil); embedments; and attached components, such as base plates and anchor bolts. These components should be directly viewed (ideally 600 mm or less focal distance) if possible, with photographs or video images taken of any discontinuities and pertinent findings. Comprehensive direct viewing may require the installation of temporary ladders, platforms, or scaffolding. Use of binoculars, fiberscopes, and other optical aids (indirect visual inspection) is recommended if needed to gain better access to augment the inspection, and to further examine any discontinuities. Such equipment should have similar resolution capabilities, under ambient or enhanced lighting, as gained for direct viewing. The condition of surrounding structures should also be observed to better assess the aggressiveness of the local operating environment or to detect if differential settlement has occurred. Documentation of physical condition and alignment may also be enhanced through the use of close-range photogrammetry.[34] This technique provides a computer file of the mapped surface or geometry for comparison to repeated data gathering in the future (see Sect. 4.5 also).

Visual inspection may also include the use of physical measuring equipment for dimensioning and measuring the size of degraded areas. This equipment should be in good working order and properly calibrated. Calibration may be achieved through statistical review of equipment accuracy via multiple measurements or through formal calibration performed by a certification body such as the National Institute of Standards and Technology (NIST). For crack investigations,[9] a feeler gauge, optical crack comparator, or crack width meter should be used for quantifying the width and depth (if possible). For crack length measurement and general dimensioning purposes, a standard retractable metal tape should provide the desired accuracy.

Limitations of visual inspection techniques include the requirement that the surface of interest be visible, that fine or internal defects are often not detectable, and that quantitative bases and criteria for use in damage assessments are not available. The usefulness of visual techniques as an evaluation tool decreases as the accessibility of the structure's surfaces for inspection decreases (e.g., direct visual inspection is of limited use for evaluation of the containment basemat foundation). The relationship between damage observed in a visual inspection of a concrete

structure and future structural performance is similarly complicated. Upon completion of a visual inspection, the results should be assessed by comparing any findings to standard acceptance criteria. Such criteria, addressed in Chap. 5, are difficult to establish for nuclear structures because of their complex design bases and required response to multiple load combinations. However, use of acceptance criteria is necessary from an inspection perspective to provide a consistent means for reporting observed conditions.

Inability to access all surfaces of a structure reduces the ability to completely verify physical condition and absence of degradation. For structures that are largely inaccessible (e.g., foundations, below-grade walls), this represents a primary concern for use of direct visual inspection as the main inspection tool. In addition, certain degradation mechanisms such as fatigue may manifest and propagate within a structure before any visible signs are displayed. For structures exposed to thermal effects and time-varying or vibratory loads, consideration should be given to supplementing any visual inspections with nondestructive or destructive testing to examine subsurface conditions. For normally inaccessible structures, information on the aggressiveness of the surrounding environment should be obtained (see Sect. 3.2.2). This information will aid in establishing current and future performance of the structure.

Documentation of visual inspection results should include a general description of observed surface conditions, location/size of any significant discontinuities, noted effects of environmental exposure, and presence of degradation. Sketches, photographs, and other means listed in Sect. 4.5 should be used to supplement text descriptions. In the event that additional testing is needed, any limitations on use such as "access to one side only" should be noted on the visual inspection report. Upon completion of a visual inspection, results can be assessed through comparison of findings to standard acceptance criteria if they have been developed. Availability of acceptance criteria are limited, however, because of the difficulty in establishing relationships between pertinent factors such as structural importance, degradation state, and environmental exposure (see Chap. 5). If available, however, these criteria would provide a consistent means for interpreting the condition and integrity of a structure.

## 3.2 NONDESTRUCTIVE TESTING

Nondestructive evaluation (NDE) techniques commonly employ specialized equipment to obtain specific data about the structure in question and, in certain instances, its surrounding environment. The non-homogeneity of concrete, coupled with thick cross sections and typically large quantity and size of reinforcing steel in NPP structures, limit the effectiveness of many nondestructive testing tools. The goal of this type of testing is to provide quantitative information about a structure and its constituents without removing any material. Useful NDE techniques applicable to safety-related concrete structures, with their primary function noted in parenthesis, include:

1. Load testing (structural integrity);
2. Sonic and ultrasonic methods (subsurface concrete cracking/degradation);
3. Microseismic and radar techniques (subsurface concrete degradation);
4. Infrared testing (subsurface concrete voids and large cracks);
5. Modal and vibration analysis, and structural motion monitoring (structural integrity);
6. Acoustic emission and impact methods (subsurface concrete degradation);
7. Magnetic methods (reinforcing steel size, location, and insight on condition);
8. Electrical potential/resistance measurements (reinforcement corrosion);
9. Radiographic methods (reinforcement placement/corrosion activity);
10. Adhesion and holiday testing (coating durability);

11. Surface hardness testing (relative strength determination);
12. Tomography (subsurface voids, cracks, and degradation); and
13. Enhanced visual testing (crack and fracture investigation).

Many of these nondestructive techniques are oriented toward providing specific information about the properties or condition of constituent materials and the presence of internal defects (e.g., voids, cracks, and poor consolidation), as opposed to verifying overall structural integrity. As an example, surface hardness testing (i.e., rebound hammer) may be coupled with visual inspection to examine structures for general condition, but will likely not identify cracking associated with internal degradation mechanisms. Nondestructive examination methods can be grouped according to whether they are structure-specific or environment-specific. NDE methods used to characterize the condition or strength of a structure are structure-specific and are further addressed in Sect. 3.2.1. As previously noted, the longevity of a structure is often directly correlated to the aggressiveness of the surrounding environment. NDE methods used to characterize aggressiveness of the surrounding environment are also considered as nondestructive (environment-specific) and are addressed in Sect. 3.2.2.

### 3.2.1 Structure-Specific NDE

Table 3.1 provides a listing of structure-specific NDE methods for concrete, metal liner and its attachments, and steel reinforcement materials. Also included in the table is a summary of uses, capabilities, and limitations, as well as an assessment of the particular methods applicability to NPP reinforced concrete structures. Several of these methods (e.g., ultrasonics and radar)[70] have been successfully used by the nuclear industry to locate subsurface voids and cracks, both before repair and after to verify repair integrity. However, due to the massive size of many of the NPP concrete structures and extent of steel reinforcement used, application of many available NDE methods is limited. Also, there are constraints related to radiological conditions (e.g., outage and non-outage), cost of testing, reliability of equipment, and expertise required for interpretation of results. Many of the test methods provide an indirect indication of the property or condition of interest, so a valid correlation with a representative calibration standard of similar material is required to relate the measurement obtained to the property. An example is the correlation needed between the rebound number from Schmidt hammer testing and estimated concrete compressive strength. Despite these limitations and the lack of industry-accepted procedures for many of the methods (i.e., especially NDE methods 2–9 listed previously), NDE can be a valuable tool for use in the evaluation of existing structures. When utilized, the NDE method, equipment type(s), calibration records, and a complete description of the procedure (e.g., ASTM test specification) should be documented along with a description of the location where the examination or test was performed. In addition to test results, the names of personnel performing the testing and date(s) of work performed should be recorded.

Structure-specific NDE is generally performed for one of two purposes:

1. Determination of in-place material properties (e.g., concrete compressive strength); or
2. Examination of internal structural condition, continuity, and integrity of the structure (e.g., presence of voids).

Methods used to measure material properties are generally different from those related to condition examination. Methods typically used to address condition of the materials and structure are contained in Table 3.1. Estimation of in-place concrete compressive strength using NDE has been the subject of significant research. [52,53,78]

The "detectability" or confidence in the ability of many of these NDE methods to identify and quantify degradation is still relatively uncertain. Table 3.1 identifies the damage types for which the method has the highest degree of detectability. As continued experience is gained with these and other methods applied to concrete structures, their detectability will be better understood and their usefulness within the nuclear plant inspection program will grow (e.g., their detectability function may change from an insensitive representation like curve "C" in Fig. 3.1 to a sensitive one similar to curve "E", with minimum damage sizes approaching "0").

Use of multiple inspection and testing methods generally provides a more comprehensive means of identifying structural condition and increasing the detectability of damage. The unbonded tendon ISI program at NPPs is an example of a combination of visual, nondestructive, and limited destructive testing. Visual inspection is used to identify the presence of environmental degradation, such as tendon or anchorage corrosion. Nondestructive lift-off load testing is used to measure the remaining prestressing force level in specific tendons. Tendon wires or strands are removed from service and subjected to mechanical testing in the laboratory. The lift-off load testing is an important aspect of the inspection program, as it is used to identify any significant force losses or structural concerns with containment performance. This procedure is primarily limited to containment vessels as few other post-tensioned structures exist in U.S. NPPs that are accessible or configured to support lift-off load tests.

Load testing, as defined in Chap. 20 of the ACI 349 Code,[2] and modal analysis[70] are a special subcategory of nondestructive testing involving the behavior and condition of the entire structure. These methods may provide information about the remaining structural integrity; however, the massive size of safety related structural elements presents significant challenges and obstacles to such testing (see Sect. 3.4 also). ACI Report 437R-89[4] and Rewerts[71] provide additional information on the load testing method.

### 3.2.2 Environment-Specific NDE

As previously discussed, age-related degradation may be the result of exposure to an aggressive operating or natural environment. If surfaces of a structure are accessible, measurements to evaluate the significance of any degradation present can be made directly (i.e., structure-specific NDE methods). Structures or components thereof, that are primarily accessible only after the removal of soil (e.g., basemat foundation), coating or weatherproofing materials, or portions of neighboring structures or liners; however, require an indirect assessment to provide an indication of their current physical condition. One method of indirect assessment is to qualify the aggressiveness of the surrounding environment. This is generally done through an evaluation of the surrounding air, soil, humidity, cooling water, or groundwater. Methods employed are primarily based on chemical evaluations that provide results such as the chloride or sulfate ion content of the groundwater adjacent to the structure. Based on these results, an indication of the potential for degradation to occur and the requirement for additional testing can be determined from known exposure thresholds. Other examples of environment-specific measurements that may be used in a structural assessment include monitoring of thermal and condensation exposures, radiation fluence levels, and the degree and frequency of saturation by water or other fluids.

Table 3.2 provides a listing of test methods for air, soil, and groundwater adjacent to a specific structure of interest. The aggressiveness of cooling water and humidity may be measured using tests similar to those for evaluating groundwater. Sampling methods must be modified accordingly. Where applicable, a pertinent ASTM test method for sampling and testing the particular matter is also cited. Additional information on associated exposure limits is provided in later report sections (see Sects. 5.2 and 5.4). Other methods to evaluate the effectiveness of installed protection devices (e.g., sprayed-on or layered waterproofing, fiberboard, and coating systems) for specific environments are also available or are in the course of development by

organizations such as ASTM. Implementation of an environmental-specific NDE method should be performed by personnel experienced in both field collection or sampling and laboratory equipment use. Documentation requirements specified in Sect. 3.2.1 should also be followed for these NDE methods.

## 3.3 DESTRUCTIVE TESTING

Destructive testing focuses on the removal and testing of material from an existing structure to determine its physical, chemical, and mechanical properties; or to investigate its microstructural condition. Destructive sampling and testing may be applied to reinforced concrete, metal liner, tendon, tendon encasement, and reinforcing steel components of the structure of interest. These tests provide valuable input for the determination of the effects of aging or environmental factors (e.g., exposure to soil, groundwater, radiation, or other potentially deleterious condition) on material performance. Results provide the basis for indicating current condition of a structure and estimating its future service life, particularly if similar tests have been conducted previously. From a "detectability" perspective, destructive testing generally provides a high degree of sensitivity relative to a specific condition. For example, if a void or area of low strength concrete is suspected below the surface, extraction of a core (and compressive strength testing) provides a high degree of confidence on damage presence. Table 3.3 provides a listing of destructive testing methods and pertinent test specifications.

Generally, destructive testing is limited to removal of a "controlled" number of specimens to minimize its impact on overall structural performance. Other limitations include the cost of obtaining and testing samples, potential radiological consequences associated with obtaining samples in areas where contamination may be present, local impact on the sampled structure, and accuracy of the laboratory test methods and equipment. Despite these limitations, destructive testing provides the only means to directly evaluate the properties of concrete and materials that are embedded in, or attached to, concrete.

Destructive testing of concrete samples removed from a structure provides information on its key mechanical properties (e.g., unconfined compressive strength and modulus of elasticity). Other properties and dimensions of interest that may be obtained from removed concrete samples include the concrete's permeability and porosity, concrete cover thickness, presence of deleterious ions, reactivity (e.g., cement-aggregate interactions), condition of underlying or adjacent reinforcing steel, extent of carbonation, and condition of protective coatings. Accelerated testing may also be applied to removed samples to review durability to specific environmental exposures of concern. Petrographic studies[13] of concrete samples provide valuable information related to concrete condition and quality, the cementitious matrix, chemical and physical properties (e.g., entrained air content, presence of ettringite, degree of hydration, and aggregate soundness), and presence of degradation such as microscopic cracking, cement-aggregate reaction, freeze-thaw damage, and chemical attack. Petrographics is performed on prepared specimens from removed cores or other samples, and often involves the use of low and high power microscopes. Presence of deleterious ions, such as chlorides and sulfides, and internal cracking in the matrix or aggregates may be identified. Often, the source of observed cracking can be assessed. Petrographics may be used to assess the performance of concrete exposed to extreme loads or environments, including vibrations, and fire and thermal exposure. Petrographics is also the primary method used to examine for deleterious cement-aggregate reactions and their source; further information on this method is contained in Table 3.4.

Many of the items obtained from destructive testing of concrete are important to the performance of metallic materials contained or embedded in concrete because they contribute to the likelihood of corrosion occurrence (e.g., chloride ion content and depth of carbonation).

Nondestructive methods, such as electrical potential or resistivity measurements that indicate the likelihood of corrosion of embedded steel reinforcement, can be supplemented by confirmatory destructive sampling and testing. Cover concrete can be removed to examine the degree of corrosion and bond characteristics with the surrounding concrete (monolithic behavior).[6,80] Samples of embedded materials can be removed for use in mechanical property determinations. Destructive tests are also conducted on a limited number of wires or strands from the post-tensioning systems in NPPs to determine strength and elongation characteristics. These tests are generally conducted in conformance with ASME Section XI, Subsection IWL,[12] or USNRC Reg. Guide 1.35 requirements.[81] Associated with these tests is the removal of representative samples of corrosive inhibitor (e.g., grease or wax) from the anchorage regions of post-tensioning systems for determination of the presence of water, chlorides, nitrates or sulfides, and the reserve alkalinity.

One indirect and cost-effective method, useful for retrieving core or cube samples, is for the plant owner to retain and identify the source of all concrete removed during plant modifications. This effort must be written into plant modification procedures, including the method for removing and preserving samples, with the individual to receive the material identified. The previously noted tests may then be performed to quantify condition. Destructive testing is especially useful for reviewing the ultimate impact of degradation and establishing the presence of microstructural and internal damage. Destructive mechanical testing and petrographics may provide the greatest insight on future structural performance of the sampled structure.

## 3.4 ANALYTICAL AND OTHER NONDESTRUCTIVE METHODS

Analytical methods involve the use of supplemental calculations or analysis techniques to evaluate the behavior and resistance of the structure. Examples of analytical methods include the use of computer-enhanced stress analysis (e.g., recent developments in static/dynamic linear and nonlinear analysis software), finite-element analysis (FEA), and structural "reanalysis" using ultimate strength design (USD) methods that have been added to the ACI 318 and 349 Codes since the original design phase of many plants. Analysis methods may also be necessary to recreate "design" calculations for a structure in question as original calculations may not be available or the design was governed by calculations for a physically similar but different structure. Reanalysis may be required to reevaluate the capacity of a member if a change in performance requirements (e.g., plant modification) is needed or degradation that could potentially impact structural capacity is found during inspection/testing. A useful analytical exercise for a degraded structure is to perform an independent structural calculation, after determining design-basis resistance requirements and in situ material properties, for comparison to the original structural calculations.[44] This may uncover overconservatisms in the original design, or confirm the need to implement a rehabilitation program.

The role of analytical methods is often to evaluate the structural integrity of a member in its degraded condition and to identify any requirements for rehabilitative techniques, such as strengthening or repair.[3] If results of limited destructive tests are available, analytical methods can be used to estimate the capacity of the existing structure on the basis of "in-situ" concrete and reinforcing steel strength. In-situ concrete strength, in particular, is often higher than the value used in original design calculation. Results from destructive tests can also be used in conjunction with analytical methods that incorporate service life models to estimate the time period required for occurrence of degradation that can eventually lead to a loss in structural capacity or function (e.g., time required for chloride ions to penetrate concrete cover and initiate corrosion of reinforcing

steel). The "acceptance criteria" for use in these evaluations generally would come from the NPP technical specifications and structural codes such as the ACI 349 Code for nuclear safety-related concrete structures.

