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CONTRACTOR REPORT

IN-SERVICE INSPECTION AND STRUCTURAL INTEGRITY ASSESSMENT METHODS FOR NUCLEAR POWER PLANT CONCRETE STRUCTURES

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EXECUTIVE SUMMARY

The Structural Aging (SAG) Program, sponsored by the Nuclear Regulatory Commission (NRC) and conducted by the Oak Ridge National Laboratory (ORNL), has the overall objective of providing the NRC with an improved basis for evaluating nuclear power plant concrete structures for continued service, i.e., beyond the 40 years for which an operating license was issued. Basic components of the program include development of a structural materials property data base, establishment of structural component assessment repair procedures, and formulation of a quantitative methodology for continued service determinations. This document addresses the second of the major program components and has the objective of reviewing and assessing nondestructive evaluation, sampling, and structural integrity testing techniques which have application to the evaluation of safety-related concrete components in nuclear power plants. The results of this study will provide input for development of inspection methodologies which can be used for performing a condition assessment of the concrete structures in a nuclear power plant and for developing data which can be used to trend the performance of structural components so that the residual life of these structures can be evaluated. To meet the above objective, the scope of work and this report are divided into six primary sections which address a description of Category I concrete structures, potential degradation factors which can influence the life of the concrete structures, testing techniques which are available to evaluate the concrete and reinforced concrete materials, current in-service inspection methodologies, recommend testing methods to detect concrete degradation, and potential new techniques to assess concrete degradation.

The first major section provides a description of the safety-related concrete components in nuclear power plant structures. These structures include reactor containment buildings, containment base mats, biological shield walls and buildings, and auxiliary buildings.

Potential environmental stressors and aging factors to which the nuclear power plant concrete structures may be subjected are summarized in the next section. Included in the summary are descriptions of the mechanisms of degradation as well as their symptoms. Table 3.1 presents a summary of the primary degradation factors that can affect the concrete, mild steel reinforcing, and prestressing steel systems in Category I concrete structures.

The third major section reviews available nondestructive and destructive testing techniques that may be utilized in the assessment of concrete components in common civil engineering structures. Capabilities, limitations, and accuracies are presented for each of these techniques. Table 4.1 presents the applicability of the different testing techniques and Tables 4.2(a) and 4.2(b) summarize reviewed results for the nondestructive and destructive testing techniques, respectively.

Current in-service inspection methodologies that have been utilized in the assessment of concrete structures are reviewed in the next section. Topics covered include routine and periodic inspections, condition surveys, and examples of applications of several of the nondestructive testing techniques.

The fifth section describes recommended testing methods for use in the detection of the occurrence of the effects of the various degradation factors that are provided in Table 3.1. Results of this section are summarized in an application guide (Table 6.1) which presents a listing of degradation factors, their symptoms, and recommended testing methods.

A review of relatively new techniques that may potentially have application to assessing degradation of concrete is presented in the next section. These techniques include magnetic (leakage flux, nuclear magnetic resonance), electrical (capacitance, polarization resistance, half-cell potential using impulse radar), ultraviolet radiation, and finite-element analysis methods.

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IN-SERVICE INSPECTION AND STRUCTURAL INTEGRITY ASSESSMENT METHODS FOR NUCLEAR POWER PLANT CONCRETE STRUCTURES

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ABSTRACT

One of the important activities in meeting the objectives of Task S.3 (Structural Component Assessment/Repair Technology) in the Nuclear Regulatory Commission's (NRC) Structural Aging Program is to provide information for development of inspection methodologies for concrete structures in nuclear power plants. These methodologies will have application to current condition assessments of these structures as well as developing data for use in the prediction of their residual life. This phase is an important part of the overall program which has the objective of providing the NRC with an improved basis for evaluating nuclear power plant concrete structures for continued service.

Safety-related concrete structures in nuclear power plants are described. Potential environmental stressors and aging factors which can impact the durability of these structures are identified, their degradation mechanisms noted, and symptoms of occurrence listed. Available nondestructive and destructive testing techniques for use in concrete material or structural evaluations are identified and their capabilities, limitations and accuracies noted. Inspection methodologies relevant to general civil engineering structural evaluations are described and several examples of application provided. Recommended testing methods for use in the detection of the effects of pertinent degradation factors are presented in the form of an application guide. Several relatively new techniques which are presently not widely used in concrete structural evaluations are identified and described.

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1. INTRODUCTION

1.1 BACKGROUND

Within the nuclear power industry, the aging of plant structures, systems, and components has become the subject of significant research in the last few years. This interest is prompted by the need to quantify the effects of aging in terms of potential loss of component integrity or function and to support current or future condition assessments of critical components. Since certain concrete structures play a vital role in the safe operation of nuclear power plants, guidelines and criteria for use in evaluating the remaining integrity (residual life) of each structure are needed.

In its role of regulating the safe use of commercial nuclear power generation, the U.S. Nuclear Regulatory Commission (USNRC) has instituted several programs aimed at providing criteria to demonstrate the continued safe and reliable performance of nuclear power plants during any license extension period. This is to ensure that no age-related degradation alters the performance of components, systems, and structures significant to safety and reliability. One of these programs is the Structural Aging Program (Ref. 1.1) initiated at the Oak Ridge National Laboratory to provide the USNRC with potential structural safety issues and acceptance criteria for use in evaluating nuclear power plants for continued service.

1.2 OBJECTIVE AND SCOPE

Task S.3 of the Structural Aging Program, entitled "Structural Component Assessment/ Repair Technology," has the following objectives: (1) development of a systematic methodology for quantitatively assessing the presence, magnitude, and significance of any environmental stressors or aging factors which could impact the durability of safety-related concrete components in nuclear power plants; and (2) providing recommended in-service inspection or sampling procedures which can be utilized to develop the required data both for evaluating the current structural condition as well as trending the performance of these components for use in continued service assessments. One of the activities under this task, "In-service Inspection Techniques," has the objective of reviewing and assessing nondestructive and destructive evaluation, sampling, and structural integrity testing techniques, which have application to the evaluation of safety-related concrete components in nuclear power plants, e.g., containment building, containment base mat, reactor pressure vessel supports, biological shield walls and buildings, etc. Actual in-service inspection results are to be factored into the review and assessment.

In meeting the objective of this study, potential environmental stressors and aging factors to which nuclear power plant concrete structures may be subjected are identified and their manifestations noted. Appropriate nondestructive and destructive testing techniques that can be used to assess the severity of each of these degradation factors are identified and reviewed. Testing methodologies being used to perform condition assessments of concrete components are also reviewed and assessed. Effectiveness, uncertainties, and limitations of these techniques and methodologies are also covered. An application guide has been developed providing recommended techniques for assessing the presence of the various degradation factors that can impact the performance of concrete structures in nuclear power plants. Finally, potential new techniques for use in assessment of concrete structures in nuclear power plants are noted.

1.3 APPROACH

Three primary activities were conducted under this subtask.

1. A review and evaluation was conducted of existing nondestructive and destructive testing and inspection methods to identify capabilities, accuracies, and limitations in the detection/assessment of the aging factors and environmental stressors to which the concrete components in a nuclear power plant may be subjected. Included in the review are potential new techniques, which, although they may not be widely used at present, possess significant potential. Field applications have been included in the review and assessment of the various methods.

2. Methods were identified which can be utilized to evaluate the severity of each of the various degradation factors to which the safety-related concrete structures in nuclear power plants can be subjected. A rating or ranking system was developed to provide an indication of the effectiveness of the methods. Uncertainties associated with each method were included as well as capabilities to perform volumetric examinations (as applicable). In the evaluation process, examples were utilized, whenever possible, to compare actual in-place conditions with that indicated by the particular method.

3. Available structural integrity assessment methodologies which have been or are presently being utilized to perform a condition assessment of concrete components or structural members were reviewed. Included in the review was information on what techniques have been applied to general civil engineering structures as well as nuclear power plants and what these techniques have revealed in the form of degradation. An indication of the effectiveness of the various techniques is

provided. Information on residual life determinations has been factored into the review.

1.4 REFERENCES

1.1 D. J. Naus et al., "Structural Aging (SAG) Program Five-Year Plan: FY 1988-1992," ORNL/NRC/LTR-89/1, Martin Marietta Energy Systems, Inc., Oak Ridge National Laboratory, Oak Ridge, Tenn., March 1989.

2. DESCRIPTION OF SAFETY-RELATED CONCRETE COMPONENTS IN NUCLEAR POWER PLANT STRUCTURES

A myriad of concrete-based structures are contained as a part of a light-water reactor facility. Although the components may vary somewhat according to the selection of the nuclear steam supply system and containment concept, the structures can be grouped into four categories for discussion: reactor containment buildings, containment base mats, biological shield walls or buildings, and auxiliary buildings (balance-of-plant structures). More detailed descriptions of these structures, relative to that provided below, can be obtained from Refs. 2.1 to 2.4.

2.1 REACTOR CONTAINMENT BUILDINGS

The containment building is one of the most important structures of a nuclear power plant facility because it potentially serves as the final barrier against the release of radioactive fission products to the environment under postulated design basis accident conditions (Ref. 2.1). Containment design is based on pressure and temperature loadings associated with a loss-of-coolant accident. The containment is also designed to resist low-probability environmental loadings such as those generated by earthquake, tornado, or other site-specific environmental events such as flood or tsunami. In addition, the containment is required to provide biological shielding under both normal and accident conditions. If necessary, it is also required to protect the internal equipment from tornado- or turbine-generated missiles and aircraft impacts.

Concrete containment buildings were first built in the mid-1960s and typically consisted of an approximately 1.4-m-thick cylindrical reinforced concrete wall with an approximately 1.1-m-thick hemispherical dome and a flat base slab. Leak tightness is provided by a steel liner. Fully prestressed containments were first built in the late 1960s. The fully prestressed containments typically consisted of an approximately 1.1-m-thick cylindrical wall with an approximately 0.8-m-thick shallow dome, a large ring girder at the intersection of the dome and the wall, buttresses, and a flat reinforced concrete base slab. The more recently built prestressed concrete containments have fewer buttresses. In some designs, the ring girder was eliminated through the use of inverted U-shaped vertical tendons.

2.2 CONTAINMENT BASE MATS

As indicated in Ref. 2.1, base mats for reactor containment vessels can be either conventionally reinforced, prestressed, or a combination of conventionally reinforced and prestressed construction. However, the concrete foundation mats in all nuclear facilities have been conventionally reinforced. Design of the base mat considers the system parameters such as peak internal pressure, temperature, and environmental loads. In addition, the influence of the soil-structure interaction must be considered. The base mat is required to support other loads, including direct equipment loads and dead loads transmitted through the containment wall, primary loop compartment, and primary shield wall. Thicknesses of the concrete base mats vary according to the loading and soil conditions. In general, base mat thicknesses have ranged from approximately 2.6 m to 4.1 m or thicker. The base mats are circular in design and may be greater than 45 m in diameter.

2.3 BIOLOGICAL SHIELD WALLS AND BUILDINGS

Biological shield walls for commercial reactors are usually fabricated from standard weight reinforced concrete. Thicknesses of the shield walls typically range from approximately 1.5 to 4 m. The walls can either support all or part of the reactor pressure vessel weight (Ref. 2.1).

A shield building, or secondary containment unit, is a medium leakage reinforced concrete structure that surrounds a steel primary containment vessel (Ref. 2.1). The building is designed to provide biological shielding from accident conditions; biological shielding from parts of the reactor coolant system during operation; and protection of the containment vessel from low temperatures, adverse atmospheric conditions, and external missiles (Ref. 2.5). The building is typically a reinforced concrete cylinder with a base slab and spherical dome. Cylinder wall thickness is approximately 0.9 m and the dome is approximately 0.6 m thick.

2.4 AUXILIARY BUILDINGS

Based on information obtained from Ref. 2.1, auxiliary buildings include functional units such as diesel generator building, control room/building, spent-fuel pit, fuel-handling buildings, safety valve rooms, radioactive waste buildings, and waste management buildings. In general, these structures are box-shaped, shear-wall buildings constructed of reinforced concrete but may contain steel beams that support floor slabs. Basic structural components of the auxiliary buildings include exterior and interior walls, base or foundation slab, roof slab, floor slabs, and columns.

The main function of the exterior walls is to protect safety equipment and piping from external events such as earthquakes, tornados, and tornado-generated missiles. Typical reinforced concrete wall thicknesses range from approximately 0.45 to 1.2 m. Interior walls may be constructed of reinforced concrete, concrete masonry, or heavy weight concrete. Typical thicknesses range from approximately 0.3 to 1.2 m.

The foundation slab is generally constructed of reinforced concrete. Thickness requirements depend on site foundation conditions and plant seismic considerations. Typical thicknesses range from approximately 1.8 to 4.1 m.

Typical roof slabs are approximately 0.5 m thick as determined by requirements to resist tornado-borne penetrating missiles. Reinforced concrete floor slab thicknesses range from approximately 0.3 to 0.9 m. Where a composite steel beam/concrete floor slab is used, the concrete floor is generally 0.3 m thick. Occasionally, radiation shielding requirements may dictate floor slab thickness requirements.

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

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3. DEGRADATION OF CONCRETE STRUCTURES IN NUCLEAR POWER PLANTS

3.1 INTRODUCTION

3.1.1 General

The relevant degradation factors that may influence concrete components in nuclear-safety related concrete components vary by application (Ref. 3.1). Potential degradation factors for reinforced concrete containments (RCCs) are related to those that cause deterioration of the concrete or reinforcing steel. Degradation factors for prestressed concrete containments (PCCs) are the same as for RCCs except that additional factors causing deterioration of the prestressing system would have to be considered. Factors affecting base mats are essentially the same as those for RCCs, plus the factors that contribute to foundation settlement and aggressive chemical attack by the groundwater. Biological shield walls are also susceptible to temperature and irradiation factors that could produce a loss of concrete strength or shielding efficiency.

3.1.2 Approach

As indicated in Refs. 3.2 and 3.3, deterioration of concrete is an extremely complex subject. It would be simplistic to suggest that it will be possible to identify a specific, single cause of deterioration for every symptom detected during an evaluation of a structure. In most cases the damage detected will be the result of more than one mechanism (degradation factor). However, given a basic understanding of the various damage-causing mechanisms, it should be possible, in most cases, to determine the primary cause or causes of damage to a particular concrete structure.

The intent of this chapter is to identify and describe potential age-related degradation factors that may have an impact on concrete structures and components in nuclear power plants. This includes the significance and manifestation of these degradation factors on the physical condition of the reinforced concrete. Results of this section form a background that assists in identifying the appropriate sampling methodology, nondestructive and destructive, that can be utilized to evaluate the severity of each of the various degradation factors to which concrete components in nuclear power plant structures may be subjected.

Table 3.1 presents a summary of the primary degradation factors that can affect the concrete, mild steel reinforcing, and prestressing steel systems (Refs. 3.1 to 3.5). A brief description of these factors in terms of mechanisms of degradation and symptoms (manifestations) follows.

3.2 PRIMARY DEGRADATION FACTORS AFFECTING CONCRETE MATERIAL SYSTEMS

If concrete is properly designed for the environment to which it will be exposed and proper construction methods are utilized, it can be capable of maintenance-free performance for decades (Ref. 3.6). However, concrete is potentially vulnerable to attack under a variety of different exposures. Degradation of concrete can be caused by adverse performance of either its cement-paste matrix or aggregate constituents under either chemical or physical attack.

3.2.1 Chemical Attack

Chemical attack is the alteration of concrete through chemical reaction with either the cement paste or the coarse aggregate. Generally, the attack occurs on the exposed surface region of the concrete (concrete cover), but with the presence of cracks or prolonged exposure, chemical attack can affect entire structural elements (cross sections). The rate of chemical attack on concrete is a function of the pH of the aggressive fluid and the permeability, alkalinity, and reactivity of

concrete. Chemical attack of concrete may occur in several different forms as noted in the following sections.

3.2.1.1 Alkali-Aggregate Reactions

Alkali-aggregate reactivity includes two primary types of reactions: alkali-silica* and alkali-carbonate.

Mechanism. Some aggregate materials containing silica that is soluble in highly alkaline solutions may react to form either a solid non-expansive calcium-alkali-silica complex or an alkali-silica complex which can imbibe considerable amounts of water and then expand and disrupt the concrete (Ref. 3.7). Similarly, certain aggregates of carbonate rock have been found to be reactive in concrete, with the reactions ranging from beneficial to destructive. The destructive reactions pertain to those involving impure dolomitic aggregates and are a result of either dedolomitization or rimsilification reaction (Refs. 3.3 and 3.8). A distinguishing feature which differentiates alkali-carbonate reactions from alkali-silica reactions is the lack of silica gel exudations at cracks (Ref. 3.3).

Symptoms. Damage to concrete as a result of alkali-aggregate reactions is generally in the form of swelling and cracking. Evidence of map or pattern cracking and swelling can generally be observed on the surfaces of concrete structures affected (Refs. 3.2 and 3.7).

3.2.1.2 Sulfate Attack

Sulfates of sodium, potassium, and magnesium, present in alkali soils and waters (rain, groundwater, sea water), can cause deterioration of concrete structures.

Mechanism. The sulfates react chemically with the hydrated lime and hydrated calcium aluminate in cement paste to form calcium sulfate and calcium sulfoaluminate. This reaction is associated with expansion and disruption (cracking) of the concrete.

Symptoms. Deterioration in the form of pattern cracking as well as general disintegration of concrete can be observed (Ref. 3.3). Sulfate attack can also cause progressive loss of strength and loss of mass due to deterioration of cohesiveness of the cement hydration products. The rate of attack is a function of the reactivity of the cement paste and concentration of sulfate compounds.

3.2.1.3 Efflorescence and Leaching

Efflorescence occurs on the surface of concrete following the percolation or seepage of a fluid (e.g., water) through the material, either intermittently or continuously, or when an exposed surface is wetted and dried. Efflorescence results from the dissolution of salts contained in the cement matrix, which are leached out of the concrete and crystallized upon subsequent evaporation of fluid or interaction with carbon dioxide in the atmosphere. As such, efflorescence is primarily an aesthetic problem rather than a durability problem but may indicate that alterations to the cement paste are taking place in the concrete.

Leaching is a form of mild chemical attack which alters the cement paste matrix.

Mechanism. Pure water from condensation of fog or water vapor, or soft water from rain or melting snow, which contain little or no calcium ions has a tendency to hydrolyze or dissolve the calcium-containing products in concrete (Ref. 3.9). In hydrated portland cement pastes, calcium hydroxide is the constituent most susceptible to hydrolysis. The rate of leaching is dependent on the permeation of the fluid through the cement matrix, the temperature, and the reactivity of the concrete.

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* Alkali-silicate reaction also can occur, but it is not as common as the alkali-silica reaction.

Symptoms. Extensive leaching causes an increase in porosity and permeability, thus lowering the concrete strength (Ref. 3.10) as well as making it more vulnerable to hostile environments.

3.2.1.4 Bases and Acids

Hydrated cement paste is an alkaline material (pH $\geq$ 12.5) and not normally susceptible to attack by alkaline materials.

Mechanism. Contact between concrete and high concentrations of alkaline materials (from water treatment or industrial processes), however, can result in deterioration processes other than direct chemical reaction with hydroxide ions. On the other hand, concrete is vulnerable to acidic solutions which readily attack basic materials, e.g., sulfuric acid and carbonic acid in groundwater and certain plant internal fluids such as boric acid. Acid rain presents another potential source of distress to concrete.

Symptoms. Disintegration of concrete leading to loss of cement paste and aggregate in the matrix (erosion) can be expected (Ref 3.3).

3.2.1.5 Salt Crystallization

Water containing large quantities of dissolved solids (e.g., $CaSO_4$, $NaC\ell$, Na_2SO_4) can cause damage to concrete through the development of crystal growth pressures. Structures in contact with fluctuating water levels or in contact with groundwaters containing salts are susceptible to this type of deterioration.

Mechanism. As water containing large quantities of dissolved solids permeates through concrete, the salts crystallize in open pores due to evaporation.

Symptoms. Repeated or continuing evaporation of water containing dissolved solids will cause salt deposits to build up generating pressures that can cause cracking of the concrete.

3.2.2 Physical Attack

Physical attack is the second major cause of deterioration of concrete. Although it is difficult to separate physical attack from chemical attack of concrete, for purposes of discussion, physical attack will include degradation factors that result from environmental or mechanical effects.

3.2.2.1 Moisture Changes

Volume changes of concrete can be caused by humidity or moisture changes. Drying shrinkage is due to loss of physically absorbed water from the calcium-silicate hydrate (Ref. 3.11). However, expansion of concrete can occur due to gain of physically absorbed water by the calcium-silicate hydrate.

Mechanism. If uniform shrinkage or expansion of the concrete due to loss or gain, respectively, of the physically absorbed water takes place without restraint, no stresses will be induced into the concrete. However, if the concrete is restrained, stresses are generated. Differential moisture movement can also produce similar effects. In the presence of stress the effects can be accentuated due to creep.

Symptoms. Cracking of concrete will occur if the tensile stresses developed due to changing moisture conditions exceed the tensile strength of the concrete.

3.2.2.2 Freeze/Thaw Cycling

Concrete, when in a saturated or near-saturated condition, can be susceptible to damage during freezing and thawing cycles. Factors controlling the resistance of concrete to freeze/thaw damage include air entrainment (sizing and spacing of air bubbles) as opposed to entrapped air, water/cement ratio, curing, strength, and degree of saturation (Ref. 3.12).

Mechanism. As the temperature of concrete in a saturated or near-saturated condition is lowered, the water in the capillary cavities of the cement paste can freeze and expand in volume by approximately 9% to expel excess water. If subsequent thawing is followed by refreezing, further expansion takes place so that repeated cycles of freezing and thawing have a cumulative effect. Frost damage to concrete is believed to be due primarily to the subsequent freeze-thaw cycles producing a buildup of pressure during ice lense formation.

Symptoms. The extent of damage due to freeze/thaw action may vary from surface scaling to complete disintegration as layers of ice form. The ice formation starts at the concrete member's exposed surface and progresses through its depth (Ref. 3.12). The most common damage that occurs to concrete during freeze/thaw cycling is cracking or spalling. Cracking parallel to joints and edges of slabs (D-cracking) can occur when certain (porous) coarse aggregates have been used in the concrete mix.

3.2.2.3 Thermal Exposure/Thermal Cycling

Temperature variations, or thermal cycles, can become important if the deformations of the concrete structure resulting from the changing temperature are constrained. Although most reinforced concrete structures are subjected to thermal cycling due to daily temperature fluctuations, they are designed accordingly, i.e., inclusion of steel reinforcement. When concrete is exposed to elevated temperatures, its response is dependent on a number of factors, e.g., type and porosity of aggregate, rate of heating, permeability, moisture state, etc. Reference 3.13 suggests that concrete exposed to a temperature of 90°C may lose up to 10% of its room-temperature strength and modulus of elasticity values. Significant deterioration of concrete strength does not generally occur until the exposure temperature reaches approximately 400°C at which dehydration of calcium hydroxide occurs (Ref. 3.14). Variation of modulus of elasticity of concrete with elevated temperatures is reported in Ref. 3.15. Figure 3.1 shows reduction in the modulus of elasticity as the temperature increases. The effects of temperature on carbonate and siliceous aggregate concretes are shown in Figs. 3.2 and 3.3, respectively. Elevated temperatures are also important in that they effect the volume change and creep of the concrete.

Mechanism. The mechanical property variations of concrete resulting from thermal cycling or elevated temperatures occur largely because of changes in the moisture content of the concrete constituents, and progressive deterioration of the cement paste and aggregate due to differing thermal expansion values.