In addition to the general analytical methods utilized to investigate the integrity of a structure, there are also methods referred to as "loads reconciliation" and "damage tolerance analysis." Loads reconciliation[44] involves the review of actual dead and live loadings that are applied to the structure to determine actual demand. The results of this review are compared to the original design loading combinations to identify structural margins against limit states involved. Any margins that exist are evaluated to ensure that appropriate safety factors are maintained to support the structural function. Excess margin may support allowance of the presence of a limited amount of discontinuities or degradation, along with the associated reduction in structural capacity or function. Damage tolerance analysis is defined by Hookham[46] as "an analytical method to define current and future levels of damage that a structure can withstand before exceedance of strength and serviceability limit states occurs." This method consists of constructing limit state models for specific structural elements such that time-dependent degradation of critical strength or performance values can be technically evaluated. This method draws upon currently available structural analysis methods and limits, as well as system behavior, probabilistic, and life-cycle costing concepts. Various degradation rates and functions are reviewed to assess the integrity of the existing structural member and available rehabilitation options. This methodology has possible application to NPP concrete structures if detailed information including original design load combinations and calculations are available.

Probabilistic methods such as a probabilistic risk assessment (PRA), independent plant examination of external events (IPEEE), and time-dependant reliability analyses may also be useful during an evaluation. Such methods have been applied at several plants relative to seismic concerns and vulnerability to earthquake damage. Conclusions from these evaluations are useful as a means for prioritizing structures and determining the degree of degradation that may be tolerated while meeting functional requirements. Probabilistic concepts are also of value when limited data on material properties, applied loads, etc., are known about a structure. For material property determination, the deterministic sampling requirements of ASTM C 823[13] are generally employed with follow-up laboratory mechanical testing. If sufficient numbers of samples cannot be obtained from a structure to meet the ASTM requirements, Bayesian statistics can be used to improve confidence in results provided from tests involving a limited number of samples. Bayes theorem allows the determination of material properties from a reduced quantity of test data with certain confidence, assuming expected values for the same property or prior knowledge exists.[18] Probability-based analysis and computation of applied load distributions is further addressed in Refs. 48 and 59. The use of probabilistic concepts to support scheduling of inspection/testing activities is briefly addressed in Chap. 6.

Full-scale load testing and strength evaluation of a concrete structure may also be used to characterize performance under applied loads, with or without the presence of degradation, to determine serviceability.[4,27,64,71] This method involves applying a known temporary load (generally static) in a specific position on a structure to produce a specific action (e.g., shear, flexure); an examination of the response in terms of deflection, rotation, strain, or other measurable change is then conducted. The results support decision-making in terms of expected future performance under load. However, as previously noted, the massive size of most NPP concrete structures likely requires use of an elaborate equipment set-up and data acquisition system, and size effects may restrict the usefulness of data that is retrieved. Hall[39] examined the reliability of structures that have survived service exposures and loads without degradation, and response to prior proof loads (full-scale load testing) to express the future behavior in reliability terms. It was concluded from this study that the time-dependent reliability increases, and probability of failure

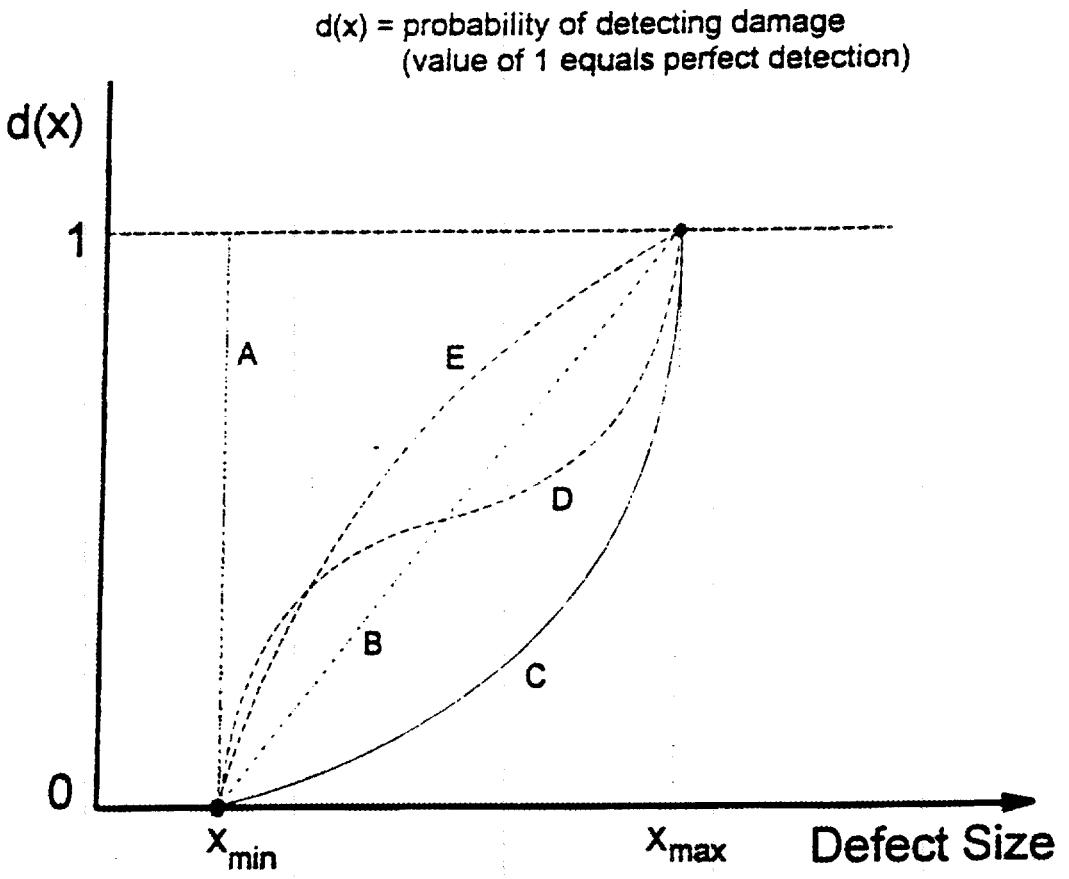
decreases, for structures shown to have an absence of gross errors in design or construction, and are free of degradation, and have survived service loads. The greater the witnessed service load with acceptable response, the higher the reliability of future structural performance. In-service inspection and load testing are both recommended by Hall to support determination of reliability. The methodology proposed has only limited application to safety-related concrete structures, however, because of their generally lightly loaded condition during normal operation and required performance under high-magnitude, short-duration loads from low probability events. However, the combined use of load testing and time-dependant reliability theory may be used in certain instances as a decision-making tool regarding repair vs. replacement.

A number of other inspection techniques are presently under development and may ultimately be used for inspection of NPP structures. These include acoustic tomographic imaging,[17] topography,[40] and magnetic resonance imaging.[51] These techniques involve extension of nondestructive testing techniques developed in other industries for application to civil structures. Their methodology involves introduction of sound or magnetic waves into a structure, collecting and monitoring the internal reflections and wave response, and comparing the results to "calibrated" responses from both undamaged concrete and samples containing various forms of degradation. The challenges with these methods are similar to others such as ultrasonic testing – the massive size of NPP structures, congested reinforcing steel patterns, and accessibility limit the detectability and accuracy of results to the point where the techniques may not be effective.

## 3.5 SUMMARY OF METHODOLOGIES

The selection and use of inspection and testing methods for NPP structures must be based on the characteristics of the specific structure and goals for the inspection (e.g., internal defect survey, reinforcing steel corrosion review, or other), and must be carefully planned and implemented. Selection of the method(s), desired accuracy, and quantity and location of inspections/tests must also occur on a per-structure basis. In general, visual inspection and condition survey should always be included and used where possible to identify needs for further inspection.

The aspect of application of Chap. 3 tools to NPP concrete structures is further addressed in Chap. 4. For NDE or other test methods, use of a statistically significant quantity of tests and adequate test coverage is important to the accuracy of the results. The inspection practices noted in the reference documents, such as ACI Report 364.1R-94[3] and other ACI and industry reports referenced at the end of this report, should be used in the development of inspection procedures. Methods identified previously that appear to have the greatest value for NPP concrete structures include ultrasonics, tomography, petrography, half-cell potential, finite-element analysis, and damage tolerance methods. Ultimately, the acceptance of a degraded or repaired structure must be based on demonstrated and continued ability to meet the original design code and plant licensing commitments. Inspection provides the evidence upon which decisions such as acceptance may be made.




Curves A through E represent different detection capabilities for visual, NDE, and destructive methods for various damage sizes and types. An example would be visual surface crack detection, represented by curve A. All cracks above width, Xmin, would be detected with high probability.

Fig. 3.1. Defect detectability chart.
Adapted from Ref. 59

Table 3.1. Nondestructive testing methods applicable to NPP structures (structure – specific).

Concrete

A. **Surface Hardness and Penetration Resistance Techniques**: Tests including the Windsor probe and Schmidt hammer provide relative strength and hardness information for the tested structure; this information is useful in establishing the overall condition of concrete (strength can be estimated from results).[52,53,60,78] These methods have limited ability to detect damage other than those producing reduction of concrete cover strength. (ASTM Tests C803 and C805 are of this type).

B. **Ultrasonic Methods**: Tests including the pulse-echo, pulse-velocity, and through-transmission methods can identify the presence of internal discontinuities and cracks, provide relative strength and soundness, and give the approximate concrete strength of a tested member.[54,61,74] Sound waves are placed into the structure and the reflection of waves and transit time response are used to identify internal discontinuities. The detectability of these methods is highly variable and dependent on configuration; pulse-echo and pulse-velocity have suitable detectability of voids and cracks in thin, lightly reinforced members. (ASTM C597 is a pulse velocity test method).

C. **Stress Wave, Resonant Frequency, and Impact-Echo Testing**: Test method provides improved understanding of internal concrete structural condition from a single surface. Equipment used ranges from a simple impactor and sound receiving and processing device, to sophisticated spectral and waveform analyzers.[26,54,66,74,75] Similar in principle to ultrasonic methods, the impact-echo method has been successfully used in several NPP structures, including basemats, and many other civil structures and pavements to find voids and other discontinuities. Impact-echo is one of the more promising techniques for minor internal damage with relatively high defect detectability.

D. **Microseismic and Radar Testing**: Short-pulse radar or microseismic electromagnetic waves have been shown to be capable of identifying shallow subsurface discontinuities in concrete structures such as delaminations, the extent of cement hydration, location of reinforcement, member thickness, and water content with reasonable accuracy. However, the equipment is expensive and detectability of other forms of damage is still questionable.[28,54,70]

E. **Acoustic Emission Testing**: This method involves monitoring the "noise" and release of strain energy created by the formation and propagation of defects such as cracks in structures. Its use has been limited primarily to laboratory testing because of difficulties in measuring signals and interpreting their cause in the field. Method can also be applied during load testing. The equipment currently available is also expensive and difficult to transport.[56,70] This method has limited applicability to nuclear structures.

F. **Infrared Thermography**: Infrared waves may be used to identify volumetric subsurface discontinuities by measuring differences in temperature (heat flow). It is particularly useful for finding delaminations and shallow voids that contain air or moisture at different temperatures than the concrete, although this testing cannot easily identify damage depth or thickness dimensions. Equipment is becoming more affordable and is relatively simple to use.[70,87] The method has reasonable detectability, especially for external structures exposed to moisture, that will display greater thermal differences. Minimum damage size detected is greater than for other methods.

Table 3.1. (cont'd)

G. **Tomography**: Testing consists of using acoustic wave or x-ray transmission into a structure to examine for presence of subsurface discontinuities; analysis of the wave form and response is managed via computerized imaging. This technique has demonstrated good resolution and accuracy in the laboratory but has had little field application.[17,70] The acoustic method is judged to have reasonable sensitivity for relatively small internal voids and discontinuities to depths of 460 mm.

H. **Physical Response Monitoring and Testing**: The response of a structure to load may introduce behavior, such as high strain rates, deflections, settlement, or vibration. Structures that demonstrate sensitivity in response to loading may be analyzed through the use of a variety of instruments, gages, motion detectors, and measuring devices. This testing is analogous to static load testing.[4] Detectability of damage and loss-of-integrity is greater for smaller, lightly reinforced concrete structures.

I. **Radiography**: Gamma or x-ray radiation may be used to locate internal defects, such as voids, and reinforcing steel location and general condition through differences in radiation intensity passing through the structure that is captured on photographic film placed on the opposite side of the structure from the source. However, the equipment is expensive and requires special licensing; and this method is not suitable for partially accessible, thick, and heavily reinforced members.[57,70] This method has reasonable detectability for larger internal damage in thin and lightly reinforced members.

J. **Magnetic Resonance Imaging**: This method is based on the interaction between nuclear magnetic dipole moments and a magnetic field, as developed in the medical industry. This interaction is used as the basis for determining the amount of moisture present in a material. Although a prototype system has been developed, only limited results are available and damage detectability is uncertain.[51]

K. **Surface Topography**: This method involves the examination of concrete crack and fracture surface profile through the use of enhanced scanning with a scanning electron microscope and use of Fourier spectral analysis. Although the technology behind this method has a long history, its application to concrete has been slow primarily due to the nonhomogeneous nature of concrete and difficulty with correlating damage observed to cause and significance. Only limited experience has been gained with this method and damage detectability is uncertain.[41]

<u>Steel Liner</u>[76]

A. **Leakage Testing**: Presence of cracks or other thru-section discontinuities may be identified through leak testing, depending on geometry, and if access to liner back surface or leak testing ducts (behind primary welds) is available. Leak testing may be performed with a variety of techniques, ranging from the sophisticated helium mass spectroscopy to simple soap bubble tests. The method has good detectability of damage when applied locally; however, its use is limited to thru-liner damage (ASTM test method E432 provides an overview of various leak testing methods available).

Table 3.1. (cont'd)

B. **Ultrasonic Thickness Testing**: Ultrasonic waves may be passed into a liner plate or attachments to measure thickness based on the principal that sound passes through certain materials with known velocity. This method is useful for quantifying remaining thickness of the liner nondestructively. The method has a high degree of damage detectability to corrosion and planar material loss if suitable procedures are followed and contact surfaces are not heavily damaged. ASTM Method E797, "Measuring Thickness by Manual Ultrasonic Pulse-Echo Contact Method" may be used.

C. **Weld and Base Metal Defect Testing**: To identify the presence and size of weld defects in the NPP liner plate, ultrasonics, liquid penetrant, and magnetic particle test methods can be used. Test procedures have been developed by ASTM and ASME (B&PV Code, Section V). Damage detectability is a function of size and orientation for each method; continued development of the ultrasonics has greatly improved detectability and minimum detectable damage size. Liquid penetrant testing has the least-sensitive detectability function.

D. **Coating Assessment**: Although not addressed in this report, condition of the protective coating system used on a liner plate or concrete structure may be performed with adhesion, holiday, and augmented visual testing methods. Further information on the ISI of protective coatings may be found in the "Protective Coating Standards For Use in Nuclear Power Plants," published by the ASTM. Use of multiple methods can provide a high degree of damage detectability in coating systems.

<u>Embedded Steel</u>

A. **Half-Cell Potential Testing**: This test involves measuring the electrical potential difference between a locally exposed and connected reinforcing steel bar to the neighboring embedded bars through use of a copper-copper sulfate electrode and high impedance voltmeter (ASTM C876). However, this method only indicates the presence of corrosion activity and not the rate of attack.[6,27,80] The damage detectability of the method is greatly enhanced when concrete resistance measurements are also taken and used with reinforcing steel potentials to define corrosion cell locations. Detectability is limited by factors including varying moisture contents and operator inexperience. This method has the highest detectability for conventional reinforcing steel corrosion, short of destructive cover concrete removal.

B. **Linear Polarization Testing**: This method is similar to the half-cell method, only the cyclic potentiodynamic polarization resistance of the embedded steel is measured and an impressed current in the embedded steel is used to compute the shift in potential, corrosion current, and degree of corrosive activity.[49,70] Lack of field experience limits the relative detectability of this method; it is more labor intensive to conduct than the half-cell method.

C. **Magnetic Location Testing**: Using the principles of magnetic induction, flux leakage, or nuclear magnetic resonance, the location, depth, and size of embedded reinforcing steel may be determined. Simple, inexpensive equipment is available for this testing, that is restricted in accuracy by the presence of large size or quantity of steel.[51] The damage detectability of this method is limited, as the minimum damage size identifiable is relatively large (concrete surface damage may already be present).

D. **Radiography:** See discussion under concrete.

**Table 3.2. Nondestructive testing methods applicable to NPP structures (environment – specific).**

## I. AIR

A. Acidity (ASTM Methods D1654, D3815, G50, and G92);

B. Carbon dioxide content (standard methods);

C. Humidity (ASTM Methods D4230 and E337); and

D. Temperature range (standard methods).

## II. SOIL

A. Corrosivity/pH (ASTM Method G51);

B. Oxygen content (ASTM Methods D888 and D4646);

C. Microorganisms and bacteria (ASTM Method D4412);

D. Sulfate and chloride ion content (ASTM D4542 and standard chemical testing);

E. Resistivity (ASTM Method G57); and

F. Moisture content (ASTM Methods D2216 and D3017).

## III. GROUNDWATER*

A. Water table elevation and sampling (ASTM Methods D512, D1293, and D4448);

B. Corrosivity (ASTM Methods D1067, D1293, and E70);

C. Hydrostatic pressure (standard methods);

D. Dissolved oxygen content (ASTM D888); and

E. Microorganisms and bacteria (ASTM D4412, adapted for fluid sample).

* Methods noted may also be applicable to cooling water and other fluid exposures.