Symptoms. Thermal cycling, even at relatively low temperatures (<65°C), can have some deleterious effects on concrete's mechanical properties, i.e., compressive, tensile and bond strengths, and modulus of elasticity are reduced. As concrete is subjected to higher temperatures, its mechanical properties are reduced. Excessive thermal exposure is typically characterized by cracking or spalling of the concrete cover at the exposed surface. Evaluation of mechanical properties of concrete requires removal of representative samples and appropriate laboratory test procedures.

3.2.2.4 Irradiation

Irradiation in the form of fast and thermal neutrons emitted by the reactor core, or gamma rays produced as a result of capture of neutrons by members (particularly steel) in contact with concrete, can affect the concrete.

Mechanism. The fast neutrons are mainly responsible for considerable growth, caused by atomic displacements, that has been measured in certain aggregates, e.g., flint. Gamma rays produce radiolysis of water in cement paste that can affect concrete's creep and shrinkage behavior to a limited extent and also result in evolution of hydrogen (Ref. 3.2).

Symptoms. Prolonged exposure of concrete to irradiation can result in decreases in compressive and tensile strengths and modulus of elasticity. Damage of concrete due to excessive irradiation is in the form of cracks and spalls at exposed surfaces (Ref. 3.16). Evaluation of the

effects of irradiation on concrete's mechanical properties requires removal and testing of representative samples of material.

3.2.2.5 Abrasion/Erosion/Cavitation.

Progressive loss of material at concrete surfaces can occur as a result of abrasion, erosion, or cavitation (Ref. 3.17).

Mechanism. Abrasion generally refers to dry attrition with erosion used to describe wear by the abrasive action of fluids containing solid particles in suspension. Cavitation relates to the loss of surface material by formation of vapor bubbles and their sudden collapse due to an abrupt change of direction or pressure in rapidly flowing water at the surface of a structure.

Symptoms. Abrasion, erosion and cavitation result in progressive loss of material at concrete surfaces. Surfaces subjected to abrasion appear worn and polished (Ref 3.18). Surface wear resulting from erosion depends on the quantity, shape, size, and hardness of the particles being transported, on the velocity of their movements, on the presence of eddies, and the concrete quality (Ref. 3.12). Cavitation produces severely pitted and extremely rough concrete surfaces.

3.2.2.6 Fatigue/Vibration

Concrete structures subjected to fluctuations in loading, temperature, or moisture content (which are not large enough to cause failure in a single application) can be damaged by fatigue.

Mechanism. Fatigue damage initiates as microcracks in the cement paste, proximate to large aggregate particles, reinforcing steel, or stress risers (e.g., cracks). Upon continued or reversed load application, these microcracks may propagate to cause bond loss with proximate steel and to form structurally significant cracks which can expose the concrete and reinforcing steel to hostile environments or produce increased deflections.

Symptoms. Damage to reinforced concrete subjected to fatigue or vibration loadings can be in the form of cracking and/or excessive deflections. Ultimate failure of a concrete structure in fatigue will occur as a result of excessive cracking, excessive deflections, or brittle fracture.

3.2.2.7 Creep

Creep is defined in Ref. 3.12 as the increase in strain under a sustained stress. Creep may also be viewed from another standpoint. If the restraints are such that a stressed concrete is subjected to a constant strain, creep will manifest itself as a progressive decrease in stress with time called stress relaxation (Ref. 3.19).

Mechanism. It is indicated in Ref 3.19 that the source of creep is in the cement paste and is related to internal movement of absorbed or intracrystalline water (internal seepage). Experiments have shown that concrete from which all evaporable water has been removed exhibits practically no creep. However, the change of creep behavior of concrete at high temperature suggests that water ceases to play a role and the gel itself becomes subjected to creep deformation. Reference 3.19 further indicates that in mass concrete, creep may be a cause of cracking when the restrained mass undergoes a cycle of temperature change due to development of the heat of hydration and subsequent cooling. Creep relieves the compressive stress induced by rapid rise in temperature so that the remaining compression disappears as soon as some cooling has taken place. On further cooling of concrete, tensile stresses develop, and, since the rate of creep is reduced with age, cracking may occur even before the temperature has dropped to the initial value.

Symptoms. As indicated in Ref. 3.19, creep can cause cracking of mass concrete subjected to change in temperature and increased deflections. Furthermore, Ref. 3.4 notes that creep in prestressed concrete members, if neglected, may lead to excessive prestress loss that in turn can result in cracking as loads are applied.

3.3 PRIMARY DEGRADATION FACTORS AFFECTING MILD STEEL REINFORCING SYSTEMS

Mild steel reinforcing systems are provided in concrete structures to control the extent of cracking and width of cracks at operating temperatures, resist tensile stresses and compressive stresses for elastic design, and provide structural reinforcement where required by limit design procedures (Refs. 3.20 and 3.21). Potential causes of degradation of the mild steel reinforcing are corrosion, elevated temperature exposure, irradiation, and fatigue.

3.3.1 Corrosion

Although deterioration of concrete may be attributed to the combined effects of more than one cause, corrosion of reinforcement is one of the principal causes. Corrosion of steel in concrete is an electrochemical process. The electrochemical potentials which form the corrosion cells may be generated in two ways: (1) composition cells formed when two dissimilar metals are embedded in concrete, such as steel rebars and aluminum conduit, or when significant variations exist in surface characteristics of the steel; and (2) concentration cells formed due to differences in concentration of dissolved ions in the vicinity of steel, such as alkalies, chlorides, and oxygen (Ref. 3.9). As a result, one of two metals (or different parts of the metal when only one metal is present) becomes anodic and the other cathodic. Other potential causes of corrosion include the effects of stray electrical currents or galvanic action with an embedded steel of different metallurgy.

Mechanism. The transformation of metallic iron to ferric oxide (rust) is accompanied by an increase in volume which, depending on the state of oxidation, may be as large as 600 percent of the original metal. In good quality, well-compacted concretes, reinforcing steel with adequate cover should not be susceptible to corrosion because of the highly alkaline conditions present within the concrete (pH > 12). The high alkaline condition causes a passive iron oxide film to form on the iron surface, i.e., metallic iron will not be available for anodic activity. However, when the concrete pH falls below 11, a porous oxide layer (rust) can form on the reinforcing steel due to corrosion. Carbonation and the presence of chloride ions can destroy the passive iron oxide film. Reduction of the concrete pH can occur as a result of leaching of alkaline substances by water or carbonation, i.e., calcium hydroxide is converted to calcium carbonate (calcite). The penetration of CO_2 from the environment into the concrete structure can be accelerated due to the concrete being porous (poor quality) or the presence of microcracks. The penetration of chloride ions can also destroy the passive oxide film on the reinforcing steel, even at high alkalinities (pH > 11.5). For typical concrete mixtures, the threshold chloride content to initiate steel corrosion is about 0.6 to 1.2 kg $C\ell^-/m^3$ (Ref. 3.22). Chlorides may be present due to external sources (deicing salts, etc.) or may be naturally introduced into the concrete via aggregate or water used in the mix. Furthermore, when large amounts of chloride are present, concrete tends to hold more moisture, which also increases the risk of steel corrosion by lowering concrete's electrical resistivity. Once the passivity of the steel is destroyed, the electrical resistivity of concrete and availability of oxygen controls the rate of corrosion.

Symptoms. Corrosion of mild reinforcing steel embedded in concrete is typically evidenced by rust staining of the concrete surface. As corrosion progresses, cracks occur due to formation of the corrosion product (expansive reaction). Cracks produced by corrosion generally run in straight, parallel lines and at intervals corresponding to spacing of the reinforcement. Corrosion of mild reinforcing steel can also cause surface delamination as a result of the internal expansive forces caused by formation of corrosion product. As corrosion continues, spalling of concrete cover can occur to expose the rebar.

3.3.2 Elevated Temperature

The properties of mild steel reinforcement used in design are generally a function of the yield stress, which is affected by exposure to elevated temperature.

Mechanism. Elevated temperatures increase the diffusion rate and produce crystal transformations which result in changes in material properties. As the temperature increases, the steel can be recrystallized to form larger crystals to produce a material which becomes weaker and more ductile.

Symptoms. Data for reinforcing steels from Ref. 3.23 indicate that for temperatures up to approximately 200°C, the yield strength is reduced by ≤10% and that at 500°C, it falls to about 50% of its reference value with hot-rolled steels performing better than cold-twisted or cold-drawn steels. The modulus of elasticity exhibits a similar reduction pattern with increasing temperature. References 3.24 and 3.25 confirm the effects of temperatures above 200°C on reinforcing steel as well as a threshold temperature of 300°C for loss of reinforcing steel bond properties with concrete. As these threshold values are below typical light-water reactor temperatures, thermal damage to reinforcing steels should be negligible except at local areas or structures in close proximity to the nuclear steam supply system. Assessment of the effects of elevated temperature on mild steel reinforcing properties requires removal and testing of representative material samples.

3.3.3 Irradiation

Neutron irradiation produces changes in the mechanical properties of carbon steel. In light-water reactor facilities, the critical reinforcing steel that may experience irradiation effects is located in the primary shield wall adjacent to the reactor pressure vessel.

Mechanism. Changes in mechanical properties of carbon steel result from the displacement of atoms from their normal sites by high-energy electrons causing the formation of interstitials and vacancies.

Symptoms. Irradiation produces an increase in yield strength and a rise in ductile/brittle transition temperature of carbon steels. As a consequence, reduced ductility will increase the possibility of brittle failure. Evaluation of the effects of irradiation on mild steel reinforcing properties requires removal and testing of representative material samples.

3.3.4 Fatigue

Fatigue of mild reinforcing steel would be coupled with that of the surrounding concrete.

Mechanism. Fatigue damage initiates as microcracks in the cement paste, and, upon continued or reversed load application, the microcracks propagate throughout the structure.

Symptoms. The repeated loadings or vibrations lead to a loss of bond between the steel reinforcement and concrete. For extreme conditions, the strength of the mild steel reinforcing system may be reduced or failures may occur at applied stress levels less than the yield strength. Because of the typically low normal stress levels in reinforcing steel of the light-water reactor facilities, fatigue failure is not likely to occur.

3.4 PRIMARY DEGRADATION FACTORS AFFECTING PRESTRESSING STEEL SYSTEMS

A post-tensioned prestressing system consists of prestressing tendons which are installed and tensioned using jacks and other devices and then anchored to hardened concrete. A number of light-water reactor concrete containments utilize steel prestressing tendons to provide primary resistance to tensile loadings. Three major categories of prestressing system exist, depending on the type of tendon utilized: wire, strand, or bar. Protection of the prestressing systems is provided by filling the ducts containing the post-tensioned tendons either with organic corrosion inhibitors, which include microcrystalline waxes (e.g., petrolatums for ungrouted tendons), or portland cement grout (grouted tendons). Ungrouted prestressing systems have been used in most of the light-water reactor plants. Potential causes of degradation of prestressing systems include corrosion, elevated temperature exposure, irradiation, and loss of prestressing force.

3.4.1 Corrosion

Corrosion of prestressing systems can be highly localized or uniform.

Mechanism. Most corrosion-related failures have been the result of localized attack produced by pitting, stress corrosion, hydrogen embrittlement, or a combination of these. Failure of prestressing tendons can also occur as a result of microbiological-induced corrosion or possibly due to stray electrical currents.

Symptoms. Pitting is the electrochemical process that results in local penetrations into the tendon reducing the cross section to the point where it is incapable of supporting applied loads. Stress corrosion cracking results in the fracture of a normally ductile metal or alloy under stress (tensile or residual) in specific corrosive environments. Hydrogen embrittlement, frequently associated with hydrogen sulfide, occurs when hydrogen atoms enter the metal lattice and significantly reduce its ductility.

3.4.2 Elevated Temperature

Elevated temperature can have significant effects on all heat-treated and drawn wires.

Mechanism. Elevated temperature destroys the crystal transformations achieved by the heat process utilized for fabrication.

Symptoms. Short-term heating, on the order of three to five minutes, even to temperatures as high as 400°C, may not significantly affect the properties of the prestressing steel (Ref 3.26). Results from Ref. 3.23, involving 30 types of prestressing steels, indicate that thermal exposures up to approximately 200°C do not significantly reduce (<10%) the tensile strength of prestressing wires or strands. References 3.25 and 3.27 support results presented in Ref. 3.23. Elevated temperature exposures also affect the relaxation and creep properties of prestressing tendons. Reference 3.28 indicates that losses in a 15.2-mm-diam strand initially stressed to 75% of its guaranteed ultimate tensile strength at 40°C will be 5 to 6.4% after 30 years. Relaxation losses of tendons composed of stress-relieved wires have relaxation losses of about the same magnitude as stress-relieved strand, but relaxation of a strand is greater than that of its straight constituent wire because of the combined stress relaxation in the helical wires (Ref. 3.29). Creep of stress-relieved wire is negligible if stressed up to 50% of its tensile strength. Also, the creep effect in steel varies with its chemical composition as well as with mechanical and thermal treatment applied during the manufacturing process. As temperature levels experienced by the prestressing steels in light-water reactor facilities are below 200°C, thermal damage to the prestressing steels under normal operating conditions is unlikely. Assessment of the effects of elevated temperature on the mechanical properties of prestressing steels requires removal and testing of representative material samples.

3.4.3 Irradiation

Irradiation of prestressing steels can affect its mechanical properties.

Mechanism. Irradiation of steel results in atoms being displaced from their normal sites by high-energy neutrons to form interstitials and vacancies.

Symptoms. Defects formed as a result of irradiation can grow together and effectively both strengthen the steel and reduce its ductility, or, at higher temperatures, they can recombine and annihilate each other and, for a given dose, reduce the irradiation damage (Ref. 3.30). Results obtained from studies (Ref. 3.30) in which 2.5-mm-diam prestressing wires were stressed to 70% of their tensile strength and irradiated to a total dose of 4×10^{16} neutrons/cm^2 (flux of 2×10^{10} neutrons•cm^2•s) showed that, for exposures up to this level, the relaxation behavior of irradiated and unirradiated materials was similar. These flux levels are higher than the level likely to be experienced in a light-water reactor containment vessel. Evaluation of the effects of irradiation on prestressing steel properties requires removal and testing of representative material samples.

3.4.4 Loss of Prestressing Force

Primary contributors to loss of original force level that was applied to prestressing tendons include duct friction, slip of anchorages, tendon relaxation, elastic shortening, and concrete creep and shrinkage. Of these, tendon relaxation is aging related due to its time-dependent nature.

Mechanism. Tendon relaxation is related to tendon material properties, initial stress level, exposure temperature, and time. Loss of prestress force occurs due to atomic diffusion, whereby atoms migrate to positions of lower energy. Creep of concrete was discussed in Section 3.2.2.7.

Symptoms. Excessive loss of prestressing force in flexural members will result in increased deflections and cracking of the concrete. In prestressed concrete containments, loss of prestressing force would not be noticeable because of the relatively thick sections and nature of loadings under normal operating conditions. A tendon liftoff test would have to be performed to determine the level of prestress in these structures.

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

Material System	Degradation Factor	Primary Manifestation
Concrete	Chemical attack	
	Alkali-aggregate reactions	Volume change/cracking
	Sulfate attack	Volume change/cracking
	Efflorescence/leaching	Increased porosity
	Bases and acids	Increased porosity/erosion
	Salt crystallization	Cracking
	Physical attack	
	Moisture changes	Cracking
	Freeze/thaw cycling	Cracking/spalling
	Thermal exposure/thermal cycling	Cracking/spalling
	Irradiation	Volume change/cracking
	Abrasion/erosion/cavitation	Section loss
	Fatigue/vibration	Cracking
	Creep	Volume change/deflections
Mild steel reinforcement	Corrosion	Concrete cracking/spalling/surface delamination
	Elevated temperature	Decreased yield strength
	Irradiation	Reduced ductility
	Fatigue	Reduced strength
Prestressing system	Corrosion	Reduced section
	Elevated temperature	Reduced strength
	Irradiation	Reduced ductility
	Loss of prestressing force	Increased deflections, cracking

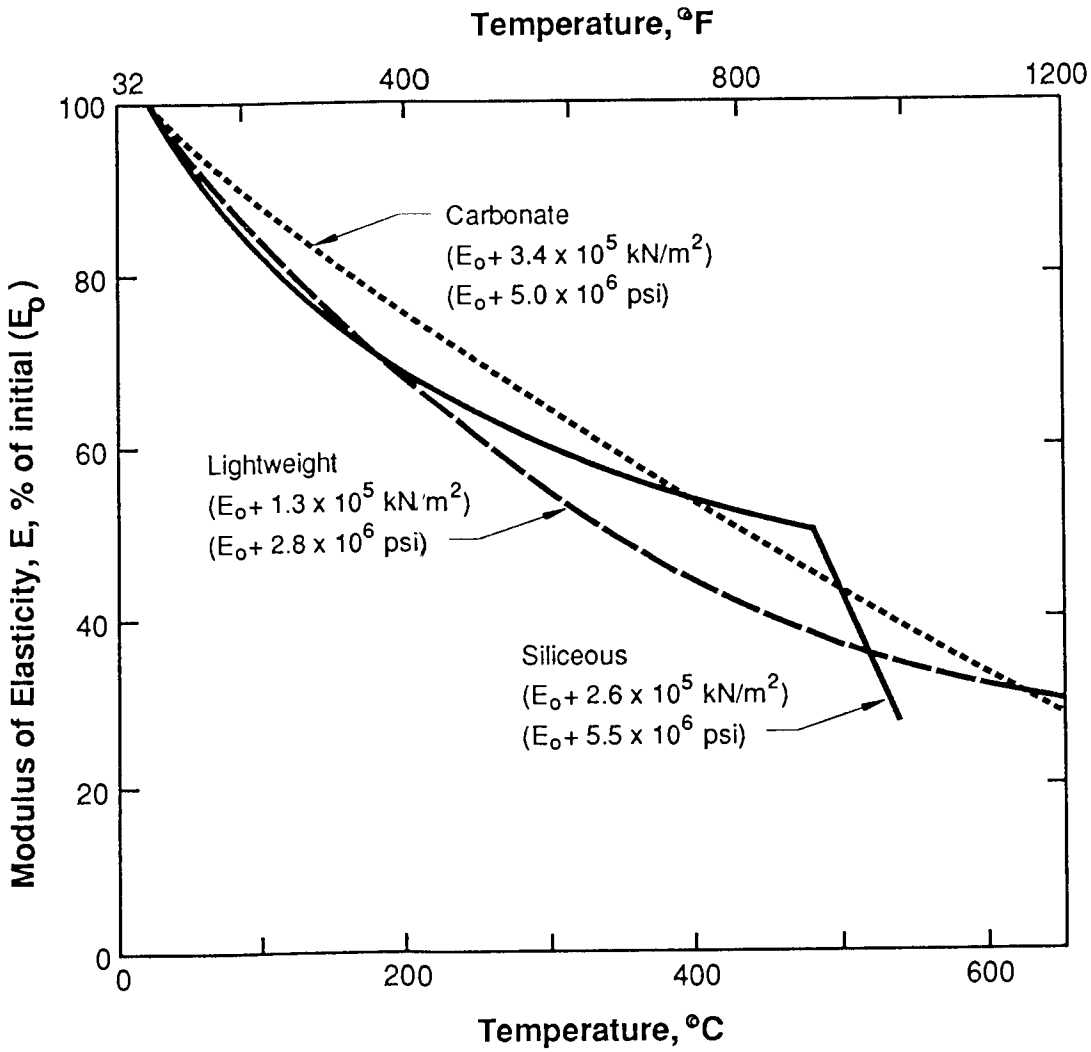

Fig. 3.1 Modulus of elasticity of concrete versus temperature (Ref. 3.15).

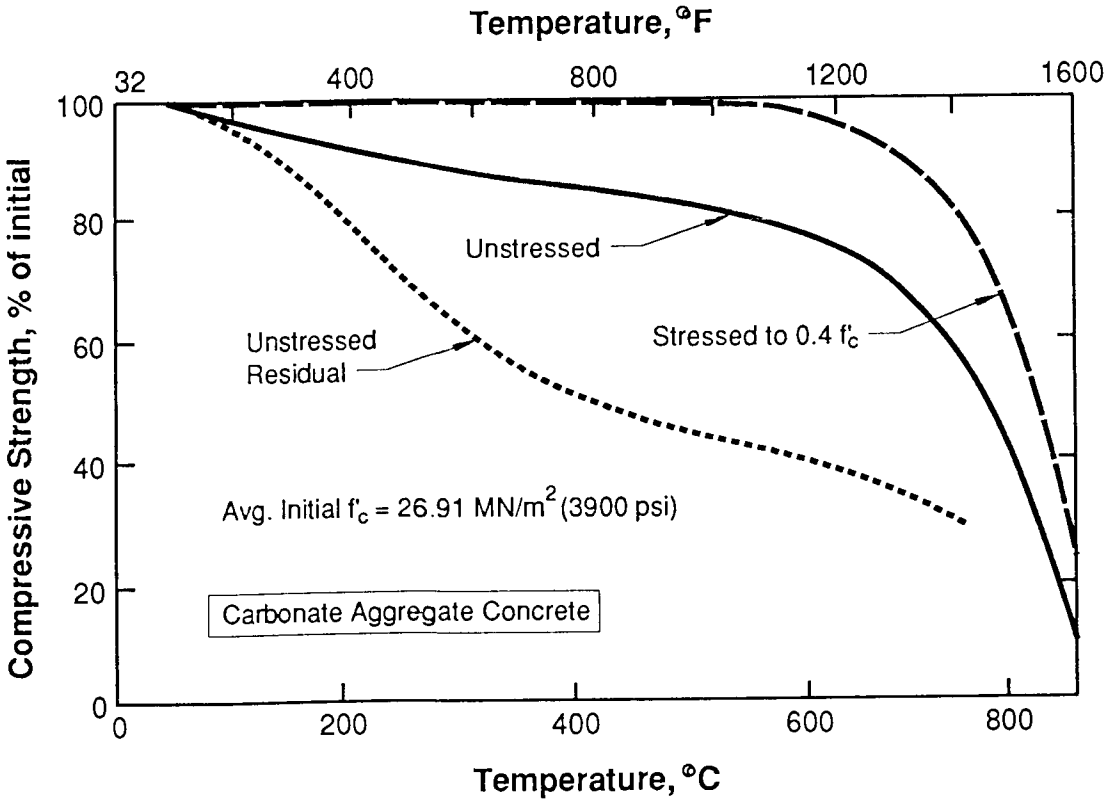

Fig. 3.2 Compressive strength of carbonate aggregate concrete at high temperature after cooling (Ref. 3.15).