**Table 3.3. Destructive sampling & testing methods applicable to NPP structures.**

The following sampling and testing methods are often employed when additional information, unavailable from visual or NDE techniques, is needed to better characterize the condition of a concrete structure. These techniques are often used to augment ISI activities and generally have a high degree of damage detectability when used for the intended purpose.

I. CONCRETE & REINFORCING STEEL (*)

A. **Core Sampling and Laboratory Mechanical, Petrographic, and Chemical Testing** (ASTM C 42, C 823);

B. **Local Static or Dynamic Load Testing (to failure)** (ACI 437R);

C. **Reinforcing Steel Excavation, and Laboratory Mechanical and Metallurgical Testing;**

D. **Powder Sampling and Sulfide or Chloride Ion Testing** (powder samples taken via drilling into concrete to varying depths from surface for determination of deleterious ion content, ASTM C 1152);

E. **Miniature Core Sampling and Depth-of-Carbonation Testing** (phenolphthalein etching of miniature core obtained from cover concrete identifies depth of carbonation from surface);

F. **Tendon Removal and Laboratory Mechanical and Metallurgical Assessment** (see USNRC Regulatory Guide 1.35 and ASME Section XI, Subsection IWL for background information);

G. **Tendon Grease or Grout Removal and Laboratory Chemical Testing** (chemical screening for presence of water, chlorides, nitrates, sulfides, and reserve alkalinity – ASTM D95, D512, D4327, D992, and D3867, APHA 427; and ASTM D974 procedures may be used); and

H. **Coating Sampling and Laboratory Chemical Testing** (analysis of past performance, source of damage, and integrity of existing protective coatings per ASTM test methods listed in "Protective Coating Standards For Use in Nuclear Power Plants" and criteria listed in II.C. below).

II. LINER / EMBEDMENTS(*)

A. **Core Sampling and Laboratory Testing** (removal of liner core or plug sample for laboratory mechanical and metallurgical testing, and access to substrate concrete for inspection and testing);

B **Destructive Strength Testing (anchorages, embedments)** (in-place tensile or shear tests to establish actual properties at plant); and

C. **Coating Sampling and Laboratory Testing** (analysis of performance and quality; ANSI N5.9, N101.2, and N101.4, USNRC Regulatory Guide 1.54, and numerous ASTM test methods).

(*) See Table 3.4 for material testing on "destructively" removed samples.

**Table 3.4. Petrography and material tests.**

## I. PETROGRAPHIC TESTING ON CONCRETE SAMPLES

Petrographic analysis includes a variety of laboratory tests that have the purpose of indicating the durability, physical condition, and soundness of as-built concrete. These methods may also be used to characterize original mixture proportions, type of aggregate, and likely durability. The following list identifies specific tests and their use:

A. **Light Microscopy** (air void system, aggregate soundness, cement hydration, water/cement ratio, stress level, and degradation type);
B. **X-Ray Diffraction** (extent of hydration and type of aggregate);
C. **X-Ray Spectroscopy** (approximate cement content and impurity content);
D. **X-Ray Radiography** (cracking and stress patterns);
E. **Wet Chemical Analysis** (cement content, chemical constituents);
F. **Electron Microscopy** (type of aggregate, presence of degradation or distress); and
G. **Infrared Spectroscopy** (presence of organic material).

## II. MATERIAL TESTS ON CONCRETE AND METALS (*)

The following tests are typically used to quantify in-situ material condition, mechanical property, or other relevant data needed to characterize the sampled structure. These methods have generally been standardized by ASTM. Of particular importance are the concrete compressive strength (ASTM C39 and C42) and metal tensile yield strength (ASTM A370). This list is not exhaustive:

### A. Concrete

1. Compressive strength,
2. Splitting-tensile strength,
3. Modulus of elasticity,
4. Permeability/porosity, and
5. Chemical testing (chlorides, sulfides, etc.).

### B. Metals

1. Ultimate or yield strength,
2. Elongation,
3. Toughness (brittle behavior), and
4. Corrosion susceptibility.

# 4. APPLICATION OF INSPECTION METHODS

As noted in Chap. 3, application of available visual, destructive, and nondestructive testing techniques to nuclear power plant (NPP) structures is limited by accessibility, effectiveness, cost, and other factors. This chapter further examines these limitations, with past experience by the author and others used for illustration. Also presented are an overview of each structural type, most effective inspection techniques for each, and considerations addressing the global issue of inspection program documentation.

## 4.1 NUCLEAR POWER PLANT STRUCTURES

Reinforced concrete has played a prominent role in the construction of all U.S. light-water reactor nuclear plants, both as a structural and radiation containment material. Table 4.1 contains a list of typical concrete structures at NPPs, including both safety-related structures and other common and shielding structures.[45,62,65,66] Safety-related structures are those essential to the function of the safety class systems or components, or that house, support, or protect safety class systems or components, and whose failure could lead to loss of function of the safety class system and components housed supported, or protected. A previous report[45] further defines safety-significance of structures, and their design bases. Due to their passive nature, detailed in-service inspections (ISIs) specifically addressing these structures and performance assessments generally have not been performed. However, as these structures age and are exposed to various operating and natural environments, the potential for degradation increases. The goal of ISI is to identify such degradation at an early stage before the integrity of the NPP is challenged and before significant repairs are required.

Traditionally, NPP ISI programs have focussed on the periodic inspection of safety-related components at various intervals and frequencies depending primarily on component importance and past performance; however, passive components such as safety-related concrete structures have generally received little attention in overall ISI programs. The containment vessel is generally the only concrete structure included, primarily because of the 10CFR50 requirement to perform periodic leak-rate tests and general visual inspection. The concept of periodic inspection is evident within current concrete post-tensioned containment inspections. Post-tensioning tendons have a specific percentage randomly selected at periodic intervals for testing that includes visual inspections and prestressing force measurements during the operating life of the plant. The quantity of tendons tested and time intervals between tests is dependant on results from the inspections/measurements of prestressing force, tendon integrity, and effectiveness of the corrosion-inhibiting medium.[82–84] The requirements are performance-based in that if no degradation or problems are observed in the first three inspections, the quantity of tendons to be sampled is reduced and testing interval is increased. This same concept of performance-based inspection can be extended to the balance of NPP concrete structures. Regulatory or industry requirements that can be used to form the basis of such an inspection program for concrete structures do not currently exist. The basis for prioritization and selection of structures that are important to the performance of a plant, however, have been previously developed[45] and should be utilized in formulation of the program.

## 4.2 LIMITS OF INSPECTION

A number of limitations exist in NPPs with respect to ISI of concrete structures. These include the prevention of access due to radiation and radioactive contamination, both during normal operations and outages; thermal gradients and gaseous environments [boiling-water reactor (BWR)

containments); massive physical size; and presence of surrounding structures, liners, equipment, and soils. Other limitations are presented by the accuracy of the inspection technique(s) and by the skills and capabilities of the inspection team (see Sect. 7). Few structures are accessible on all surfaces for conduction of ISI. Depending on the extent of inaccessibility, this can result in the requirement for use of indirect methods (e.g., environmental-specific testing) to characterize and anticipate the condition of a structure. Although some NDE methods such as impact-echo and ultrasonics can be used to examine internal conditions and continuity if a reduced number of surfaces are accessible, results obtained from these methods may be of limited use due to other constraints that may be present (e.g., accuracy of equipment or procedure, large quantity of reinforcement). These limits may require the performance of "partial inspections" and necessitate the extrapolation of results to represent the balance of the structure not inspected.

Past experience from ISIs at over 15 domestic and international NPPs has been gained by the author, with additional results extracted from numerous inspection documents from the U.S. and United Kingdom.[30,33,36,37,41,81,88] From this experience, it was often concluded that both physical data from visual inspection of accessible surfaces, local or global NDE data, and environmental quantification information could be used to make suitable judgements on structure condition. As the goal of ISI is to gain confidence in component performance, this approach represents the most practical and efficient means of assessing NPP concrete structures. If more enhanced degradation is suspected or found, additional inspection, testing, and evaluation is required. If a large percentage of the structure is inaccessible, the provisions of Sect. 4.3 should be considered in the ISI program.

## 4.3 INACCESSIBLE STRUCTURES

Partial or totally inaccessible structures require that a different approach and emphasis be taken towards ISI. The most obvious shift in emphasis is away from visual inspection practice towards environmental quantification. A possible inspection approach for a partially accessible structure could consist of the following steps:

1. Perform visual inspection on accessible surfaces;
2. Perform NDE on accessible surfaces, as directed by visual results;
3. Study original design and identify key durability features;
4. Quantify environmental conditions potentially affecting inaccessible portions of the structure;
5. Perform initial review of data to identify any shortcomings; and
6. Conduct further inspection and testing and evaluate the structure's condition using all available data.

In the event the structure is partially accessible and site investigations have found the environmental exposure to be non-aggressive, no further action is probably required. However, if site investigations determine that the environment is potentially aggressive and the structure is inaccessible, it may be necessary to expose a portion of the structure via boring, soil/structure removal, or other effort to evaluate the extent of degradation, if any. Coring through the liner to obtain samples for testing should be discouraged since this can compromise the leak-tight integrity of any element that is part of a pressure-retaining boundary. However, this may be the only means available to investigate concrete condition. It is also unlikely that robotics equipment or a down-hole borescope could be practically applied to NPP structures.

If the environmental exposure is confirmed to be non-aggressive, further inspection may not be required. Partial inspection (and knowledge) may provide enough information to gain an understanding of structural integrity. Similar situations for other inaccessible components in nuclear ISI programs have been handled by plant owners through request for a waiver from inspection from the USNRC.

Surfaces of structures that are completely lined with a metal liner for fluid retention and not part of a pressure boundary may be locally assessed via core sampling through the liner. As the liner is generally not considered as a structural element in design, local liner repair efforts after coring should not be substantial. This form of core sampling could prove quite valuable as the liner condition, liner-to-concrete bond, and below-liner concrete condition (and possibly that of embedded reinforcing steel) are all determinable with a single coring operation.

## 4.4 PRIORITIZATION OF APPLICATION

Use of specific inspection methods and testing techniques must be based on the desired outcome from the testing (e.g., compressive strength), potential manifestation of degradation factors (e.g., cracking or loss of strength), detectability of damage, and the importance of the structure investigated to overall safety (i.e., required accuracy of results obtained). Unfortunately, no universal test method or procedure exists that is capable of quantifying all forms of degradation or for determining all material properties important to aging. Selection of a testing protocol is further complicated due to the limitations associated with each of the techniques noted in Chap. 3. No single method is best suited for acquisition of the desired data. Best results are generally obtained when using two or more techniques in tandem. For example, if internal cracking or voids in a concrete structure are suspected, infrared thermography potentially can be used to identify suspect areas. Ultrasonics can then be used to indicate the extent of voids, and the results can be confirmed through core sampling into the suspected area. Reference [70] and the tables provided in Chap. 3 identify tests and methods and their applicability to NPP concrete structures. Assistance in the selection of a particular technique(s) can be obtained from the six-step process noted in Sect. 4.3 and consultation with a structural engineer or structural materials testing consultant. When possible, visual inspection provides the best method to initially assess the condition of the structure and identify potential limitations and concerns.

Decision-making with respect to which ISI method(s) should be applied to particular structures must be based on accessibility, economics, availability of proper equipment and personnel, desired outcome (e.g., type, accuracy, and detectability), and priority of the situation. For general ISI of non-containment structures, a typical program involving visual inspection, quantification of the aggressiveness of the surounding environment, and local NDE of discontinuities and internal concrete should be comprehensive. If a particular structure is subjected to vibratory loads, characterization of its microstructure (core sampling and petrography) is recommended to identify any fatigue-related cracking at an early stage. If a particular structure is inaccessible, program modifications should follow Sect. 4.3 guidelines. If damage or degradation is observed, further NDE, destructive sampling, and laboratory testing may then be required.

For structures exposed to brackish or sea water (direct contact or spray), chlorides from deicing operations, or an environment conducive to carbonation, further assessment of the reinforcing steel system is warranted. As noted in previous Structural Aging (SAG) Program documents,[45,80] reinforcing corrosion is a primary degradation mechanism for NPP structures and early detection is important. An example of a structure suffering from corrosion is the San Onofre intake structure that has required substantial repair after less than 20 years service.[38,81] One of the most commonly used methods to detect the presence of corrosion of reinforcing steel embedded in concrete is the half-cell potential method.[13] The time required for the onset of

corrosion of steel reinforcement in concrete can be estimated through periodic chloride ion profiling (powder sampling) and carbonation depth study (taking miniature core samples and applying phenolphthalein to surfaces). The time rate of permeation of chloride or carbonation "fronts" into the cover concrete can also be established by this approach.

## 4.5 DOCUMENTATION

Documentation of the ISI program for concrete structures, including procedures, schedules, and results should be carefully integrated with the existing plant inspection program in terms of written format and storage method (e.g., hard copy, diskette, or magnetic tape). Accessibility to previous inspection results is especially important to trend performance over a period of time. Direct comparison of inspection results from two different times in the life of the structure is useful in demonstrating structural reliability. The media used for storage is equally important because of longevity concerns, retrievability, space required, and ledgeability. It is now possible, with relatively high resolution, to perform and record visual inspections with a video camera, optical scanning and storage equipment, and remote computer monitor. This technology is especially attractive for monitoring the growth of discontinuities when used with a measuring grid affixed to or marked on the surface proximate to the discontinuity. In addition, most nondestructive and destructive test methods can be linked electronically to data acquisition systems that record results on strip charts (hard copy) or computer diskette.

It is recommended that ISI results be maintained for the life of the NPP. Inspection results are valuable to the plant as a means of demonstrating suitable performance, and as a basis for license renewal and maintenance effectiveness. To minimize the cost of this effort, the inspection program should contain explicit instructions on how results are to be documented and integrated with other inspection program formats, define who is responsible for the program, and note the storage location for records.

**Table 4.1. Typical NPP concrete structures and their accessibility.**

| Safety Related Concrete Structures | Accessibility for Visual Inspection |
|---|---|
| Primary Containment | |
| Containment dome/roof | Internal liner/complete external |
| Containment foundation/basemat | Internal liner (not embedded) or top surface |
| Walls | Internal liner/ external above grade |
| Containment internal structures | |
| Slabs and walls | Generally accessible |
| Reactor vessel support structure | Typically lined or hard to access |
| Crane support structure | Generally accessible |
| Reactor shield wall (biological) | Typically lined |
| Ice condenser dividing wall (PWR -ice cond) | Lined or hard to access |
| NSSS equipment supports/vault structures | Accessible on several surfaces |
| Weir and vent walls (Mark III - BWR) | Lined with limited access |
| Pool structures (Mark III - BWR) | Lined |
| Diaphragm floor (Mark II - BWR) | Lined with limited access |
| Drywell/wetwell slabs/walls (Mark III – BWR) | Internal liner/partial external access |
| Secondary Containment/Reactor Buildings | |
| Slabs, columns, and walls | Accessible on multiple surfaces |
| Foundation | Top surface |
| Sacrificial shield wall (metal containments) | Internal lined/ external accessible |
| Fuel/Equipment Storage Pools | |
| Walls, slabs, and canals | Internal lined/partial external |
| Other Building Structures (walls, slabs, and others) | |
| Auxiliary building | Generally accessible |
| Control room or building | Generally accessible |
| Diesel generator building | Generally accessible |
| **Plant-Specific Safety Related Structures** | |
| Piping or electrical cable ducts or tunnels | Limited accessibility |
| Radioactive waste storage building | Generally accessible |
| Stacks | Partial internal/external above grade |
| Intake structures | Internal accessible/external above grade and water line |
| Pumping stations | Partially accessible |
| Cooling towers | Accessible above grade |
| Plant discharge structure | Internal accessible/external above grade and water line |
| Emergency cooling water structure | Limited accessibility |
| Dams | External surfaces above waterline |
| Water wells | Limited accessibility |
| Turbine buildings (Category I portions) | Generally accessible |
| **Non-nuclear Structures** | |
| Miscellaneous buildings | Generally accessible |
| Water diversion structures | Accessible above water line |
| Turbine building (non safety-related) | Generally accessible |
| Warehouses | Generally accessible |

# 5. ACCEPTANCE CRITERIA

## 5.1 INTRODUCTION

This chapter provides recommended guidelines for the classification and treatment of conditions or findings that might emanate from an inspection of a nuclear power plant (NPP) reinforced concrete structure. These guidelines focus on "common" conditions that have a higher probability of occurrence and are not meant to be all-inclusive. The guidelines have been developed based on a classification of visual inspection results since visual inspection is a key element of any in-service inspection (ISI) program addressing concrete structures. Few acceptance criteria have been published for nondestructive and destructive test methods. Two formats are used to present the acceptance criteria guidelines.