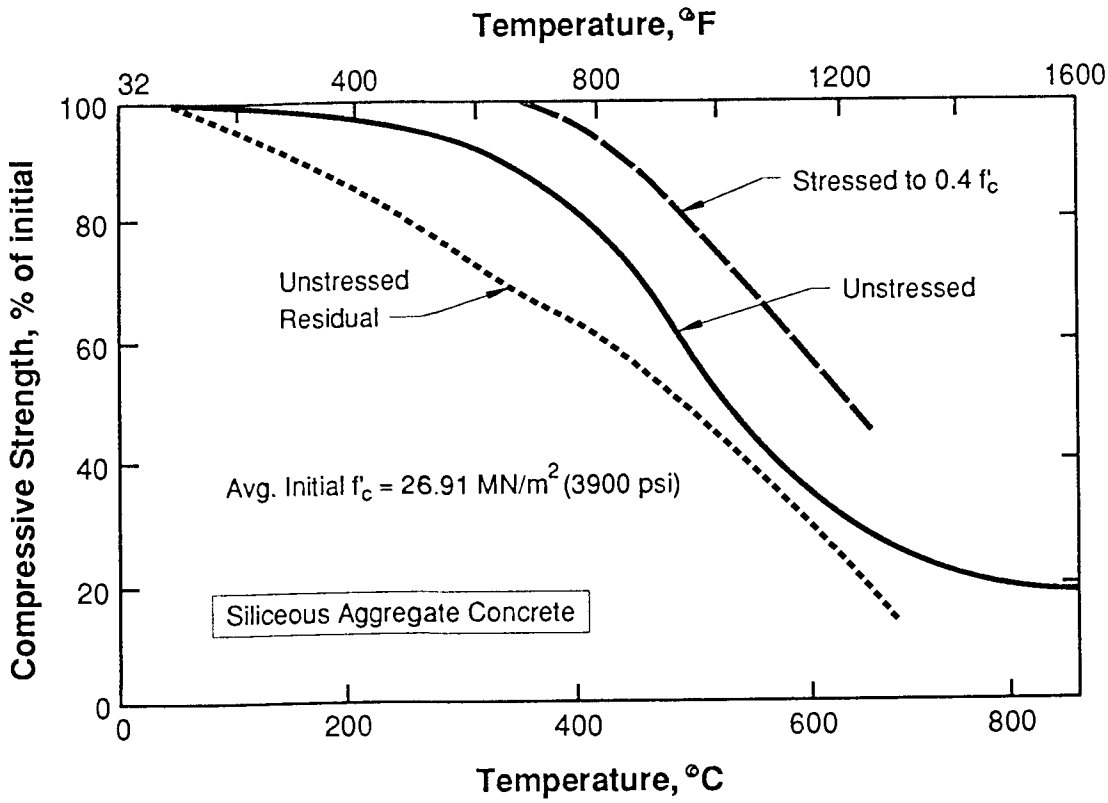

Fig. 3.3 Compressive strength of siliceous aggregate concrete at high temperature and after cooling (Ref. 3.15).

4. TESTING TECHNIQUES TO EVALUATE CONCRETE AND REINFORCED CONCRETE MATERIALS

4.1 INTRODUCTION

Methods used to detect degradation of concrete materials generally fall into two categories: direct and indirect. Direct techniques involve a visual inspection of the structure, removal/testing/analysis of material(s), or a combination. The indirect techniques generally measure some property from which an estimate of strength, elastic behavior, extent of degradation, etc., can be made through existing correlations. Quite often, however, evaluation of concrete materials and structures requires the use of a combination of test methods since no single testing technique is presently available that will detect all potential degradation factors. In the remaining sections of this chapter, nondestructive and destructive testing techniques which are available to evaluate concrete and reinforced concrete materials are described, their capabilities and limitations noted, and accuracies provided. Section 6 of this report provides a summary of information on the ability of selected techniques to determine either the presence of specific degradation factors or to evaluate their effects.

4.2 NONDESTRUCTIVE TECHNIQUES

Nondestructive techniques can be utilized to indicate the strength, density and quality of concrete; locate voids or cracks in the concrete; locate the steel reinforcement and indicate depth of concrete cover; and indicate corrosion of steel reinforcing materials. Visual inspection of the structure is generally the first nondestructive testing method utilized in a typical evaluation program. Nondestructive testing methods include: (1) audio, (2) electrical, (3) impulse radar, (4) infrared thermography, (5) magnetic, (6) stress wave reflection/refraction, (7) modal analysis, (8) radioactive/nuclear, (9) rebound hammer, and (10) ultrasonic.

Quantitative data regarding the frequency distribution of detected (indicated) strength and probability of nondestructive testing methods to detect a flaw (void or crack) having a specific size and orientation is not currently available. However, quantitative information relating variation in detected strength with control parameter is provided in the following subsections when applicable.

4.2.1 Audio Methods

Audio methods are useful for detection of cracks, voids, and delaminations in concrete. The procedure requires the use of hand tools including hammers, steel rods, chains, etc., which are used for striking the structure to detect sound differentials between good and defective concrete. The procedure is repeated over the entire surface of the selected test areas of a concrete structure. Any defects such as delaminations, debonds, voids, and cracks are distinguished by a "hollow" sound when struck (Ref. 4.1). The theory underlying the audio method of testing is when a metallic object strikes a solid (nondelaminated) area, a metallic ringing sound will be heard. If a defect is present, a "hollow" sound would be noted. The chain drag, Delamtect, and WesCet represent existing techniques based on audio methods. These techniques may not be applicable to nuclear power plant concrete structures because of their large size (i.e., thicknesses up to several meters) and positions in the plant.

4.2.1.1 Chain Drag

Description. The chain drag method is the most commonly used method for evaluation of defects in the top surface of horizontal placed concrete. A set of three to seven strips of chains are hung from the end of a "T"-shaped wooden handle. The chains are dragged from side to side in a sweeping motion on the surface of the concrete and any "hollow" sound is noted. It has been

found that logging chains with a 50.4-mm link produce better results than small chains because the lightweight chains do not provide a distinct "hollow" sound when dragged over delaminations.

Capabilities and limitations. Although the chain drag is one of the most accurate of the audio testing methods, it has a tendency to miss small delaminations that do not provide a distinctive hollow sound when swept over (Ref. 4.2). The method is usually effective for defect depths not greater than the concrete cover. Furthermore, chain drag is a slow and tedious method of locating and recording delaminations.

Accuracy. The accuracy of the chain drag method ranges from good to excellent (Ref. 4.1). No quantitative information is available.

4.2.1.2 Delamtect

Description. The Delamtect was developed by the Texas Highway Department in cooperation with Texas Transportation Institute (Ref 4.3). The Delamtect is based on the same principles as the audio method. It has the ability to emit a sound, receive its reflection, and record it. A transducer is used to produce the impacts at the surface of the concrete, and the stress wave that returns is picked up by a receiver, converted into electrical impulses, amplified, filtered, and sent to a processor to be analyzed. The impulses are proportional to the pressure fluctuations produced in the listening wheel by vibrations of the concrete surface. The analyzed data are then sent to a recording device which produces a hard copy of the output.

Capabilities and limitations. The Delamtect provides an effective means of locating large delaminations in exposed concrete members and has an effective evaluation depth of up to the thickness of the concrete cover. The method may miss small delaminations due to transducer placement and is considered to be less accurate than the chain drag method.

Accuracy. Research has indicated that the accuracy of the Delamtect testing method ranges from good to excellent when used to detect voids in a concrete slab (Ref. 4.1). No quantitative information regarding the accuracy is available.

4.2.1.3 WesCet

Description. WesCet operates using principles similar to the Delamtect and has had application to determining the condition of concrete bridges. WesCet, however, utilizes four wheels which roll on the surface to be examined and the impulse wave is transmitted by "shooting" small pellets propelled (122 m/sec) by pressure (7 MPa) from a carbon dioxide cylinder. Examination of the compression waveform is used to determine areas of deteriorated concrete. The received shear wave is used to confirm findings of the compression wave and to provide information on water-filled fractures.

Capabilities and limitations. WesCet is capable of locating deteriorated concrete, concrete delaminations, cracks, and water-filled fractures in concrete thicknesses up to 30.5-mm (Ref. 4.4). The process is time consuming; however, a modified version of the method can be used to perform vertical surveys of structures.

Accuracy. No information regarding the accuracy of this method is available.

4.2.2 Electrical Methods

Electrical methods use resistance and potential difference measurements of a structure to determine the moisture content, estimate thicknesses, estimate probability of significant corrosion, and indicate the current rate of corrosion. No information regarding the accuracy of electrical methods is available.

4.2.2.1 Four-Electrode Method

Description. The four-electrode method (Ref. 4.5) measures the resistivity of concrete and relates it to the probability of presence of steel corrosion. The method utilizes four contact points,

equally spaced in the concrete at a depth of approximately 6.4 mm, which are aligned as shown in Fig. 4.1. Alternating current is passed through the outer electrodes and the potential difference determined using the inner electrodes. The resistivity is determined by (Ref. 4.5):

$$\rho = 2\pi\alpha E/I$$

where

ρ = resistivity (ohm-cm),
α = electrode spacing (cm),
E = potential drop between two inner electrodes (Volt), and
I = current flow between outer two electrodes (amp).

The resistivity measurements can be used to estimate the probability of corrosion.

Capabilities and limitations. Studies to relate the resistivity of concrete to the probability of steel reinforcement corrosion have been conducted in England (Ref. 4.6). Moist concrete has a resistivity of about 10^4 ohm-cm and oven-dried concrete a value of about 10^{11} ohm-cm. In general it has been shown that if (Ref. 4.8):

(1) $p > 12{,}000$ ohm-cm, corrosion is unlikely;
(2) 5,000 ohm-cm $< p < 12{,}000$ ohm-cm, corrosion will probably occur; and,
(3) $p < 5{,}000$ ohm-cm, corrosion is almost certain to occur.

One drawback of the method is that the resistivity measurements are limited to relatively close to the concrete surface. Increases in probe spacings, which would allow evaluations at deeper concrete depths, frequently result in the steel reinforcement interfering with the results obtained (Ref 4.5). Also, the method is still in the experimental stages and a complete field model has not been developed.

4.2.2.2 Copper-Copper Sulfate Half Cell.

Description. The copper-copper sulfate half cell consists of a copper rod submerged in a capsule filled with copper sulfate as shown in Fig. 4.2. An electrical connection is made to the reinforcement at a convenient position enabling electrode potentials to be measured at any desired location by moving the half cell over the concrete surface in an orderly manner. The value of potential measured is used to estimate the likelihood of corrosion but cannot indicate the corrosion rate or amount of corrosion present (Ref. 4.7). The method is described in detail in Ref. 4.8.

Capabilities and limitations. Accepted values of interpreting readings from the copper-copper sulfate half cell are contained in Ref. 4.8 and include the following:

(1) if potentials over an area are more positive than -0.20 V saturated copper-copper electrode (CSE), there is a greater than 90% probability that no reinforcing steel corrosion is occurring;
(2) if potentials over an area are in the range of -0.20 V to -0.35 V CSE, corrosion activity of the reinforcing steel in that area is uncertain;
(3) if potentials over an area are more negative than -0.35 V CSE, there is a greater than 90% probability that reinforcing steel corrosion is occurring in that area; and
(4) positive readings, if obtained, generally indicate a poor connection with the steel, insufficient moisture in the concrete, or the presence of stray currents and should not be considered valid.

The copper-copper sulfate half cell may produce false readings because of the temperature- and moisture-dependence of the electrochemical process (Ref. 4.2).

4.2.3 Impulse Radar

Description. The impulse radar equipment, shown schematically in Fig. 4.3, consists of three basic components: main radar unit, graphic printer or computer-aided analyzer, and antenna. The main radar unit is the primary processing section of the radar equipment. The graphic printer provides a two-dimensional output image with a time scale displayed on the y-axis and a scan travelled on the x-axis. The antenna houses the transmitter as well as the receiver. New technology has made coloring capability of the output possible in addition to numerous functions such as filtering, arithmetic summation, etc. The impulse radar system radiates short-time-duration electromagnetic pulses (0.5 to 1.5 nanoseconds) into the material to be examined from a broad-band antenna, which is electromagnetically coupled to the material's surface (Ref. 4.1).

The impulse radar technique is based on principles of electromagnetic wave reflection. A contacting transducer the size of a hand-held calculator is passed over the surface and emits an electromagnetic wave into the structure. When the electromagnetic wave travels through different mediums (different dielectric constants), a portion of the wave is reflected back to the antenna and the remainder of the wave is refracted into the next medium. Reflected pulses are received by the transducer and electronically processed. The loss of strength of the reflected wave form, measured in volts, is used in the evaluation of data. When locating voids in concrete, these pulses are reflected by the difference in the dielectric constants of the concrete and air.

There are two types of ground penetrating radars available today — the Geophysical System Survey Incorporated (GSSI) radar unit and the Penetradar Systems radar unit. The two types are manufactured by GSSI, Inc., Salem, N.H., and Penetradar Corp., Niagara Falls, N.Y., respectively. The Penetradar System radar unit is more sensitive to flaws occurring at the surface while the Geophysical System Survey Incorporated radar unit is more sensitive to flaws occurring at greater depths. The Penetradar System radar uses a horn-shaped antenna while the Geophysical System Survey Incorporated radar unit uses a variety of different shaped antennas which have to be in contact with the surface. Different types of antennas allow for varying depths of penetration.

Reference 4.9 presents a description of the method for determining slab thickness.

Capabilities and limitations. Impulse radar has a wide range of applications in the testing of concrete, e.g., detection of delaminations, voids, dowel bar alignment, etc. The technique also shows potential for other applications such as monitoring of cement hydration or strength development in concrete, studying the effect of various admixtures on concrete curing, determination of water content in fresh concrete, and measurement of thickness of concrete members (Ref. 4.10). The radar waveform depth of penetration is influenced by the moisture present and the amount and type of reinforcement. Use of a radar antenna with high resolution or short pulse width (< 1 ns), permits concretes as thin as 3.1 to 6.1 cm to be examined. For dry and unreinforced concrete, the radar can penetrate to depths of approximately 0.6 m to detect defects and indicate their depths.

Accuracy. Investigators have reported that the accuracy of radar in locating delaminations ranged from 91% (Ref. 4.11) to 80% (Ref. 4.12), with the radar occasionally giving an indication of a delamination where the concrete was actually sound. Radar also failed to indicate some delaminated areas 0.3 m wide or less. The accuracy of the short-pulse radar in determining thickness of a concrete slab has been investigated (Ref. 4.13). Figure 4.4 shows that thickness estimates compare favorably with the lengths of cores extracted from the slab. The absolute errors ranged from 0 to 2.25 cm. This corresponds to accuracy of approximately 100% to 91%. No information regarding the probability of the radar method detecting a flaw of specific size and orientation is currently available.

4.2.4 Infrared Thermography

Description. Infrared thermography is based on the theory of heat transfer. Subsurface anomalies in a material affect heat flow through that material and can be located by measuring surface temperatures. A thermographic scanning and analysis system, such as shown schematically in Fig. 4.5, consists of four subsystems: (1) infrared scanner head and detector,

capable of detecting temperature variations as small as 0.1°C; (2) real-time microprocessor coupled to a monitor which displays different temperature levels as different colors or as gray-tone images when a black and white display is used; (3) data acquisition and analysis equipment consisting of an analog-to-digital converter, computer with high resolution color monitor, and data storage and analysis software; and (4) image recording and retrieving devises (Ref. 4.14). Delaminated areas of concrete, or areas where voids are present, do not transmit heat as fast as nondelaminated areas. Also, the magnitude of temperature differences between undelaminated and delaminated areas (or areas containing voids) provides an indication of the depth of the defect, i.e., larger temperature differences are associated with shallower depths.

Capabilities and limitations. Infrared thermography is capable of detecting concrete subsurface voids, delaminations, and other anomalies. It has advantages over other nondestructive testing techniques in that it covers a large test area in a short time. For best results, small calibration cores should be utilized which have known areas of good quality and defective concrete. In order to perform an infrared thermographic inspection, a movement of heat must be established in the structure. Parameters which affect surface temperature measurements include solar radiation, cloud cover, ambient temperature, wind speed, and surface moisture (Ref. 4.14). Although infrared thermography is very reliable in locating concrete defects, it does not provide information on the cause and extent of deterioration or the depth or thickness of a void. It can, however, determine the location and horizontal dimensions of a defect in concrete.

Accuracy. Reference 4.15 indicates that this testing technique is quite accurate in locating delaminations. An accuracy of approximately 97% in locating delaminated areas is reported.

4.2.5 Magnetic Methods (Induction)

Description. Magnetic induction is only applicable to ferromagnetic materials and the equipment consists of a primary coil connected to a power supply delivering a low frequency (10 to 50 Hz) alternating current and a secondary coil which feeds into an amplifier circuit (Refs. 4.16 and 4.17). When a ferromagnetic test object, such as a steel rebar, is introduced near the coils, a much higher secondary voltage is introduced with the amplitude of the induced signal being a function of the magnetization characteristics, location, and geometry of the object. The induction principle resulted in the development of equipment for determining the location, size, and depth of steel reinforcement in concrete. These instruments are commonly known as cover meters, pachometers, or R-meters (Ref. 4.5).

Capabilities and limitations. Cover meters can detect reinforcement as deep in concrete as 30.5 cm. Although the meters can be used with a variety of probes, the probes are highly directional and the meter must be calibrated for each probe, i.e., the relationship between the induced current and distance from the probe to the reinforcement is not linear because the magnetic flux intensity decreases with the square of the distance. Reference 4.16 notes that rebar sizes between 9.5 and 57 mm can be identified using the devices. The accuracy of the magnetic induction techniques is reduced by factors which affect the magnetic field within the range of the instrument, e.g., presence of multiple rebars, metal tie wires, bar supports, iron content of cement, etc. Also, the magnetic field is temperature dependent and the method should not be used below 0°C.

Accuracy. Accuracy of 98% up to depths of 15.2 mm for any bar size has been indicated, with a bar size accuracy of 90% to a concrete depth of 20 cm (Ref. 4.16). However, there is a reduction in accuracy as depth of embedment increases.

4.2.6 Stress Wave Reflection/Refraction

Stress wave reflection/refraction utilizes stress waves to make inferences about internal conditions of concrete structures based on the effect the structure has on the wave (Ref. 4.18). Striking of an object with a hammer, as shown in Fig. 4.6, and listening to the "ringing" sound is a common method for detecting the presence of voids, cracks, or delaminations. Stress wave propagation methods include pulse-echo, impact-echo, and stress wave refraction.

4.2.6.1 Pulse-Echo

Description. In the pulse-echo method a stress pulse is introduced into an object at an accessible surface by impact. As the stress wave propagates into the test article, flaws or interfaces will cause reflections. The surface response caused by the arrival of the reflected waves is monitored by either the transmitter acting as a receiver (true pulse-echo) or by a second transducer located near the pulse source (pitch-catch) (Ref. 4.18). Receiver output is displayed on an oscilloscope, and, by using the time base of the display, the round-trip travel time of the pulse is determined. If the wave speed of the material is known, the travel time can be used to determine the depth of the reflecting interface.

Capabilities and limitations. Pulse-echo techniques have been used to measure the thickness of thin concrete sections and to identify and locate defects and discontinuities. Concrete, because of its extensive pore system, the presence of cracking, and its heterogeneous nature, causes multiple reflections when very high frequency pulses are used. This makes interpretation of results difficult. Development of a practical pulse-echo system for concrete has been hindered by difficulties in developing an adequate low-frequency transducer which can emit a short duration pulse (Ref. 4.18). Optimum performance requires calibration and referencing. Reference 4.19 indicates that, for pulse-echo testing using a mechanically generated stress pulse, the object dimensions must meet the following requirements: (1) the thickness must be large enough so that specular reflections arrive at the receiver after the source wave and (2) the lateral dimensions must be large enough so that reflected surface waves will not interfere with the initial portion of the signal from specular reflections.

Accuracy. The accuracy of the pulse-echo technique in measuring the thickness of a 0.25-m-thick concrete slab specimen is approximately 98% (Ref. 4.20).

4.2.6.2 Impact-Echo

Description. An impact-echo system consists of an impact source, a receiving transducer, and a digital processing oscilloscope or waveform analyzer which is used to capture the transient output of the transducer, store the digitized waveforms, and perform signal analysis (Ref. 4.18). On impact a stress pulse is introduced into the test article as spherical wave fronts (P- and S-waves). In addition, a surface wave (R-wave) travels away from the point of impact. Reflections of the P- and S-waves by internal interfaces or external boundaries are sensed at the receiving transducer as displacements and recorded by the digital oscilloscope. Examination of amplitude spectra from scans taken at several positions on the test article surface are used to indicate the depth and approximate size of defects.

The differences between the pulse-echo and impact-echo test methods are the transducers and receivers that are used. The pulse-echo testing method uses a piezoelectric crystal for its driving force while the impact-echo testing method uses a mechanical impactor. The impact-echo testing method has the capability of using an instrumented hammer that produces a characteristic transfer function of the structure. This allows the relationship of the input pulse and the output signal to be obtained at the receiver.

The pulse-echo method uses one of the following methods as a possible receiver: the same piezoelectric crystal that transmits the pulses, another piezoelectric crystal, or an accelerometer. The impact-echo testing method uses an accelerometer to receive the return signal.

Capabilities and limitations. At the National Institute of Standards and Technology (Gaithersburg, Maryland) the method has been used in the laboratory to detect various types of interfaces and defects in concrete wall and slab structures, including cracks and voids in plain and reinforced concrete, the depth of surface-opening cracks, voids in prestressing tendon ducts, honeycombed concrete, and slab thickness (Ref. 4.21). Complex signal processing is required to interpret the force-time history of the impact.

Accuracy. Review of published literature (Ref. 4.19) addresses this method as being a reliable nondestructive testing method. However, no specific quantitative information is indicated.

4.2.6.3 Stress Wave Refraction

Description. Stress wave refraction uses a hammer to transmit signals into a material. Receivers are located in four directions at equal directions from the impactor. The signal travels through the material as spherical and surface waves, and the time required to travel from the point of impact to the closest transducer is measured (Ref. 4.3). There are usually two available paths which can be taken, a direct path and a refracted path. Good quality concrete is indicated by each of the transducers receiving the impact signal at the same time. Deteriorated concrete will cause an increase in transit time.

Capabilities and limitations. Stress wave refraction is capable of detecting the quality of concrete and the presence of cracks. The presence of rebars can mask the refraction wave produced in the concrete; therefore, transducer positioning is important. The technique has not been widely applied to concrete. Velocity is affected by the moisture state of the concrete and the type of aggregate.

Accuracy. No literature has been found regarding the accuracy of this technique. This could be attributed to its limited use.

4.2.7 Modal Analysis

Description. Modal analysis is a very reliable way of obtaining data to determine response of a structure to either an external or internal load. It is also a very reliable way of predicting response of a structure to potential future catastrophic events.

Modal analysis utilizes the dynamic response of a structure to indicate its condition. Mathematical modeling is used to describe the structure's dynamic or vibrational behavior. The dynamic behavior is used to compare field measured and theoretically obtained vibration modes. The modal analysis involves thorough integration of three components: (1) theoretical basis of vibration, (2) accurate measurements of vibration, and (3) realistic data analysis (Ref. 4.22). The modal analysis testing technique consists of two routes (Ref. 4.22): (1) theoretical route to vibration analysis and (2) experimental route to vibration analysis. The theoretical route consists of three models: (1) the spatial model, (2) the modal model, and (3) the response model. The spatial model gives a description of the structure's physical characteristics such as mass, stiffness, and damping properties. The modal model provides a description of the structure's behavior as a set of vibration modes. It is defined as a set of natural frequencies with corresponding vibration mode shapes and modal damping factors. The response model provides the solution of how the structure will vibrate under given excitation conditions. The response simulation can be used for nuclear power plant concrete structures to evaluate the structural capacity to provide a safe-shutdown in earthquake situations (Ref. 4.23).

The experimental route is used to determine the response of the structure to a certain excitation mode applied to the structure. It is possible sometimes, within limits, to determine the modal and spatial properties. The following excitation techniques have been used in nuclear power plants:

- Ambient loading
- Steady state loading with vibrators
- Impulse loading with rockets
- Impulse/seismic simulation with buried explosives.

For more details regarding each of the excitation techniques, refer to Ref. 4.23.

Capabilities and limitations. Modal analysis testing methods can be used to detect defects or deteriorations in a structure by comparing measured dynamic behavior to expected behavior. Anomalies in measured characteristic shapes can possibly be used to pinpoint defective areas in the structure. Further detailed testing and inspection can then focus on these areas of concern.