The first format provides guidelines in terms of specified limiting conditions for various types of degradation, using a "three-tiered" hierarchy similar to that used by ACI Committee 349.[1] Certain conditions are considered "acceptable" if their dimensions or observed effects are not severe and within first tier limits. Observations exceeding the first-tier limits require additional technical evaluation and analysis to validate acceptability of the existing condition. A comparison to second-tier criteria is then performed to determine acceptability. Observations exceeding the second-tier set of quantitative limits require either further technical evaluation and analysis to validate the existing condition, or implementation of a repair action to reestablish structural function. Additional information on the "three-tiered" hierarchy is contained in Sect. 5.3.

The second format for acceptance criteria is based on a pairing of a specific degradation type and results from use of a specific inspection method. The resulting criteria is provided graphically on "damage state" charts. These charts allow the inspection team to quickly determine acceptance and take appropriate actions from test measurements taken. This degradation-based criteria is further defined in Sect. 5.4, with guidance on implementing both formats provided in Sect. 5.6. Recommendations for treating conditions of greater severity are also contained herein.

Acceptance criteria for available nondestructive and destructive testing are not easily quantified and defined because of the lack of past experience data and the many differences in structural configuration. In addition, variations in test procedures, test equipment capabilities, response/results from testing, and physical constraints posed by the size and construction of safety-related structures also pose difficulties. The Evaluation Team must develop specific acceptance criteria for each structure tested and method employed. Use of industry-accepted methods and experienced personnel for conducting these specialized tests are also quite important. For test methods that have enjoyed significant use in industry, acceptance criteria are provided in damage state charts and descriptions contained in Sect. 5.4.

## 5.2 INFLUENCE OF DEGRADATION ON PERFORMANCE

The influence of degradation (e.g., damage) on the performance and function of a concrete structure is widely varied, given the non-homogeneity of the reinforced concrete, available margin to withstand postulated physical loads in a degraded state, size and geometry of the discontinuity, and rate of propagation. This area of structural engineering which includes quantitatively identifying actual structural performance vs. that predicted in the design phase, remains relatively unexplored. Material discontinuities, such as minor impurities in steel and locally poor consolidation or small voids in concrete, are common and generally of little structural significance. However, errors made during construction, such as improper placement of rebars, and the initiation and propagation of various degradation mechanisms may result in loss of function and

inability to support applied loads. Mechanisms such as chemical attack and reinforcing steel corrosion often occur at time-varying rates based on chemical concentration, diffusion and permeation characteristics in concrete, and surface conditions. The damage tolerance level, applied steady-state loads, and physical characteristics of the affected structure are of particular concern when assessing possible influence of these effects. The goal of ISI is to carefully inspect the NPP structures for signs of construction problems or degradation that have the greatest potential for causing significant damage. The mechanisms of primary concern have been cited in previous Structural Aging (SAG) Program documents[45,49,70] and are summarized below on the basis of material affected and potential influence on the structure:

| CONSTITUENT | MECHANISM | STRUCTURAL EFFECT |
|---|---|---|
| Concrete | Chemical Attack | Cracking, erosion, leaching of paste, increased permeability |
| | Thermal Exposure | Cracking, loss of mechanical properties |
| | Irradiation | Cracking, loss of mechanical properties, loss of shielding |
| | Vibration/Fatigue | Cracking, loss of strength and damping |
| | Cement-Aggregate Reaction | Cracking, loss of certain mechanical properties |
| Conventional Reinforcing Steel | Corrosion | Loss of monolithic behavior; loss of strength; cracking and spalling of concrete |
| | Other mechanisms | Loss of bond, change in mechanical properties |
| Prestressed Reinforcing Steel | Corrosion | Loss of section and capacity |
| | Fracture | Loss of capacity, brittle behavior and fracture due to environmental and metallurgical effects |
| | Stress Relaxation | Loss of prestress force (tendon) and compressive state (structure) |
| Liner and Embeds | Corrosion | Loss of section/capacity and bond, reduction in leak-tightness for postulated loads |

Several common characteristics may be observed from this categorization. First, concrete cracking is a very common damage by-product from a large number of degradation mechanisms. Active concrete cracking is difficult to assess in terms of current and future structural behavior and is difficult to repair after extensive propagation. Thus, inspection methods that support the early identification, sizing, and determination of cracking cause and means for propagation in concrete structures are of primary interest for future inspections. Second, the primary concern for all metallic constituents of concrete structures is corrosion and corrosion-related damage. Inspections that identify early signs of corrosion cell initiation and define the means for propagation are similarly valuable.

Design of NPP structures is based on extreme low-probability loading conditions that probably will never be approached. Due to the potential significance of these loadings, however, any damage or degradation present requires an assessment of its significance. The massive size of these structures also creates difficulty when assessing the influence of a single crack or localized reinforcing steel corrosion on performance. The acceptance criteria presented in subsequent

sections of this report provide guidelines for acceptance of minor degradation without requiring remedial actions. Also provided as part of the acceptance criteria are guidelines to identify conditions potentially severe enough to impact structural integrity that require detailed evaluation.

## 5.3 VISUAL ACCEPTANCE CRITERIA

The following three-tiered acceptance criteria were developed paralleling ACI report ACI 349.3R[1] criteria. The tiered acceptance approach was adopted to allow simple and expeditious treatment of minor discontinuities (meeting Sect. 5.3.1 criteria) and to provide guidelines for further evaluation of more significant degradation, termed "defects" (Sect. 5.3.2 and 5.3.3). The basis for these acceptance criteria include both published reports, such as ACI 224.1R[9] for cracking, and experience gained through performance of inspections of concrete structures. These criteria should not be considered as "all-inclusive," nor should they be applied without technical judgement.

### 5.3.1 Acceptance Without Further Evaluation

The following findings from a visual inspection or condition survey are considered "acceptable" without requiring any further evaluation. Definition and pictorial representation of typical forms and indications of concrete degradation may be found in ACI 201.1R.[8] In the event that these criteria are exceeded, or observed conditions are determined by the inspection team as deserving further evaluation, the criteria in Sect. 5.3.2 should be considered. Structures that are partially or completely inaccessible for visual inspections (e.g., lined, coated, or below grade) may require supplemental evaluations as environmental conditions may be present that could cause degradation that would be undetected during a general visual inspection.

1. Concrete Surfaces

   Concrete surfaces that are exposed for inspection and meet the following surface condition attributes are generally acceptable without further inspection:

   a. Absence of leaching and chemical attack;
   b. Absence of abrasion, erosion, and cavitation;
   c. Absence of drummy areas (poorly consolidated, with paste deficiencies);[8]
   d. Popouts and voids less than 20 mm in diameter or equivalent surface area;
   e. Scaling less than 5 mm in depth;
   f. Spalling less than 10 mm in depth and 100 mm in any dimension;
   g. Absence of any signs of corrosion in reinforcing steel system or anchorage components (including concrete staining or spalling);
   h. Passive cracks less than 0.4 mm in maximum width, as measured below any widening at the surface, as caused by abrasion or similar effect ("passive cracks" are defined as those having an absence of recent growth and absence of other degradation mechanisms at the crack – see Sect. 5.3.3);
   i. Absence of excessive deflections, settlements, or other physical movements that may affect structural performance; and
   j. Absence of cement-aggregate reactions, chemical attack, fire damage, or other active degradation mechanism.

2. Concrete Surfaces Lined by Metallic or Plastic Liners

   Concrete structures, with inner surfaces protectively lined with either a metallic or plastic (non-metallic) liner system, are judged to be acceptable under the following criteria:

a. Without Active Leak Detection System

1. Absence of bulges or depressions in liner plate (those that appear age-related as opposed to being created during construction);
2. Absence of corrosion or other liner damage; and
3. Absence of cracking in liner base or weld metal.

b. With Active Leak Detection System

1. No detectable leakage observed in leak detection system;
2. Absence of any liner damage, such as noted in 2(a) above; and
3. Absence of fluid penetration indications by leak chases or other detection system components.

3. Areas Around Embedments in Concrete

The condition of the concrete around embedments is acceptable if the following criteria are met:

a. Concrete surface condition attributes of Criteria 1 above are met;
b. Absence of corrosion of the exposed embedded metal surfaces and corrosion staining around the embedded metal;
c. Absence of detached embedments or loose anchorages; and
d. Absence of degradation due to vibratory loads from piping and other attached equipment.

4. Joints, Coatings and Non-Structural Components

The condition of joints, protective coatings, waterproofing membranes, and other non-structural elements is acceptable if the following criteria are met (further information and criteria on coatings is contained in ASTM's "Protective Coating Standards For Use in Nuclear Power Plants").

a. No signs of separation, environmental degradation, or water in-leakage are present in coatings, joints, or joint sealant material.
b. Loss or degraded areas of coatings are limited in surface area to 4000 square millimeters or less at one area, and 0.01 square meters over the gross surfaces of the structure. This criteria applies to structures which do not serve as a barrier to aggressive chemical flows.
c. Absence of degradation in any waterproofing membrane protecting below-grade concrete surfaces.
d. Non-structural components such as dewatering systems are serving their desired function.

5. Prestressing Steel Systems

Components of a prestressing steel reinforcement system are acceptable if the following minimum visual conditions are met:

a. Absence of active grease or corrosion inhibiting wax leakage on exposed concrete or anchorage surfaces;

b. Absence of pitting corrosion, exfoliation, or surface corrosion producing thinning of section on exposed grease cans, bearing plates, anchorages, or other components;
c. Configuration of anchorage components remains unchanged (per structural drawings);
d. Absence of concrete degradation (see Criteria 1 above) around anchorages;
e. No signs of corroded, broken, or failed prestressing elements; and
f. No loss of prestress below acceptable levels established during the design and construction phases (percent maximum loss)

### 5.3.2 Acceptance After Review

The following findings require review and interpretation by the inspection team in order to judge their acceptability. Such a review involves determining the likely source, activity level, and net effect on the afflicted structure. If the cause and degree of damage is concluded to be acceptable, possible treatments include acceptance as-is, further evaluation using enhanced visual inspection (magnification, etc.), scheduling of a follow-up inspection at a later time, use of testing or other analytical technique for better assessment, or repair. Section 5.3.3 addresses the aspect of further technical evaluation and repair. The review that is conducted at this stage should analytically examine the impact of existing degradation on the performance characteristics of the structure. In addition, the potential for propagation should be considered in the treatment selected. Should progressive degradation or propagation be found to have high potential or increasing rate, the treatment should consider more frequent evaluations of the specific structure or initial repair planning. Accessibility during plant operations should also be considered in decision making.

1. Concrete Surfaces

The following surface conditions shall be reviewed to determine if they are either acceptable, require further evaluation, or require repair. Measurable discontinuities exceeding the quantitative limits should be treated per Sect. 5.3.3:

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

2. Concrete Surfaces Lined by Metallic or Plastic Liners

a. Without Active Leak Detection System
Presence of any condition listed in Criteria 2(a) of Sect. 5.3.1 above shall be further evaluated to determine acceptability; and

b. With Active Leak Detection System
Presence of leakage in excess of amounts and flow rates committed to in the original design or plant Technical Specification will necessitate a root cause investigation and

assessment of the need for follow-up action (i.e., notification to jurisdictional and regulatory authorities). Leakage within the prescribed limits may be acceptable if the source is known and found to be inconsequential.

3. Areas Around Embedments in Concrete

   Presence of any condition listed in Criteria 1, Sect. 5.3.2, shall be further evaluated to determine acceptability.

4. Joints, Coatings, and Non-Structural Components

   Presence of any condition exceeding the limits and descriptions of Criteria 4, Sect. 5.3.1, shall be further evaluated to determine acceptability. Any observation of widespread adhesion/cohesion problems, environmental attack, or poor performance indicators are considered unacceptable.

5. Prestressing Steel Systems

   Presence of corrosion or other condition exceeding the limits and descriptions of Criteria 5, Sect. 5.3.1, shall be further evaluated to determine acceptability.

### 5.3.3 Additional Evaluation

Any condition observed outside of the acceptance criteria of Sects. 5.3.1 and 5.3.2 requires an evaluation to determine the appropriate treatment for the afflicted structure. Such an evaluation will likely require an enhanced employment of nondestructive and destructive testing and analytical methods described in Chap. 3 to better characterize current condition and future behavior. It is also likely that original design records (calculations and structural drawings) will also require retrieval and review as part of this effort. The additional evaluation performed at this stage involves analyzing the ability of the damaged structure to fulfill structural and other functional performance requirements. Should the structure be found acceptable in the current position, acceptability in future years should be examined by extrapolating the rate of degradation at varying rates (such as linear, hyperbolic, and non-linear step functions). Design-basis load combinations should be used to qualify the damaged or repaired structure. In addition, any concrete structure found to be exposed to operational stressors that exceed industry thresholds should be considered for further evaluation. The following listing includes examples of these thresholds that have been appropriately reduced to allow for measurement error and incomplete understanding: [1,45,62]

| STRESSOR: | THRESHOLD: |
|---|---|
| Radiation: | |
| Gamma Dose | $1 \times 10^9$ rad |
| Neutron Fluence | $1 \times 10^{17}$ neutrons per square meter |
| Surface Temperature | 65.6°C |

In the event that a repair is determined as necessary, supporting calculations that demonstrate performance of the repaired structure under design-basis loads are needed. These calculations may require extensive analyses and quantification of in-place material properties. The selection of a repair should follow the guidance provided in Ref. 49.

## 5.4 DEGRADATION-BASED ACCEPTANCE CRITERIA

As noted in Sect. 5.2, the influence of degradation mechanisms on the performance of a concrete structure can be highly variable (ranging from cosmetic to structurally degrading). A primary goal of ISI is to verify the ability of a structure to continue to meet its functional and performance requirements. If degradation is detected, ISI provides primary input for quantifying the extent of existing damage and determining if remedial action is required (e.g., increased frequency of inspection, repair planning, and implementation). Provided below are alternative acceptance criteria for use when degradation is detected through visual inspections, nondestructive examination (NDE), destructive testing, or a combination of these techniques. Damage state charts presented in this report are developed to provide a graphical relationship between recommended actions (e.g., repair) and pertinent factors (e.g., crack width and environmental exposure). As each NPP concrete structure performs a different function and has varying levels of safety significance, establishment of "common" chart values that serve for all structure types is not possible. The limiting values specified, therefore, are conservative with respect to published limits and require careful application to each structure. Five primary degradation mechanisms have been considered in the development of the "damage-state" charts presented in this report: concrete cracking, loss of concrete cover material, corrosion of conventional and prestressed reinforcing steel, loss of prestressing force in prestressed reinforcement, and cement-aggregate reactions in concrete. Damage state charts may also be prepared for other degradation mechanisms; however, the five mechanisms described herein address particular areas of concern for nuclear concrete structures. The acceptability of damage in a structure is a direct function of the affected structure's ability to resist applied design basis loads and meet other limit states. Structures with damage exceeding the limits noted on the charts may still be "acceptable." However, such conditions require further evaluation by the responsible engineer to verify this conclusion.

### 5.4.1 Concrete Cracking

Cracking in concrete is produced by many different factors. Two primary consequences of cracking are loss of strength and increased concrete permeability. The prevalence of cracking that historically has been observed in concrete materials requires its attention in structural design — often, concrete is treated as incapable of resisting tensile force because of propensity for cracking and material weakness under tensile force. Treatment of existing cracking in the structural analysis of an afflicted structure, as in new design, must be with care. The cause for a specific crack is often difficult to identify and should be evaluated by an experienced concrete inspector or structural engineer. Few guidelines are published on the differentiation of crack causes; ACI Reports 201.1R[8] and ACI 224.1R[9] contain limited guidance on this subject.

A number of researchers have attempted to identify maximum allowable crack widths in concrete structures. Table 5.1 summarizes many of the published allowables. In general, these limits were established to reduce the potential for enhanced degradation through ingress of contaminants leading to reinforcing steel corrosion, and to reduce the potential for inadequate structural resistance. The damage state chart relating measured crack width and crack activity to the computed (design) stress level established for NPP concrete structures[1] is presented in Fig. 5.1. Measurement of the crack width should be conducted using either a calibrated crack comparator or feeler gauge and should represent the maximum width measured along the length of the crack. Important to the acceptance of a specific crack are its cause, activity level (e.g., passive or active), depth, and geometric and physical relationship to the underlying reinforcing steel and structural concrete. In addition, presence of efflorescence, water intrusion marks, or corrosion staining along the crack must be considered in any disposition. The inspection should also attempt to quantify each of these parameters, where possible.

The steady-state stress level for primary limit states (e.g., uniaxial compressive or tensile, flexural, and shear) in the affected structure is computed based on "worst-case" dead and live loading combination using either the original design basis method, or ASME B&PVC Section III for containment structures and the ACI 349 Code for other nuclear safety-related structures. The ultimate strength of the existing structure is also computed on the basis of its configuration and generally-accepted concrete analysis methods for each principal limit state. The ratio of steady-state stress to ultimate strength is computed for each state and multiplied by 100 to express the stress level as a percentage. The maximum, or controlling, ratio is used in determining acceptability for measured crack width(s).