Another application is to verify assumptions made in analytical models. Existing material properties such as stress-strain characteristics are not usually known for an aged structure. Other

parameters such as actual support conditions and effect of cracking are not usually known to a high degree of accuracy.

In addition, the modal analysis testing method can assist in establishing a baseline for rapid future assessment of the overall condition of a structure. A set of operational frequencies and characteristic shapes can be measured in the future at regular intervals and the measurements compared to the baseline. Comparison of measurements may indicate any structural defects which develop.

Accuracy. The accuracy in this technique depends on the knowledge and experience of the operator conducting the test and his skills in analyzing the results. Response measuring devices are usually very accurate.

4.2.8 Radioactive/Nuclear Methods

Although radioactive and nuclear have specific and distinct meanings, they are often used interchangeably in the nondestructive testing context to refer to test methods which use the intersection of wave or particle radiation with matter to supply analytic or diagnostic information about the material (Ref. 4.24). These methods fall into three categories: (1) radiometry, (2) radiography, and (3) nuclear.

No literature has been found to quantitatively address the accuracy of these methods. However, the accuracy of these methods depends on the clarity of the image and the knowledge of the operator in interpreting results.

4.2.8.1 Radiometry

Description. Gamma radiometry systems consist of a radioisotope source of gamma rays, the test article, and a radiation detector and counter. Direct transmission (specimen positioned between source and detector) or backscattering (source and detector are next to each other) modes are used to make measurements. When radiation from the source passes through the concrete and reaches the detector, it is converted to a series of electrical pulses which are counted. The resulting count or count rate is a measure of the specimen dimensions or physical characteristics (density and composition). Detailed discussion of procedures for both direct transmission and backscattering methods are given in Ref. 4.25.

Capabilities and limitations. Gamma radiometry provides a rapid and accurate means for assessing the density of concrete. The backscatter method is suitable for use on fresh or hardened concrete and can scan large volumes of concrete continuously. Disadvantages of the backscatter technique are that it requires a license to operate along with appropriate radiation safety protection, a limited depth sensitivity, and it is sensitive to the concrete's chemical composition.

4.2.8.2 Radiography

Description. Radiography systems, shown schematically in Fig. 4.7, consist of a beam from a radiation source, the object examined, and an image collector (Ref. 4.24). These systems employ a photographic film placed on the opposite side of the specimen from the source as a detector. The intensity of radiation passing through the test article varies according to the article's thickness, density, and absorptive characteristics.

Capabilities and limitations. Radiography provides a means for visually examining the internal microstructure of concrete specimens as well as a means for inspecting the quality of construction or materials in concrete structures. Depending on the source, structures up to 1 m thick can be examined and voids as small as 5 mm detected. The technique is rapid and is useful for detecting concrete flaws and the location of steel reinforcement. Radiography is fairly expensive, skilled operators are required as well as a license to operate, and thin cracks or planar defects parallel to the radiation beam may not be detected.

4.2.8.3 Nuclear

Description. Neutron-gamma techniques consist of a source of neutrons, the test article, and a gamma ray collection and counting system. Under neutron bombardment, the various elements in the sample react by: (l) absorption of atoms and prompt emission of characteristic gamma rays; (2) inelastic scattering by atoms, also with prompt emission of gamma rays; and (3) absorption by atoms which decay and emit characteristic gamma rays over a longer period of time (Ref. 4.24). The elements in the material are identified by the characteristic energies of the emitted gamma rays. The number of gamma rays of specific energy released in a specified period of time is related to the amount of the element present in the test specimen. The nuclear method can basically be used to measure the moisture content in a structure. This method relates to the determination of the amount of corrosion present or the amount of moisture present at the time of testing. It has been proven capable of determining the amount of moisture present in the structure as a function of depth (Ref. 4.25).

Capabilities and limitations. Neutron-gamma techniques have potential for field composition measurements on both fresh and hardened concrete. Advantages of the technique include speed, accuracy, and capability to handle large sample sizes. No commercial systems are presently available because of heavy shielding requirements, complex electronics which are expensive, and interactions of elements other than the ones of interest which requires frequent calibrations and can limit the accuracy. This testing technique is not capable of detecting flaws in concrete.

4.2.9 Rebound Hammer

Description. The rebound hammer is probably the most commonly used of the surface hardness methods for nondestructive examination of concrete. To perform a test, a spring-loaded weight is impacted against the concrete surface and the rebound distance is measured on an arbitrary scale marked from 10 to 100 (Ref. 4.26). The strength of concrete is proportional to the rebound number, i.e., higher strength concretes will produce higher rebound numbers. A description of the test method is provided in Ref. 4.27.

Capabilities and limitations. The primary usefulness of the rebound hammer is in assessing concrete uniformity in-situ, delineating zones (or areas) of poor quality or deteriorated concrete in structures by comparing one concrete with another, and indicating changes with time of concrete characteristics. Advantages of the technique are that user expertise requirements are minimal and a large amount of data can be developed quickly and inexpensively. Test results, however, are affected by smoothness of test surface, specimen geometry and rigidity, moisture condition of concrete, type coarse aggregate, type cement, type mold, and carbonation at concrete surface (Ref. 4.28).

Accuracy. Coefficients of variation for compressive strength determinations on a wide variety of specimens using a rebound hammer averaged 18.8% and exceeded 30% for some groups of specimens (Ref. 4.29). Under laboratory conditions and when a proper calibration curve has been developed, results of this method may vary from the mean by ±15% and ±20% (Ref. 4.17). However, the probable variation of concrete strength estimation in the field is ±25% (Ref. 4.28). Figure 4.8, obtained from Ref. 4.30, presents compressive strength vs rebound hammer readings for normal weight and lightweight concretes obtained during a laboratory study comparing various nondestructive tests for concrete. Figure 4.9, obtained from Ref. 4.17, also shows a range of data of the rebound number vs 28-day compressive strength where limestone aggregate was used in the mix.

4.2.10 Ultrasonic Pulse Velocity

Description. Ultrasonic pulse velocity methods are based on the fact that the velocity of sound in a material is related to elastic modulus and material density. The test equipment consists of a means of producing and introducing a pulse into the concrete (pulse generator and transmitter) and a means of accurately measuring the time taken for the pulse to travel through the concrete to the

receiver. The pulse can be introduced into the test article using either direct, semi-direct, or indirect methods as illustrated in Fig. 4.10. Of these methods, the direct transmission method is the most desirable since maximum energy of the pulse is transmitted and received. The condition of the concrete is assessed through determination of the pulse velocity and the amplitude of the stress wave at the receiver. The pulse velocity of ordinary concrete is typically on the order of 3660 m/s (Ref. 4.31). The presence of defects in the concrete is reflected through decreases in pulse velocity. A description of the test method is provided in Ref. 4.32.

Capabilities and limitations. Because the pulse velocity depends only on the elastic properties of the material, it is a very convenient technique for evaluating concrete uniformity and quality, i.e., concrete quality is proportional to pulse velocity. Ultrasonic pulse velocity methods can be used to detect internal structure changes in concrete, cracking or voids, and changes due to freezing and thawing or other aggressive environments. By using this method it is possible to determine the dynamic modulus of elasticity, Poisson's ratio, thickness of concrete sections, and estimate in-situ concrete strength within 15 to 20% if good correlation curves have been developed. Factors affecting the results obtained include: aggregate size, type, grading, and content; cement type; water-cement ratio; admixtures; degree of compaction; curing conditions and age of concrete; acoustical contact; concrete temperature; moisture content, size and shape of specimen (finite specimen size); and presence of reinforcement (Ref. 4.31).

Accuracy. Figure 4.11, obtained from Ref. 4.30, illustrates the general relationship between concrete unconfined compressive strength and pulse velocity for normal weight and lightweight concretes. The effect of Poisson's ratio and the effect of moisture on the relationship between unconfined compressive strength and pulse velocity is presented in Figs. 4.12 and 4.13, respectively, which were also obtained from Ref. 4.30. Figure 4.14 shows the relationship between ultrasonic pulse velocity and static Young's modulus (Ref. 4.17).

4.3 DESTRUCTIVE TESTING

Destructive testing can be utilized to indicate the concrete strength, density, and quality; locate voids or cracks in concrete; locate the steel reinforcement and indicate depth of concrete cover, and indicate corrosion of steel reinforcing materials. Destructive testing methods include: (1) air permeability, (2) break-off, (3) core testing, (4) probe penetration, (5) pullout, and (6) chloride-ion permeability.

4.3.1 Air Permeability

Description. Basically the method consists of boring a hole approximately 5 mm in diameter and 40 mm in depth into the concrete; plugging the hole with a rubber stopper; inserting a needle, connected by a hose to a mercury manometer and vacuum pump, through the stopper; extracting the air from the hole; and measuring the rate of recovery of the air to the hole.

Capabilities and limitations. The air permeability method of testing is used for in-situ assessments of the resistance of concrete to carbonation and to penetration by aggressive ions, i.e., diffusion rate phenomena. It can also be used to indicate compressive strength. Factors which influence the test results include length of hose, distance and number of neighboring holes as well as whether they are plugged or unplugged, diameter of test hole, and moisture content of concrete (Ref. 4.33). Results presented in Ref. 4.33 indicate that the recovery speed of the mercury head increases as the depth of carbonation increases and decreases as the concrete strength decreases.

Accuracy. Literature regrading the accuracy of this method are not currently available.

4.3.2 Break-Off Method

Description. The break-off method is one of the few methods that measures a direct strength parameter for in-place concrete (others are core testing discussed in Section 4.3.3 and the pullout method discussed in Section 4.3.5). The method, shown schematically in Fig. 4.15, uses

equipment consisting of a load cell, a manometer, and a hydraulic pump. The concrete test specimen, 55-mm-diam by 70-mm-high, is formed using tubular plastic cylinders cast into the fresh concrete and removed at time of test or by coring the in-place concrete. The procedure consists of placing the load cell in the groove at the top of the concrete surface, Fig. 4.15, and slowly applying load using the hydraulic pump until the specimen fails in flexure (Ref. 4.34). The manometer reading is translated to concrete strength using calibration curves.

Capabilities and limitations. Although the method has primary application to quality control and quality assurance associated with form removal or force transfer in post-tensioning or prestressing systems, it can be used to evaluate existing structures. The method, when used in conjunction with proper calibration curves, has been used to accurately determine in-place compressive strengths for concretes having values ranging from 14 to 56 MPa (Ref. 4.35). Advantages of the method are that it measures the in-place concrete strength (flexural); the test is safe, fast, and simple to perform requiring only one exposed surface; and the results are reproducible and correlate well with the compressive strength of concrete. Disadvantages of the method are that it cannot be used either with concrete mixes having maximum size aggregates greater than 19 mm or structural members having member thicknesses less than 100 mm. Also, the test area must be repaired.

Accuracy. The break-off test method has an accuracy ranging from 60 to 85% depending on the coefficient of variation of the concrete (Ref. 4.35). Figure 4.16 shows the relationship between break-off manometer readings and compressive strength as well as the 90% confidence bands for five different concrete batches (Ref. 4.36).

4.3.3 Core Testing

Description. Core samples obtained from concrete structures provide a direct method for examination of the concrete. When core samples are removed from areas indicating distress, a great deal can be learned about the cause and extent of deterioration through strength and petrographic studies. Requirements and methods for obtaining suitable concrete core samples are provided in Ref. 4.37.

Capabilities and limitations. Applications of concrete core samples include calibration of nondestructive testing devices, compressive strength determinations, chemical analysis, visual examinations, determination of rebar corrosion, detection of the presence of cracks, etc. (Ref. 4.38). Conduction of compressive strength tests provides a direct method for obtaining the in-situ concrete strength. Results are influenced by various factors such as core diameter, slenderness ratio, location, etc. (Refs. 4.39 and 4.40). Since the test is destructive, a sufficient number of specimens, as required by statistical standards (Ref. 4.41), can generally not be removed and tested to ensure that the probability of obtaining a strength less than desired is below a certain level.