The resulting crack width limits in Fig. 5.1 are comparable to those cited in previous chapters and derived from Table 5.1. These limits were established primarily for structures exposed to combined stresses including compression, tension, and shear requiring composite action of concrete and embedded reinforcing steel for resistance. For structures exposed to pure compression (e.g., short gravity columns and low aspect ratio internal walls), these width limits are highly conservative. For structures containing multiple cracks of measurable widths, this approach cannot be directly utilized. Comparison of computed stress to observed cracking damage (e.g., maximum measured crack width in structure) provides a means to limit damage in structures whose performance is critical to safe plant operations. Although the cause for cracking may not be the result of structural demand, future resistance of the cracked structure to applied loads must be examined and quantified. Structures carrying a lower percentage compared to design of applied loadings (and hence, stresses) are allowed to remain as found with slightly larger cracking damage. Cracks exceeding the width/design stress ratio criteria provided in Fig. 5.1 require additional evaluation, such as described in Sect. 5.3.3.

### 5.4.2 Loss of Concrete Cover

Loss of concrete cover material or section can occur due to corrosion of embedded steel, chemically-induced erosion, impact, abrasion and scour, spalling, alkali-aggregate reaction, or other effect. "Cover" concrete is defined as the layer of concrete placed outside of the outer-most reinforcing steel for protection purposes, while "structural" concrete is the balance of load-resisting material. Figure 5.2 presents the damage state chart for a reinforced concrete member that is experiencing planar thickness (or cover) losses from the original external surface. A family of curves is presented comparing the amount of loss to exposure conditions and remaining service life anticipated for the plant (or structure). Four relations between percentage loss of concrete cover and relative age of the structure are provided in the chart, based on a minimum required service life of 40 years. The basis for this comparison is that preservation of cover over reinforcing is needed for long-term function and corrosion prevention. Presence of an aggressive environment coupled with reduced cover may lead to early degradation of the steel. For structures with limited remaining service life, an increased cover loss can be allowed, especially in a moderate environment. Results that fall above the corresponding curve for the environmental condition of interest are considered acceptable, as is limited local loss at form ties, impact points, or attachments. However, significant loss of cover concrete (and structural concrete below) resulting in increased potential for exposure of the steel reinforcement to corrosion and associated loss of net section is unacceptable. The chart has been developed with the assumption that the concrete is of high quality, as is typically found in NPPs, and that the initial cover is adequate to provide protection for the rebar during the desired service life of the concrete (e.g., exceeds code prescribed minimums). Under severe exposures, particularly when extensive cracking is present in the cover concrete, the use of this chart is precluded. Severe exposures include direct contact with sea or brackish water, exposure to strong acids and aggressive chemicals including deicing salts, and aggressive groundwater or soil exposure.

Use of this damage state chart requires that the concrete structure in question be physically accessible to the inspection team for direct measurement of material loss with calibrated rules or other measuring device(s). The damage state chart defines acceptance criteria for a range of section loss cases and provides a consistent means for addressing a damage state that can result from many factors.

### 5.4.3 Conventional Reinforcing Steel Corrosion

Corrosion will predominantly manifest itself in the outer layer of conventional reinforcing steel in NPP concrete structures with the propagation to other steel dependent on the corrosion cell characteristics. The likelihood of corrosion occurrence and its potential impact on the structure is a function of the condition of the concrete cover (e.g., concrete quality, thickness, depth of carbonation, and presence of cracks) and the particular environmental exposure (e.g., presence of chlorides, sulfates, carbon dioxide, and acids). Two damage state charts were prepared to address corrosion of conventional reinforcing steel. Figure 5.3 provides a relationship between observed damage state from visual inspection and severity of exposure conditions. As shown in the chart, the extent of corrective action required increases as the crack width and severity of the environmental exposure increases (e.g., increasing chloride ion content of environment, or significant rate of carbonation).

Figure 5.4 presents a relationship between half-cell potential readings, and associated visual inspection of surface conditions, that represent the state of damage and environmental exposure conditions (e.g., conditions ranging from controlled environment to continuous exposure to high chloride levels). This chart can be used to relate the susceptability of outer steel reinforcement to corrosion and the likelihood for existence of active corrosion from half-cell indications. Access to the structure for evaluation is also integrated into this chart as a means to limit extrapolation of "local" inspection or testing results into conclusions for the complete structure. That is, for structures only partially accessible or inaccessible (e.g., subterranean), additional evaluation is needed through local testing to indicate the presence or lack of corrosion activity.

In general, any corrosion damage observed, unless highly localized or limited to normal oxidation, is considered unacceptable. The damage state charts developed focused on relating visual inspection results and NDE (half-cell) measurements that represent damage due to environmental exposure conditions. Both Figs. 5.3 and 5.4 utilized limited industry-published acceptance criteria to graphically depict threshold levels and recommended actions. Numerous other factors must be considered when implementing these charts, including the extent of damage observed (i.e., percentage of reinforcing steel in a state of active corrosion) and variations in exposure conditions that the specific structure is exposed to (e.g., lower portions of structure may be subterranean while upper portions are exposed to natural environment). The relative importance of corrosion damage to structural performance requires that care be taken in interpreting conditions present.

### 5.4.4 Prestressed Steel Corrosion

Corrosion of prestressing steel can result due to the presence of chlorides, nitrates, sulfides, and water. In addition, the prestressing steel may be vulnerable to stress corrosion cracking if hydrogen is present. Detection of corrosion would most likely occur during scheduled ISIs when samples of tendon wires or strands are removed for visual inspection and mechanical property testing. At this time, visual inspections of the tendon anchorage components and adjacent concrete surfaces are also performed. In addition, samples of the corrosion-inhibiting medium are removed and evaluated for contamination (i.e., water-soluble chlorides, nitrates, sulfides, and reserve alkalinity). Review of bonded tendons for corrosion sensitivity includes visual inspection of anchorage components, inspection of concrete cover condition, and examination of grout for

chloride and halogen content. Any signs of excessive free water, presence of aggressive contaminants, or evidence of corrosion activity in excess of requirements, such as provided in Regulatory Guides 1.35[82] or 1.90,[84] require further evaluation. Due to the significance of the presence of corrosion in prestressing systems and the availability of inspection and acceptance criteria, a damage state chart was not prepared for this condition.

### 5.4.5 Loss of Prestress

Loss of prestress in unbonded tendons is currently monitored through "lift-off" testing conducted on a specified minimum number of randomly selected tendons of each type. The measured force in each tendon is evaluated based on a comparison to a predicted value from the original design. Record-keeping may be done using a chart system similar to the "damage state chart" format previously described. The result from each tendon tested is plotted against exposure time to create an understanding of whether loss of prestress is a primary concern. A representative damage chart for this prestressing loss is contained in Fig. 5.5. This chart may be used for containment vessels or other NPP concrete structures reinforced with prestressing. Individual charts should be used for each tendon group (e.g., vertical, hoop, inverted U-type, and retensioned tendons). Because tendon testing to date has shown force readings with considerable scatter, even within the same plant, it is important to use the complete tested population of each tendon group to interpret results.

To determine acceptability, the measured prestressing force for a tendon is compared to a predicted value, calculated as the documented force during installation less predicted losses over time (as dictated by type, orientation, and configuration of tendon) due to material and concrete structure behavior. The USNRC in Regulatory Guide 1.35.1 specifies a procedure for calculating limits (tolerance bands) for measured prestressing force as a function of time. The measured force is compared to the lower limits to determine if any excessive losses have occurred. While these bands provide upper and lower limits for singular testing events, it is also important to examine the long-term trend of prestressing losses for each tendon type and over the cross section of groups at the specific plant.

Per Ref. 43, the current inspection program mandated by the Regulatory Guides was judged to be adequate for determining the condition of post-tensioning materials and evaluating the effects of conventional degradation mechanisms. The same report has cited some concern over the current practice of using pure lift-off force as a means of determining if minimum prestress force exists. Recommendations from this report to compensate for these concerns are noted below.

Per Dougan,[33] the measured force in a single tendon should be normalized by multiplying a correction factor equal to the ratio of average prestress of tendon population divided by the actual individual tendon force. This "normalized" force is then plotted at its service life using the procedure noted below. It is recommended that two curves be drawn; one for the normalized tendon force and one for actual (minimum) force. This will allow comparison between the results and a better understanding of whether prestressing loss is a concern for the specific plant. Trending of actual measured force will also allow review of the lift-off test procedure for adequacy and consistency.

Individual damage state charts should be prepared for each tendon type (e.g., hoop, longitudinal, and dome) so that loss of prestressing force in specific groups (e.g., 0° to 120° hoop tendons) can be monitored. Each chart would present individual tendon results as well as mean/median results for the group so that groups or individual tendons exhibiting higher than anticipated loss of prestressing force can be identified.

It is believed that through normalizing and use of damage state charts for each type of tendon, accurate representation of actual tendon force loss will result. Extreme care should be taken in the measurement of lift-off force, with "calibrated" feeler gauge, calibrated hydraulic jack, or other physical method used.

### 5.4.6 Cement-Aggregate Reactions

Little research has been conducted on the impact of cement-aggregate reactions on structural integrity. These reactions include the more common alkali-silica reaction (ASR), as well as alkali-carbonate and alkali-silicate reactions. Three requirements are necessary for alkali-aggregate reactions to occur: presence of sufficient alkali; availability of moisture; and presence of reactive silica, silicate, or carbonate material in the aggregate. Expansion and cracking, leading to loss of strength, elasticity, and durability of concrete, can result. One research project[31] had a mission of determining structural impacts of the ASR reaction on various beam and plate elements. A quantitative ranking methodology was developed and is considered applicable to NPP structures. The criteria below were developed based on visual inspection results and petrographic analysis of 50 mm diameter core samples, taken from exposed structural elements.

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

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

No simple analysis methods exist for concrete structures damaged by cement-aggregate reaction; the net effect of reaction may range from relatively insignificant cracking to significant pattern cracking and spalling. Cracking is of concern to durability because the cracks can provide pathways for access of hostile environments into the concrete (e.g., steel reinforcement can be exposed to chloride ions). The criteria noted above should be used to define required actions.

## 5.5 OTHER TESTING METHODS

This section of the report cites guidelines and acceptance criteria that have been developed to aid in the interpretation of NDE results provided by ultrasonic pulse-velocity and half-cell potential testing. As these results were established for general civil engineering structures such as dams, their application to NPP concrete requires the evaluation and approval of the Responsible Engineer. Few acceptance criteria were identified in the literature surveyed for other methods identified in Chap. 3.

### 5.5.1 Ultrasonic Pulse-Velocity Measurement

The following table contains a general comparison between the velocity of acoustic energy (sound) measured through a concrete structure and the perceived quality of construction and condition. This relationship has been provided in a number of publications including the U.S. Army Corps of Engineers, Repair-Evaluation-Maintenance-Rehabilitation (REMR) Program Technical Note CS-ES-1.1, "System for Rapid Assessment of Quality of Concrete in Existing Structures." It is important to note that the specific numeric velocity readings vary with hardened concrete properties such as hardness, density, and aggregate type and distribution, and the velocities noted below are "average" for a normal weight portland cement concrete. Improved accuracy can be gained by "calibrating" the equipment on concrete cylinders cast using a similar mix design to the structure in question, or from removed cores. In general, any measured velocity below 3000 meters per second (mps) should be carefully reviewed for cause.

| Velocity (mps) | Concrete Quality |
|---|---|
| above 4500 | Excellent |
| 3600 to 4500 | Generally Good |
| 3000 to 3600 | Questionable |
| 2100 to 3000 | Generally Poor |
| below 2100 | Very Poor |

Measurement of acoustic velocity is an effective NDE method for evaluating concrete condition in thinner beam, suspended slab, or column structures that do not contain significant reinforcing steel congestion and can be accessed from two sides. In addition to gaining a perspective on concrete quality through velocity measurement, the presence of internal voids, cracking, and delaminations may be identified with this method.

This method is often used as a "quick" and inexpensive check of concrete condition prior to the introduction of more thorough and comprehensive measures (e.g., coring and destructive testing). The testing is generally conducted in accordance with ASTM C597[13] procedures. When calibrated with velocities obtained from core samples with known compressive strength, the method has proven quite accurate and beneficial in many civil structures including dams and waterways.

### 5.5.2 Half-Cell Potential Testing

Although the activity of corrosion in substrate reinforcing steel is contingent on many variables, the half-cell method provides some quantitative means of determining the likelihood of occurrence of corrosion. The primary benefit using this method is that corrosion activity may be detected before visible appearance of cracks and spalling of concrete occur, and mitigative efforts may be started before significant damage results. From past testing experience, the following data is representative based on using a copper-copper sulfate electrode (CSE) and the Numeric Magnitude Technique in ASTM test method C876:[13]

| Measured Potential | Condition of Reinforcing Steel |
|---|---|
| More positive than -0.2 volts | Low Probability of Active Corrosion (90% probability, no corrosion) |
| -0.2 to -0.35 volts | Minimal to No Active Corrosion (uncertain status) |
| More negative than -0.35 | Corrosion Activity Likely (90% probability of corrosion) |

ASTM C876 also notes that these potential ranges may not be appropriate for structures having fully carbonated cover concrete (to outer steel layer) or indoor concrete that has not been exposed to frequent wetting, and results should not be used for comparison of corrosion activity in structures exposed to different moisture levels or oxygen concentrations. If this method is applied to the same structure at several different points in time, the resulting potentials and their differences must be carefully treated because conditions such as concrete moisture content at the time of the test may influence results.

Reinforcing steel bars are often corroded only along specific portions of their total length. As each half-cell reading represents the electrical potential at one point along the length of a continuous reinforcing steel (outer layer) bar, determination of corroded portions may be mapped on the surface of the structure for each bar. A common practice is to use the potential voltage levels measured to denote "contours" or grids on the structure's surface in either 0.1 or 0.05 volt increments. These contours help establish areas of the structure in question that need additional attention. Because test results are influenced by parameters such as concrete moisture content, it is important that confirmatory exposure of the reinforcing steel and visual inspection be performed locally. For half-cell readings more negative than -0.35 volts, a visual inspection of the concrete cover should be performed to determine if corrosion-assisted cracking has initiated.

It has been the author's experience that periodic half-cell testing of a structure can provide an indication or trend of corrosion activity and support a decision to use further destructive testing, or to immediately proceed into a rehabilitation or repair effort. As reinforcing steel continues to corrode, half-cell contour maps also spread and the relative activity or damage may be readily assessed. Similarly, if both the potential voltages for the reinforcing steel and electrical resistance readings of the concrete cover are taken together, the current flow can be better assessed, the presence of corrosion activity may be more accurately determined, and spurious half-cell readings may be dispositioned. Recent improvements in half-cell equipment and computerized data acquisition have also improved the accuracy of testing and ease of operation. Application of the half-cell method requires expertise both in implementing the test method and correctly assessing measurements.

## 5.6 INTERPRETATION OF RESULTS

Observed concrete surface conditions that exceed the acceptance limits provided in Sect. 5.3, or conditions found to be detrimental to the structural or functional integrity as a result of a Sect. 5.4 review, shall be considered unacceptable and in need of further technical evaluation.

In the event that reinforcing system corrosion or another mechanism causing loss of monolithic behavior is noted, care must be taken during the evaluation process to properly characterize the degree of degradation and structural impact. Active cracking, vibration, settlements, or deflections that are observed in a structure must be carefully classified as the source may continue to act or intensify.

There are three primary conclusions that may be made by the inspection team as a result of ISI activities performed:

(1) structure is "acceptable" as-is without further assessment;
(2) structure requires further evaluation, supplemental tests or review after another operating period to classify as "acceptable," or in need of repair or maintenance; and
(3) structure is in need of repair, maintenance, or replacement as a result of inspection results.

The acceptance of a structure that has been evaluated with the aid of nondestructive testing or other analytical method must be granted by the Responsible Engineer. Verification that suitable evaluation techniques were used and that accurate conclusions were reached also should be made by the Responsible Engineer. Because of the technical demands associated with many NDE methods, the absence of industry-wide inspector qualifications, and subjective acceptance criteria, it is important that the Responsible Engineer be well-versed in concrete inspection and testing techniques (see Chap. 7 for further discussion on personnel qualifications).

Further evaluation should consider the use of other inspection, testing or analytical tools previously noted in Chap. 3 to obtain condition, and functional information on the structure(s) in question. At this stage of the evaluation process, reanalysis of structural capacity, and behavior under degraded physical conditions is often necessary. Existing material properties (e.g., mechanical and thermal) should be obtained for use in this reanalysis. Should it be determined that the original design requirements and licensing commitments can no longer be achieved, repair or replacement options must be examined for the affected structure.

Decision making that is performed at this stage of the evaluation must consider a number of factors including desired service life, costs for various corrective actions, accessibility, and desired performance. Any repair, rehabilitation, or corrective action of an unacceptable condition taken should be in accordance with the plant design basis, such as ACI 349 Code requirements.

Fig. 5.1. Damage state chart – concrete cracking in structure.

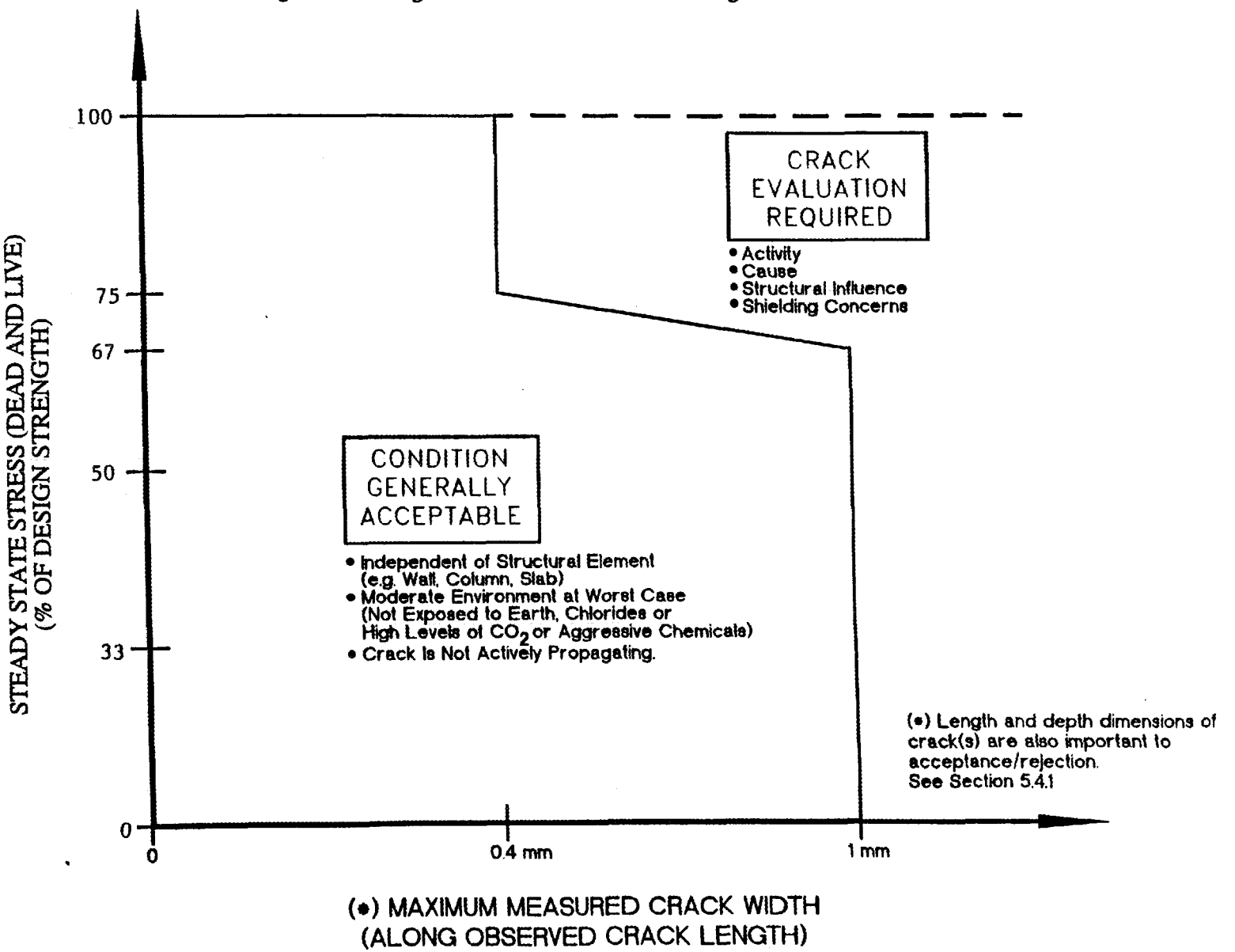
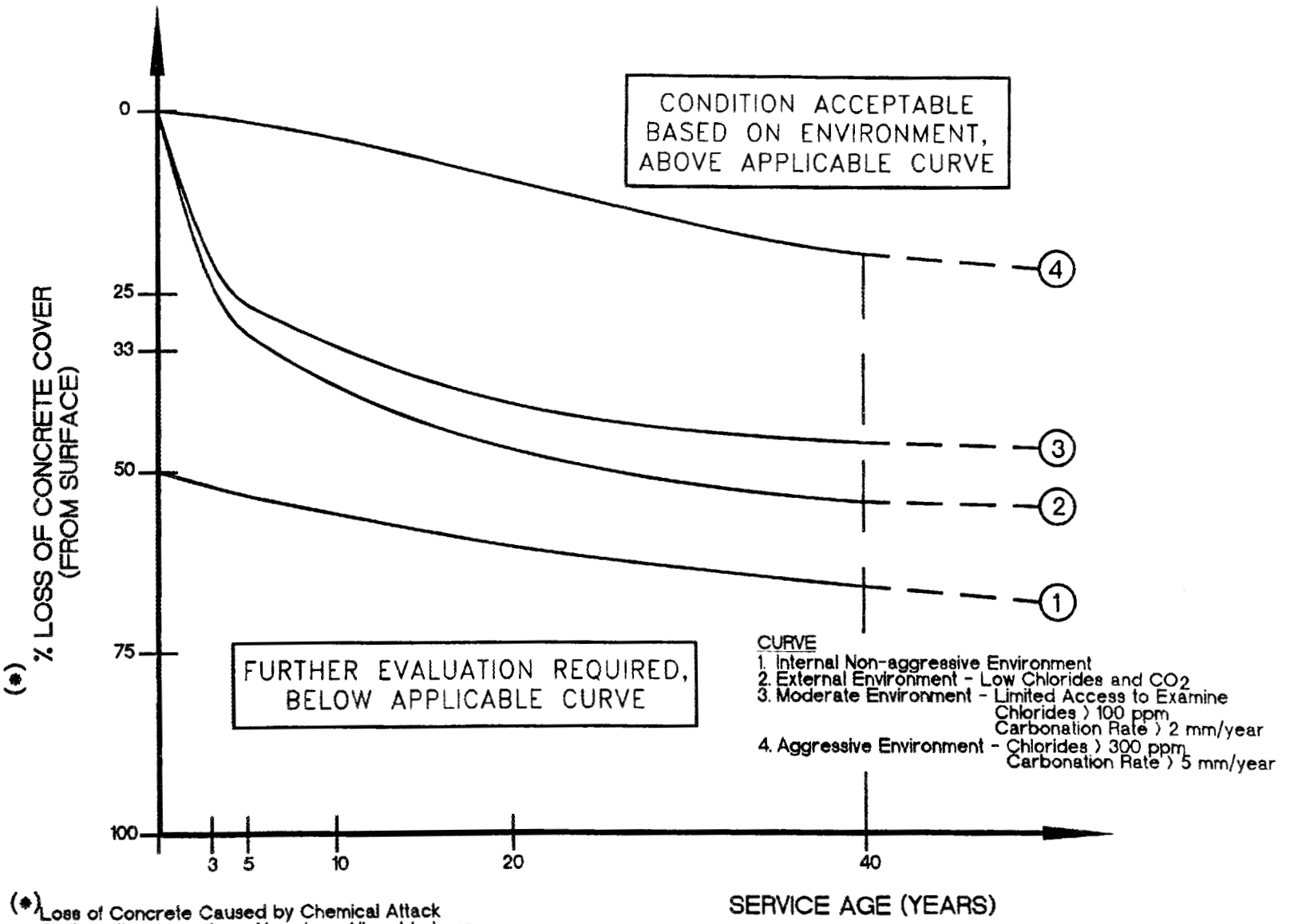


Fig. 5.2. Damage state chart – concrete cover loss.




(*) Loss of Concrete Caused by Chemical Attack Carbonation, Impact, or Abrasion. Allowable Loss May Require Reduction If Original Cover Is Known To Be Less Than Specified Cover Or Code Minimum.
% Loss = As-Measured Cover / Specified Cover (Or Code Minimum) X 100%

Fig. 5.3. Damage state chart – corrosive environment vs cracking in concrete structures

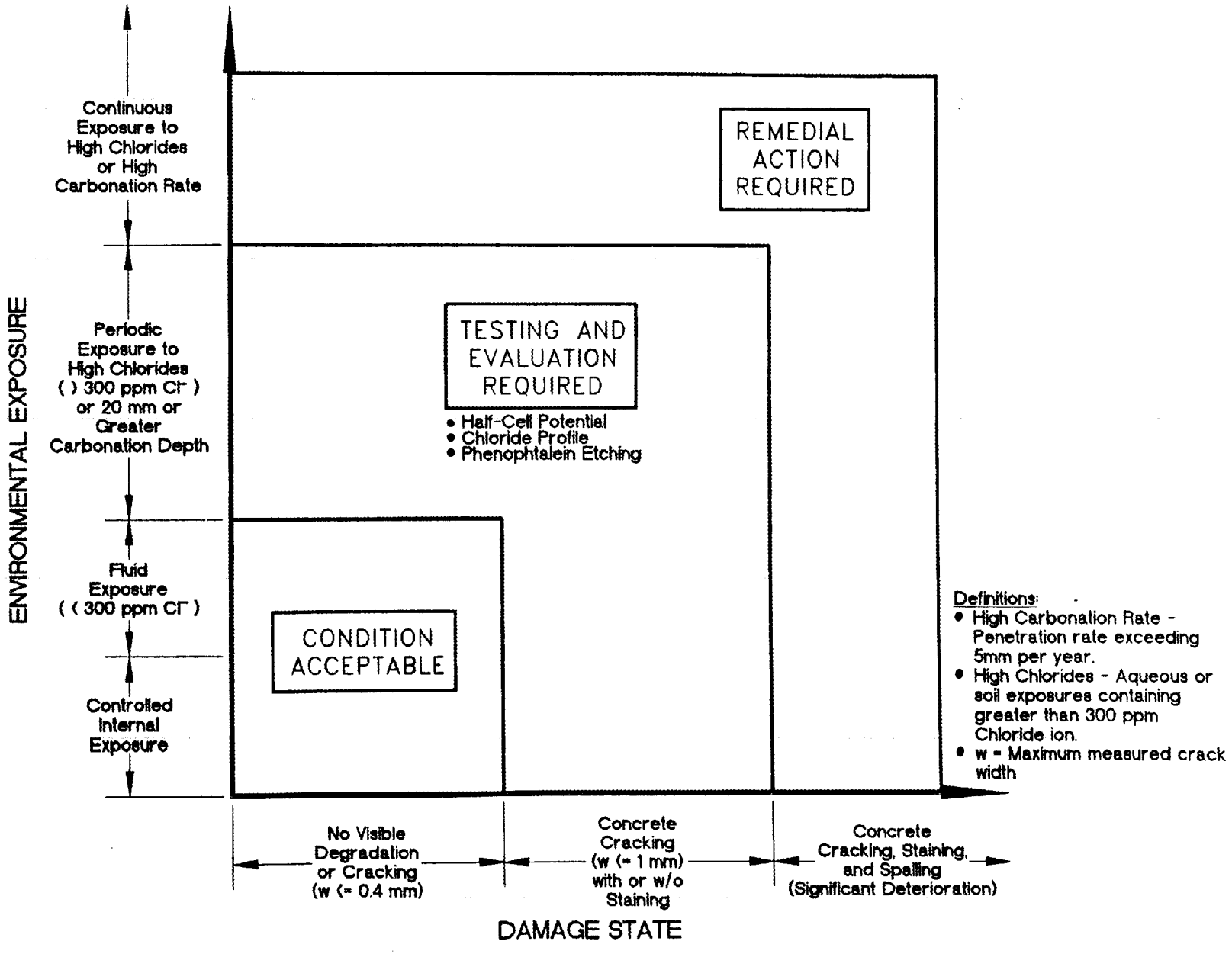


Fig. 5.4. Damage state chart – conventional reinforcing corrosion.

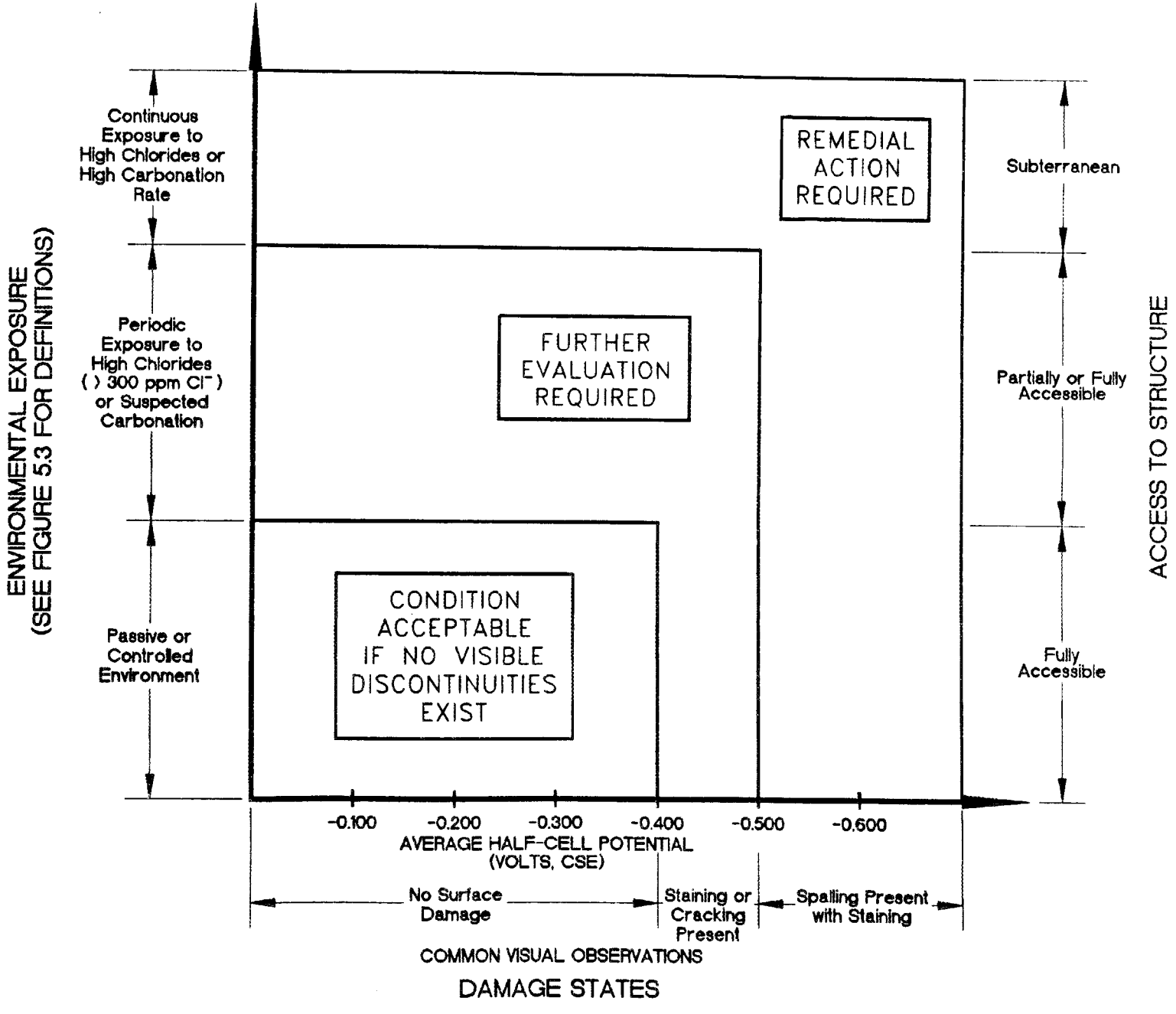
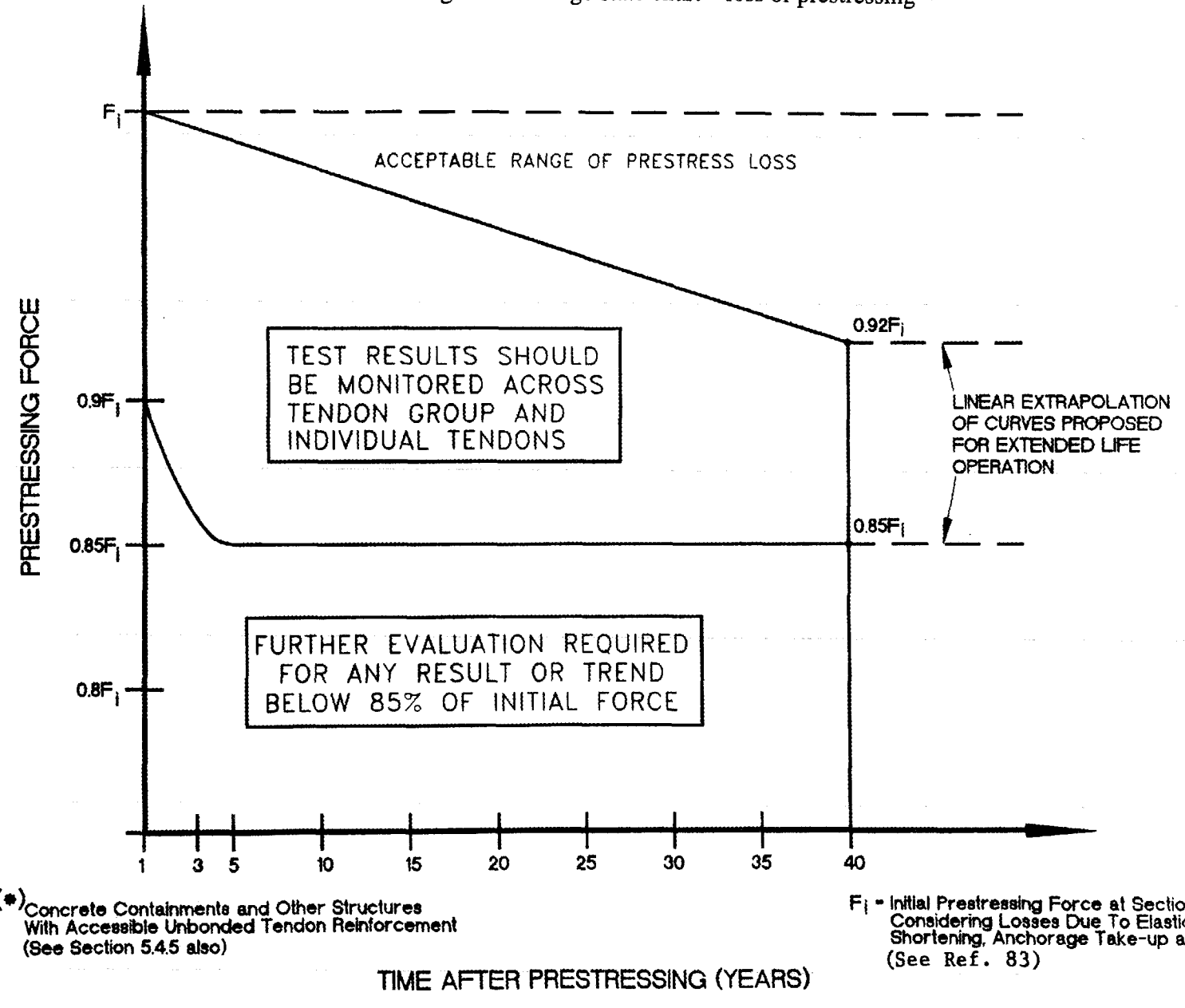


Fig. 5.5 Damage state chart – loss of prestressing(*)




(*) Concrete Containments and Other Structures With Accessible Unbonded Tendon Reinforcement (See Section 5.4.5 also)

$F_i$ = Initial Prestressing Force at Section Considering Losses Due To Elastic Shortening, Anchorage Take-up and Friction. (See Ref. 83)

Table 5.1. Typical acceptance criteria for crack widths (various authors).[49]

| 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 1/2 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 deal 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) |

# 6. INSPECTION SCHEDULING

With the exception of the proposed inspection schedules contained in ASME Section XI and tendon surveillance requirements in the Regulatory Guidelines, no other scheduling guidelines for nuclear power plant (NPP) concrete structures exists. This section of the report summarizes existing scheduling information and considers alternate means of formalizing inspection schedules.

## 6.1 OUTAGE-BASED SCHEDULING

The frequency at which inspections of NPP structures are inspected is defined by the plant owner within the requirements of Title 10, Code of Federal Regulations, and ASME B&PVC. At the present time, only concrete containments, liners, and limited other structures must be inspected. Outage scheduling and planning at the plant may expand if other safety-related concrete structures are added to the scope of work. However, most of the visual inspection work on structures can be performed concurrently with other inspections, and only limited impact on present staffing is likely to be needed. Both current containment and ASME Section XI inspection schedules are considered to be "outage-based" schedules. That is, most or all inspections are performed during planned plant outages (for refueling or other purpose) primarily because of improved access into the containment, reduced radiation and temperature levels, and operating status. These schedules are based on inspecting a percentage of structures during each subsequent "inspection period." The plant owner may schedule all containment inspection efforts to coincide with outages, or elect to perform certain efforts such as tendon surveillances at other occasions within the code-mandated frequencies and schedules.

ACI Report 349.3R[1] also identifies a recommended frequency and schedule for conducting inspections of safety-related NPP concrete structures. These frequencies were based on the relative aggressiveness of environmental conditions and physical exposures of particular plant structures. The established frequencies should also provide assurance that any age-related degradation is detected at an early stage of development and that appropriate mitigative actions can be implemented. In general, it is recommended that all safety-related structures be visually inspected at intervals not to exceed 10 years. In addition, the frequency of inspection for other components should follow those in the table below. For consistency with ASME B&PVC, Section XI in-service inspection (ISI) requirements, the frequencies noted below are alternately expressed in terms of years and inspection intervals as follows:

| STRUCTURE CATEGORY | FREQUENCY OF VISUAL INSPECTION |
|---|---|
| A. Below-Grade Structures | 10 Years (each ISI interval) |
| B. Structures Exposed to Natural Environment (Direct/Indirect) | 5 Years (two per ISI interval) |
| C. Structures Inside Primary Containment | 5 Years (two per ISI interval) |
| D. Continuous Fluid-Exposed Structures | 5 Years (two per ISI interval) |
| E. Structures Retaining Fluid/Pressure | 5 Years (two per ISI interval) |
| F. Controlled Interior Environment | 10 Years (each ISI interval) |

The integrity of any prestressing system employed at a plant, including anchorage hardware, effectiveness of corrosion inhibiting material (grease or grout), and level of prestress in non-containment structures, where feasible, should be assessed at five year intervals.

These frequencies are subject to modification based on specific plant environments or observed degradation that dictate an increased frequency be used. An example would be the observation of microbiological or other corrosion of the steel reinforcement system in structures exposed to brackish or seawater. In the event that degradation is observed in excess of the Chap. 5 criteria, increased visual inspection frequencies or enhanced testing may be required. For structures that are located below grade or are otherwise inaccessible, testing activities oriented at quantifying the aggressiveness of the environment to which the structure is exposed may be performed. For below grade structures, these tests may require examination of soil and groundwater chemistry to assess their potential to cause degradation (aggressiveness to concrete and reinforcing steel). If these environmental tests suggest that an aggressive environment exists, further evaluation at an increased frequency may be warranted.

## 6.2 CONCRETE CONTAINMENT VESSELS

Rules and requirements for ISI of reinforced concrete and post-tensioning systems of concrete containment vessels are provided in ASME B&PVC, Section XI, Subsection IWL. Inspections are performed at 1, 3, and 5 years following the containment structural integrity test, and every 5 years thereafter. Subsection IWL presently includes the scope and extent of inspections for concrete surfaces and unbonded post-tensioning systems. The scope of the visual examination in Subsection IWL is prescribed in two distinct formats; the first level of examination (VT-1C) is conducted to determine the extent of local concrete deterioration and distress. The second examination (VT-3C) is conducted to determine the general structural condition of concrete surfaces of containments, similar to the effort described in ACI 201.1R.[8] Areas and components exempted from the scope of inspection include inaccessible tendon anchorages; portions of concrete covered by the liner, surrounding foundations, or backfill (earth); and those obstructed by adjacent structures, components, parts, or appurtenances. No specific direction is mentioned in these subsections for prioritizing containment components or portions and surfaces thereof. In addition, Subsection IWL does not presently address bonded (grouted) tendon ISIs. Use of Regulatory Guide 1.90 for these tendon systems is to be continued.

The importance of ISI scheduling and aging management of concrete containments was evaluated in Ref. 46. A primary conclusion from this document was that ISI on a scheduled basis can provide assurance that containment performance is suitable and aging is not a concern. Early detection (from scheduled inspections as opposed to random efforts) and prompt treatment of aging effects are also cost-effective and significant to overall safety of the NPP.

As previously noted, a review of ASME Section XI, Subsections IWE and IWL, is presently underway by the U.S. NRC regarding their incorporation into the regulatory process. It is anticipated that future concrete containment inspections will follow Section XI requirements, with some modifications as published in either Title 10 of the Code of Federal Regulations or a U.S. NRC Regulatory Guide. Public comment regarding incorporation of these subsections has been received and is being addressed by the NRC. Upon issue, Subsections IWL and IWE will provide minimum in-service guidelines and frequencies for most containment components. Leak-rate testing would also be continued, with the frequency established in 10CFR50, Appendix J.

Both Subsections IWE and IWL are primarily focussed on the performance of visual inspections as opposed to requiring destructive or nondestructive testing. If degradation is identified beyond limits, the plant owner must perform an "evaluation." This evaluation may be conducted using the technology addressed in this report.

## 6.3 OTHER CONCRETE STRUCTURES

In-service inspection scheduling for other safety-related concrete structures is not included in ASME B&PVC Section XI, or other code documents. Recommended inspection scheduling for these structures has not been addressed by the power industry. Recommended scheduling of inspections should include at least one inspection every 10 years as a minimum. Each plant-specific structure not important to safety should be initially reviewed to determine if it should be included in the inspection program. Use of the tabled values in Sect. 6.1 is recommended for all non-containment concrete structures, especially for below-grade structures exposed to aggressive soils or groundwater. Should signs of degradation be observed, an increase in the frequency of inspection is likely justified. Selection and prioritization of structures to be inspected may be based on the methodology contained in Ref. 45, or purely on the basis of safety significance of the structure to plant operation.

For the safety-related grouping of structures, it is recommended that the ASME Section XI, Subsection IWL scheduling framework be adopted, with increased inspection frequency assigned to those structures exposed to potentially aggressive chemicals or groundwater. Similarly, if an event such as corrosion is observed or suspected, the enhanced scheduling specified in Sect. 6.1 should be implemented. An example of inspection program development and adjustment based on priority is contained in Appendix A.

## 6.4 RELIABILITY-BASED SCHEDULING

The concept of using reliability-based methods to schedule inspections of major structures including off-shore platforms has received significant attention recently in the U.S. Although these concepts are relatively well-refined for active components with operating histories, the Structural Aging (SAG) Program has studied their applicability to concrete structures. Two recent reports by Mori and Ellingwood[58,59] assessed reliability of NPP concrete structures in terms of rate of degradation, inspection detectability, remedial actions, and inspection frequency. Primary conclusions from this research were that optimized strategies for inspection and maintenance could be established that minimize future costs and maintain limit state probabilities below desired threshold levels. Such strategies are sensitive to the relative cost of inspection, repair performance and integrity, the rate of degradation and cost of replacement. Further refinement in costs for inspection, repair, and residual value is needed to verify the the specified statistical models. However, it is expected that an optimal policy for inspecting NPP concrete structures can be established through use of priority and reliability-based methods. Appropriate inspection at interval periods reflective of importance, environmental exposure, access, and damage tolerance is recommended.

Reliability concepts can also be used for defining which structures and elements thereof require the greatest attention (possess the greatest risk of significantly degrading). Time-dependant reliability may be used to expand upon the prioritization method of Ref. 45. Using data extracted from structural inspections and reliability concepts, a prudent program to manage aging in risk-sensitive structures may be established. Preparation of such an inspection program, however, is presently quite difficult because of the number of random variables and influences that are not known with certainty.[58] For example, the detectability function previously considered for

inspection techniques may only be described qualitatively and assignment of an actual probability distribution function or other quantitative measure to detectability has limited basis. Until such shortcomings are resolved, reliability methods should be confined to individual structural assessments and assignment of an inspection frequency.

## 6.5 SCHEDULING SUMMARY

In summary, utilization of a prioritized ranking of NPP structures inspected under a uniform (outage-based) program of ISI will be comprehensive and cost-effective, while preserving plant safety and longevity. Such a program would contain the inspection frequencies presented in Section 6.1 with prioritization of plant structures such as provided in Ref. 45 to formulate an inspection program that would specifically address a particular plant. Should degradation mechanisms and enhanced damage be observed, inspection frequencies should be increased. To reduce the impact of additional inspections on the plant owner, these inspections (outside of the containment) should be scheduled to occur between planned outages, as restricted by access.

# 7. QUALIFICATIONS OF INSPECTION PERSONNEL

The quality and usefulness of results obtained from an inspection of an existing concrete structure are dependant to a great extent on the qualifications and capabilities of the personnel involved. To ensure that an inspection of nuclear power plant (NPP) structures is properly performed, minimum qualifications and skills of the inspection personnel must be defined and enforced. For inspection of concrete construction, the American Concrete Institute (ACI) has a comprehensive inspector training and certification program; similarly, ASME B&PVC Section III defines minimum construction inspector requirements for containments and reactor vessels. Presently, however, there are few standards for qualification and certification of inspectors for visual inspection and testing of existing concrete structures. ASME B&PVC Section XI recently provided a limited definition of inspector qualifications for Class CC nuclear concrete containments and for visual and nondestructive testing of metallic liner and support components. ACI Report 349.3R-95 (draft) contains recommendations for the inspection team for NPP concrete structures with input from other codes; these recommendations form the basis for this chapter. Appropriate credentials are summarized in the following paragraphs.

As a minimum, the complete inspection team should include both civil/structural engineers and concrete inspectors and technicians familiar with concrete aging and degradation mechanisms and long-term performance issues. The qualifications for the person(s) responsible for the in-service inspection (ISI) of safety-related concrete structures should meet the minimum recommendations of the ACI 349.3R-95 report. The individual responsible for the administration of an ACI-defined evaluation is defined as the Responsible Engineer, who will possess one of the following sets of qualifications:

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

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

Personnel performing the balance of inspections or testing at the plant, under the direction of the responsible engineer, should meet one of the following qualification sets, or equivalent:

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

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

C. Personnel meeting the requirements for Level I or II Concrete Inspector, as defined in ASME B&PVC Section III, Division 2, Appendix VII (ACI 359) Code requirements.

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

In-service inspection results may need to be examined in terms of net effect on the afflicted structure. Structural calculations addressing the as-designed structure and projected behavior

under observed degradation should be prepared under the direction of the Responsible Engineer. The civil/structural engineer appointed to prepare these calculations should have credentials similar to those noted above.

The lead inspection engineer may identify a need for nondestructive or laboratory testing to better establish the condition of a structure. Often, plant owners do not have the capability or equipment to carry out such testing. Any subcontracted testing performed as part of an ISI program should be completed by laboratories and personnel experienced and qualified to perform the specific testing. The specialization of testing method procedures and equipment requires that care be used in selecting a contractor. For testing work conducted in a laboratory or for field activities including sample removal and specialized testing, the subcontracted inspection and testing agency should meet the provisions of ASTM Standard Practices C1077 and E329.[15] For other subcontracted activities including nondestructive testing and structural analysis, all personnel should have suitable qualifications, certifications, and registrations to ensure that the results are technically sound.

# 8. SUMMARY

In-service inspection techniques, including visual condition surveying, may be effectively used to examine the condition of accessible nuclear plant concrete structures. As most degradation mechanisms of concern manifest themselves in the cover concrete and outer layer of reinforcing steel, visual inspection may be used to detect and define the severity of any resulting damage. Visual inspection and quantification of environmental conditions are recommended initial inspection steps for all structures; in the event that degradation is observed, visual inspection should be enhanced with NDE and destructive testing to assess the degree of damage. This is particularly important for suspected degradation internal to the structure. NDE techniques that have had the most practical experience and have greatest applicability to nuclear power plant (NPP) structures include pulse-echo and impact-echo ultrasonics, penetrating radar, half-cell corrosion potential screening, tomography, and acoustic emission testing. One significant limitation of all inspection and test methods available for concrete structures is their inability to define, with accuracy, the remaining structural integrity and behavior. The usefulness and application of inspection/test results often requires the judgement of highly experienced engineers familiar with the treatment of damage in either deterministic or stochastic structural analyses.

Acceptable levels of damage from various degradation mechanisms were cited in the report, using both text descriptions and damage state charts for visual inspections and various test methods. These criteria were established to minimize the potential for damage-causing loss of integrity. As each degradation mechanism has varying rates of attack and differing influence on the affected structure, guidance was given on treatment of observed conditions. Although general degrees of acceptance have been specified, the inspection team must be cognizant of degradation characteristics and identifying the severity of its presence; adjustment of the acceptance criteria may be necessary.

Current shortcomings to the comprehensive inspection of nuclear plant concrete structures include the following:

- Many structures are either partially or completely inaccessible. Access through removal of surrounding materials and structures for visual inspection is costly, may require a plant outage, and requires restoration efforts. Current NDE techniques offer limited capability for determining the condition of inaccessible structures.

- The massive size and complex reinforcing patterns of many structures restricts the use and accuracy of available nondestructive testing techniques. Destructive testing methods may be used to enhance knowledge of condition; however, the cost to conduct such sampling and testing is relatively high and removed portions of the structure must be repaired.

- Evaluation of the acceptability of damage in a structure is influenced by many factors including strength requirements from steady-state and plant emergency loadings, source of damage and rate of attack, and influence of the damage on performance. Significant inspection efforts may be required to support an evaluation of remaining structural integrity. This remains as a relatively unexplored area of structural engineering.

To date, the performance of concrete structures at NPPs has been quite satisfactory. Most documented problems with degradation or aging have been associated with poor quality of construction or locally aggressive environments. This has been witnessed by the limited number of USNRC I&E bulletins or notices regarding concrete structures and potentially generic concerns.

This document reviews current inspection practice and makes recommendations for scheduling of in-service inspections (ISIs), and use of a qualified inspection team. For scheduling purposes, it is recommended that screening and selection of more critical structures be performed and that reliability concepts be considered. Recommended inspector qualifications are also provided, as the ultimate accuracy and value of the inspection results is contingent on the capability and diligence of the inspector.

Basic components of a recommended ISI program would include the following:

1. Written procedure defining responsibilities, scope of structures involved and inspection program, frequency of inspection, inspection methodology, acceptance criteria, inspector qualifications, and documentation requirements;

2. Full compliance with near-term defined requirements for containment vessel and tendon ISI requirements;

3. Adaptive program adjustments to allow assessment of observed degradation and procedure for classifying and treating the degree of damage in the afflicted structure; and

4. Use of qualified personnel, and subcontracted testing and inspection assistance, in the implementation of the ISI program to assess structures having the greatest concern for aging.

Guidance for the performance of ISI is provided in the form of recommended techniques, applications, acceptance criteria, scheduling, and implementation. Two appendices are provided, with the first illustrating how an inspection program would be established at an operational pressurized water reactor (PWR) plant. The second appendix provides recommendations for performing inspections at NPPs, outside of a formal ISI program.

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# APPENDIX A: INSPECTION PROGRAM DEVELOPMENT FOR EXISTING NPP

An approach for establishing a formal in-service inspection (ISI) program for nuclear power plant (NPP) concrete structures is illustrated in this appendix. This approach expands upon the prioritization methodology and information provided in Appendix B of an earlier Structural Aging (SAG) Program Report entitled *Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants.*[A-1] In this report, three existing NPPs including a pressurized water reactor (PWR) plant with large dry prestressed concrete containment were reviewed to identify critical structures using a rating system. This system was based on rating the safety significance, environmental exposure, and impact of degradation of each structure, with the complete population of NPP concrete structures at the plant included (except non-nuclear structures such as storage warehouses, training buildings, substation structures, on-site craft shops, and similar structures not important to plant operations or safety). Emphasis in the methodology is placed on those structures serving multiple safety functions, exposed to potentially aggressive environments, or susceptible to damage from aging and degradation.

An approach for developing a draft in-service inspection program for critical concrete structures at the example PWR plant is described in the following paragraphs. This approach is also applicable to boiling water reactor (BWR) plants, and may be used for any facility containing structures important to protecting the environment and society such as nuclear waste processing facilities and high-level radioactive waste storage facilities. The initial step in this approach is to prepare a draft inspection program addressing plant structures. After the draft program is written, a brief visual inspection is performed on accessible structures included therein. Using the results of this inspection and data from other sources (such as past repair and maintenance logs), necessary changes to the draft program are incorporated and the inspection program is finalized. The resulting program is one that is both performance-based and prioritized on those structures having the greatest sensitivity to aging and potential degradation. It is those concrete structures that deserve attention in a plant-wide visual inspection program. This performance-based approach also has many other applications, including demonstrated compliance with the "maintenance rule" contained in 10CFR50[A-2] and as a means of prioritizing technical studies for license renewal applications.

This approach has been used previously to develop an in-service inspection program for the subject PWR plant concrete structures. For simplicity, this approach is illustrated using a summary list of ranked concrete structures prepared for the example PWR plant, using the ranking methodology contained in Ref. A-1.

## Overview of Example PWR Plant

The example PWR plant with post-tensioned containment is founded entirely on compacted fill material with seasonal groundwater cycling between top-of-grade and 15 ft below grade, cooled by fresh water, and exposed to a fair amount of freeze/thaw and wet/dry climatic cycles. All concrete structures were constructed of concrete having a minimum compressive strength of 28 MPa and with conventional reinforcing steel having a minimum yield strength of 414 MPa. The large, dry containment vessel consists of a conventionally reinforced basemat foundation, post-tensioned cylinder walls (vertical and 120° hoop tendons), and post-tensioned dome. The post-tensioning system consisted of unbonded wire tendons encased in ducts filled with petrolatum wax-type filler material. The containment vessel internal surfaces were lined with thin (6.25 mm) carbon steel plate, while the spent fuel pool was lined with thin (6.25 mm) stainless steel. The containment dome was protectively coated with a high-solids urethane paint system. Subterranean structures were protected with limited waterproofing materials and joint stops.

The prioritization and ranking process contained in Ref. A-1 produced the listing of critical structures as noted below. The containment vessel, including dome, cylinder walls, basemat foundation, and post-tensioning system, was found to be the most "critical" structure, followed by containment-internal structures and others noted below:*

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

At the time of investigation, the example PWR plant was performing only the minimum degree of ISI, with the concrete containment and prestressing tendon system inspected per Regulatory Guide 1.35 requirements[A-3] and leak testing performed per 10CFR50. No other inspection has been conducted or is formally performed on plant structures, although spent fuel pool liner leakage is monitored for other reasons (e.g., maintenance of cooling water inventory). Performance-related records from the plant document control system regarding concrete structures were limited to tendon test results. Interviews with plant personnel, including the quality control staff, did not identify any supportive information.

Few design-phase records were recovered, although the plant owner felt that they existed with the original architect-engineer. Some original documents from the construction-phase were located in the plant historical records system, including materials testing results, summary of construction methods employed, material specifications, and as-built structural drawings. During construction, a settlement monitoring program was also used to monitor building settlement and movement. According to the records, after primary consolidation resulted in uniform building settlements of 1/4" to 1/2", no further deformation was observed and the monitoring program was discontinued (although the stakes still exist). This construction-phase information was also reviewed to identify if any special protective measures were employed to limit environmental exposure, to establish the quality of materials used, and to note any construction deviations of relevance to long-term structural performance. Estimates of in-place material properties (i.e., the concrete compressive strength, steel reinforcement yield strength, and ultimate strength of the tendons) were made, as well as relaxation curves over the 40-year operating license period for the tendons. These estimates were to assist in assessing future inspection results.

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

## Initial Inspection Program Development

Using the listing of ranked structures from Ref. [A-1] and the provisions of this report, a prioritized inspection program was initially prepared. The intent of initial planning was to select the structures to be included, define the scope and type of inspection(s) to be performed, establish initial frequencies, set-up program documentation requirements, and identify the inspection personnel available to perform and coordinate the work. The written ISI procedure developed by the plant owner should include all generic or plant-specific NRC and code-mandated requirements, if these exist (such as at San Onofre where inspections of the intake structure should be scheduled at maximum five-year intervals). Such a procedure was prepared for the example PWR plant.

Details of the recommended draft program are listed below. No change from the degree of ISI presently prescribed for the concrete containment (e.g., based on Regulatory Guide 1.35) was found necessary at the example PWR plant as significant degradation was not suspected from reviewed data. No industry-generic or special NRC concerns existed for the example PWR plant. Thus, for the example plant, the containment vessel and other high ranking structures noted previously would be inspected according to the guidelines provided below. These guidelines were formulated without having prior knowledge of structural performance beyond that contained in records provided by the plant and derived from a brief walkdown.

- Containment inspections would consist of the following:

  1. Leakage rate tests at current intervals specified in the ISI program and 10CFR50 Appendix J,[A-4] including general liner and external concrete visual inspections; and

  2. Tendon lift-off tests, detensioning/inspection, and grease testing using the current minimum testing requirements.

- For the remaining high ranking structures (Nos. 2–10), a program involving an initial visual baseline survey followed by more detailed inspection of any identified degradation would be accomplished. If no significant degradation is observed initially (all conditions within Sect. 5.1 criteria), these structures would be scheduled for inspection at 5 year intervals thereafter. These inspections would coincide with scheduled plant outages.

- Remaining structures found important to plant integrity (structures ranked Nos. 11–20) would be inspected visually to provide an initial baseline. Although 5 to 10 year inspection intervals would generally be used after the baseline survey, subsequent inspection frequency for these structures may need adjustment based on a review of environmental conditions. Quantification of the plant-general environmental exposure would be accomplished through sampling of ambient air, cooling water, groundwater and soils, and laboratory analysis. Inspections would include all liners and accessible tendon anchorages. No other testing method would be specified in the inspection plan, unless degradation was observed.

After the ISI program is initially drafted, a baseline visual inspection is recommended to be the first activity for all concrete structures. The baseline would help define which structures and surfaces are accessible, special access provisions needed to properly inspect each structure, general physical condition, and presence of degradation such as corrosion. The other purpose of the baseline survey is to better understand where future inspections should be focussed. This is the "performance-based" feature of the proposed inspection methodology.

## Initial Baseline Survey Results

Following initial program development at the example PWR plant, a baseline inspection of the concrete structures including the post-tensioned containment vessel and portions of the intake structure below the water line was conducted. Surface hardness measurements (rebound hammer) were made on exposed concrete surfaces of these structures. Also, visual assessments of the protective coatings and measurements of their dry film thickness were performed. Environmental exposure information was estimated from existing plant data. Conclusions from the baseline inspection were that:

1. All safety-related structures were found to be in acceptable condition, except for the lower tendon gallery vaults, that were found to have been damaged by leaching, cracking, and reinforcing corrosion. Several wall cracks were over 4 mm wide, and had propagated through-thickness over the full height of the wall. Intruding groundwater had caused significant leaching of concrete salts, and staining from steel reinforcement corrosion was severe. Other local problems are noted below.

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

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

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

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

5. The containment liner was found to be locally severely corroded, with pitting and exfoliation to depths of about 50 percent of the nominal liner thickness at several locations. Liner coating systems were also degraded beyond acceptance criteria and in need of repair.

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

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

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

As a result of this baseline survey, a number of adjustments to the initial draft ISI program were identified as necessary. In addition, the focus of structures to be addressed in the program and frequency of inspection also needed adjustment.

## Inspection Program Adjustments

Adjustments and modifications to the draft inspection program for the example PWR plant, needed to preserve the integrity of critical plant structures because of observed conditions, are addressed below. Acceptance criteria for the plant were developed for concrete structures, using the provisions in this report, to limit and prevent significant degradation from occurring. The following modifications were recommended to be implemented:

1. Actual aggressiveness of soil and groundwater need to be established. Procedures specifying methods to be used, scope of initial testing, and annual sampling and testing of soil and groundwater around the containment vessel perimeter were included in the program.

2. Inspection of lower portions of the containment vessel walls (exterior) after local soils excavation, Auxiliary and Turbine buildings (internal), and intake structure must occur at a maximum 2-year intervals initially because of the extent of water intrusion.

3. The tendon gallery walls and bottom slab require structural repair. An evaluation to select an appropriate repair method must be accomplished in the near term future. Post-repair inspections would be performed annually, until confidence in the repair is gained.

4. The integrity of the Auxiliary and Turbine Building basemats should be checked in the near future with the impact-echo method, to examine for subsurface damage.

5. The damaged Turbine Building wall will be examined at 6-month intervals for crack growth and propagation. A decision to repair the wall or continue monitoring will be made at a later date based on crack activity.

6. A program to identify where leakage of spent fuel pool water from the liner is occurring was specified to be implemented within one year. The reason for concern is the potential impact of borated water on spent fuel pool concrete structures

7. Many structures, including containment-internal structures, were found to be in good condition without any immediate aging concerns. These structures would be scheduled for 5-year inspection intervals. If continued suitable performance is observed, relaxed frequency of inspection is warranted.

8. An evaluation of the containment tendon grease leakage at anchorage zone concrete areas should be conducted, possibly in conjunction with other containment owners having similar problems.

Using the plant format and input from the scheduling/planning department, future inspection schedules for these structures were established. The degree of damage observed in certain non-containment structures warrants inspection at relatively frequent intervals while demonstrated suitable performance of other structures supports a reduced frequency. By focussing the attention of the in-service inspection on structures containing degradation, a performance-based plan was introduced. As repair of degradation is less costly at early stages, early identification through inspection was judged to be important by plant management.

## Conclusions

ISI of critical plant concrete structures is needed to preserve long-term structural integrity and to avoid costly repairs. Use of a performance-based and prioritized inspection program is appropriate to focus on structures and areas of potential concern and to reduce the amount of inspection effort and expense required. Initial baseline survey, and follow-up inspections, serve as a means to define structural performance and integrity. Should structures be found degrading, appropriate actions may be taken to preserve function before substantial repairs or replacements are required.

ISI procedures were developed by the author for the example PWR plant, and a number of other NPPs. Provision was provided to continually update the program based on the results of future inspections. The benefits of implementing such a program at NPPs was demonstrated at a different plant in which costly repairs and a potential structural failure were averted through early inspection and maintenance efforts. Discussions held with the plant owner suggested that the in-service inspection program was effective in mitigating aging and service-related problems, and that it would be continued. This experience and similar ones at other plants demonstrate the value of performance-based inspection programs for passive NPP structures and other components.

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A-1. C. J. Hookham, *Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants*, ORNL/NRC/LTR-90/17, Oak Ridge National Laboratory and Multiple Dynamics Corporation, Tennessee, 1991.

A-2. "Requirements for Monitoring the Effectiveness of Maintenance of Nuclear Power Plants," *Title 10, Code of Federal Regulations Part 50*, "Energy," Sect. 50.65, U.S. Government Printing Office, Washington, D.C., 1992.

A-3. *Regulatory Guide 1.35 – In-service Inspection of Ungrouted Tendons in Prestressed Concrete Containments, Rev. 3*, U.S. Nuclear Regulatory Commission, Washington D.C., 1990.

A-4. *Title 10, Code of Federal Regulations Part 50*, "Energy," Appendix J, U.S. Government Printing Office, Washington, D.C., 1992.

# APPENDIX B: RECOMMENDATIONS FOR NPP INSPECTIONS

The U.S. Nuclear Regulatory Commission (USNRC) staff occasionally conducts formal and informal inspections of operating nuclear power plants (NPPs). Formal inspections are typically carried out to address a specific plant problem (such as to address structural concerns in the intake structure at San Onofre),[B-1] while informal inspections may be scheduled for a variety of reasons. Inspections of passive structures including concrete containment vessels and other safety-related structures are often "informal" in nature, with limited time frame and resources available for execution.

The purpose of this appendix is to identify issues that the author believes are of highest importance to consider during formal and informal inspections of NPP concrete structures, based on past experience. To gain the greatest insight on the full population of plant structures or individual member structural integrity, the following steps should be carried out chronologically regardless of the formality and intent of the inspection. Provision of records, drawings, and calculations should be requested from the plant owner to aid in identification, access, and interpretation of results.

1. Review of past plant inspection and test records, and available design-basis information for the structure(s) in question. Quantify the aggressiveness of plant environments, including subterranean exposures to which external structure(s) are exposed to;

2. Select those structures (or components thereof) having the greatest importance to the plant and identify the number to be included in the particular inspection (step is skipped if particular structure(s) has already been identified). Use Ref. B-2 as appropriate to assist in the selection process;

3. Conduct visual inspection of accessible surfaces. On the basis of information from this and previous steps, determine if additional testing or inspection is required to better assess the condition of the structure(s) in question;

4. Implement additional testing and inspection through subcontract to qualified testing or consulting engineering firm. Assemble all available plant and walkdown records relevant to the structures in question (e.g., past maintenance logs, design basis data, as-built drawings). Compare inspection and test results to acceptance criteria contained in this report, and to results from past inspections of similar structures at other NPPs; and

5. Conduct brief evaluation to determine if additional analysis, repair, or other action is needed at the present time to establish structural integrity for the population of structures inspected. Document the findings of the evaluation.

Guidelines for the completion of each of these steps has been included in the main body of the report.

The NRC structural inspector(s) should have similar credentials and experience as defined for the Responsible Engineer in Chap. 7. An understanding of concrete structure design, behavior, and degradation mechanisms (e.g., their physical appearance, manifestation, and importance to structural integrity) should also be possessed by the inspector(s). Minimum documentation efforts consistent with Sect. 4.5 should be followed, with preparation of the inspection report following the format of previous documents such as Refs. [B-1 and B-3].

**References:**


B-1 "Safety Evaluation By the Office of Nuclear Reactor Regulation, Southern California Edison Company, San Onofre Nuclear Generating Station Unit No. 1, Docket No. 50-206, Intake Structure", U.S. Nuclear Regulatory Commission, April 1985.

B-2 C. J. Hookham, *Structural Aging Assessment Methodology for Concrete Structures in Nuclear Power Plants*, ORNL/NRC/LTR-90/17, Oak Ridge National Laboratory and Multiple Dynamics Corporation, Tennessee, 1991.

B-3 H. Ashar and G. Bagchi, *Assessment of Inservice Conditions of Safety-Related Nuclear Power Plant Structures*, NUREG-1522, Office of Nuclear Regulatory Reactor Regulation, U.S. Nuclear Regulatory Commission, Washington, D.C., June 1995.

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