Accuracy. The curves in Fig. 4.17 provide the number of cores that need to be tested on site in order to obtain an accuracy with the same confidence level as that obtained under laboratory conditions (Ref. 4.26).

4.3.4 Probe Penetration

Description. Probe penetration techniques, as shown in Fig. 4.18, involve measurement of the resistance of concrete to penetration by a steel probe driven by a given amount of energy. The most common device of this type is the Windsor Probe, consisting of a powder-activated driving unit that propels a hardened alloy probe into the concrete and a gage for measuring depth of penetration. Compressive strength is determined through calibration curves. The method is described in Ref. 4.42.

Capabilities and limitations. The primary application of the probe penetration test is to indicate concrete compressive strength. Advantages of the test are that it is simple to operate and correlates fairly well with core compression strength results. Limitations of the method are related to size of specimen that can be tested (minimum distance between test locations and edge of specimen is 150

to 200 mm; minimum specimen thickness of about three times depth of penetration), results are influenced by aggregate size and hardness, and some uncertainty exists in estimated strength values (Ref. 4.43).

Accuracy. Figure 4.19, from Ref. 4.17, shows a relationship between exposed probe length and 28-day compressive strength of concrete. Linear regression was used to fit the data with 96% correlation coefficient (Ref. 4.17). Figure 4.20 shows the range of variation of strength results with respect to depth of penetration (Ref. 4.17).

4.3.5 Pullout Test

Description. The pullout test, shown schematically in Fig. 4.21, measures the force required to pull an embedded metal insert with an enlarged head from a concrete specimen or structure (Ref. 4.44). The method provides a measure of the shear strength of concrete, which is converted to tensile or compressive strength through correlations. The test method is described in Ref. 4.45.

Capabilities and limitations. The pullout test has been developed primarily for determining the in-place strength of concrete during construction because of the requirement to embed (cast) the insert into the concrete. Recently, new techniques have been developed in which holes are drilled into the hardened concrete and either normal pullout inserts or split-sleeve assemblies/wedge anchors are installed and pulled out (Ref 4.46). The results are affected by the size of the coarse aggregate and a correlation relationship between pullout strength and compressive strength is needed.

Accuracy. Figure 4.22, obtained from Ref. 4.30, presents the relation between compressive strength and pullout force for a normal weight and lightweight concrete. Results of the pullout test have varied from 7 to 10% when compared to compressive strengths obtained from cores.

4.3.6 Chloride-Ion Permeability Test

Description. The chloride-ion permeability test is based on the principle that when a charge is applied to a structure the ions within the structure migrate towards the opposite pole from the applied charge. The applied voltage is then used to measure the permeability of chloride ions in concrete (Ref. 4.47). The device used is called the "Applied Voltage Cell" and is shown schematically in Fig. 4.23. The cell is comprised of two reservoirs of approximately 200 mℓ capacity. The reservoirs are coupled to a 100-mm-diam core, about 50 mm long. One reservoir is filled with 0.3 M NaOH and the other with 3.0% NaCℓ. The core specimen is coated with an epoxy around its periphery so as to maintain a watertight seal. The sample is then vacuum saturated for 18 hours prior to test. A 60 volt DC voltage is applied to accelerate the migration (Ref. 4.47). The change in NaCℓ solution concentration with time is used as a measure of chloride penetration. More details on test procedures to determine chloride-ion penetration are provided in Refs. 4.47–4.49.

Capabilities and limitations. This device has been found to be successful in predicting the penetration rate of chloride ions in concrete samples tested in the field or laboratory (Ref. 4.47).

Accuracy. No quantitative information regarding the accuracy of this method is available.

4.4 Testing Technique Summary

Table 4.1 identifies available nondestructive and destructive test methods for concrete materials and lists the applications of each of the methods, e.g., determination of concrete strength, identification of voids or cracks, etc. Summaries of the principle of operation, main applications, advantages, and limitations for the nondestructive and destructive methods are presented in Table 4.2(a) and Table 4.2(b), respectively.

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Table 4.1 Applicability of different testing methods.

TESTING METHOD \ APPLICATIONS	NONDESTRUCTIVE	DESTRUCTIVE	STRENGTH	DENSITY	VOIDS	CRACKS	CORR. RR Rebars	CORR. PC Prestressing	CHEMICAL	QUALITY	COVER	LOC. OF REBAR
AUDIO	●				●	●					●	
ELECTRICAL	●						●	●			●	●
IMPULSE RADAR	●				●	●					●	●
INFRARED THERMOG.	●				●		●	●				
MAGNETIC	●										●	●
MICROSCOPIC REF.	●									●		
MODAL ANALYSIS	●									●		
NUCLEAR	●						●	●				
RADIOGRAPHY	●				●	●	●	●				
REBOUND HAMMER	●		●	●								
ULTRASONIC TRANS.	●		●	●	●	●					●	●
AIR PERMEABILITY		●			●		●	●				
BREAKOFF		●	●									
CHEMICAL		●					●	●	●			
CORES		●	●	●	●	●	●		●	●	●	●
PROBE PENETRATION		●	●									
PULL-OUT		●	●									
VISUAL	●				●	●	●				●	●

Table 4.2a. Summary of applications for testing methods: nondestructive.

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

Table 4.2a. (cont.)

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

Table 4.2b. Summary of applications for testing methods: destructive.

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

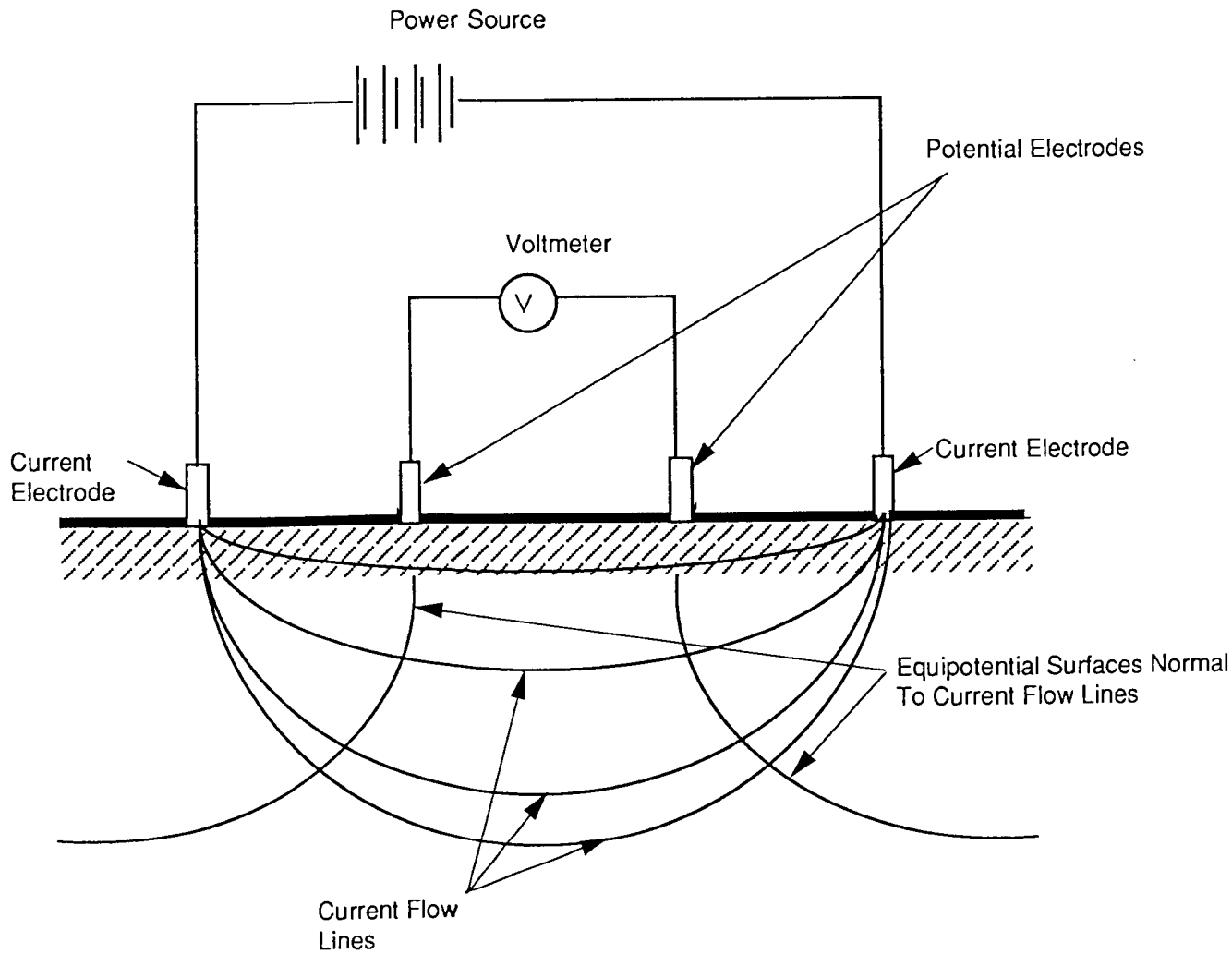

Fig. 4.1 Schematic of the four-electrtods method of resistivity measurement.

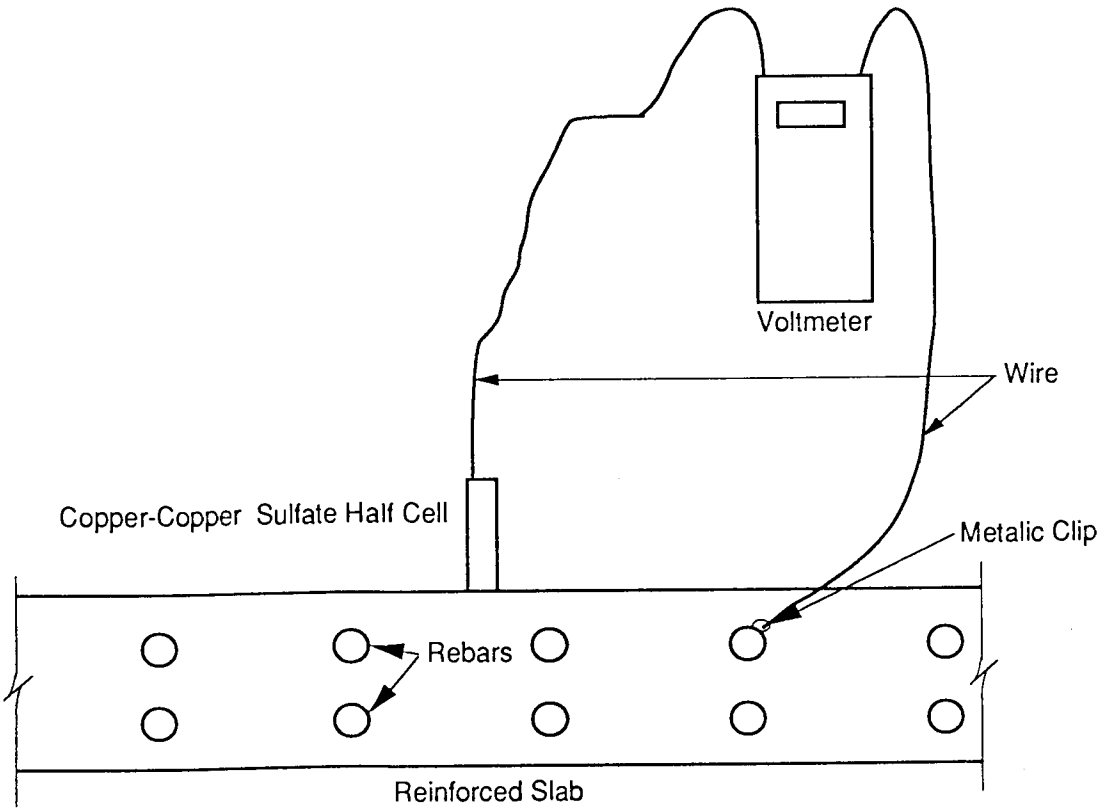

Fig. 4.2 Schematic of copper-copper sulfate half-cell system.

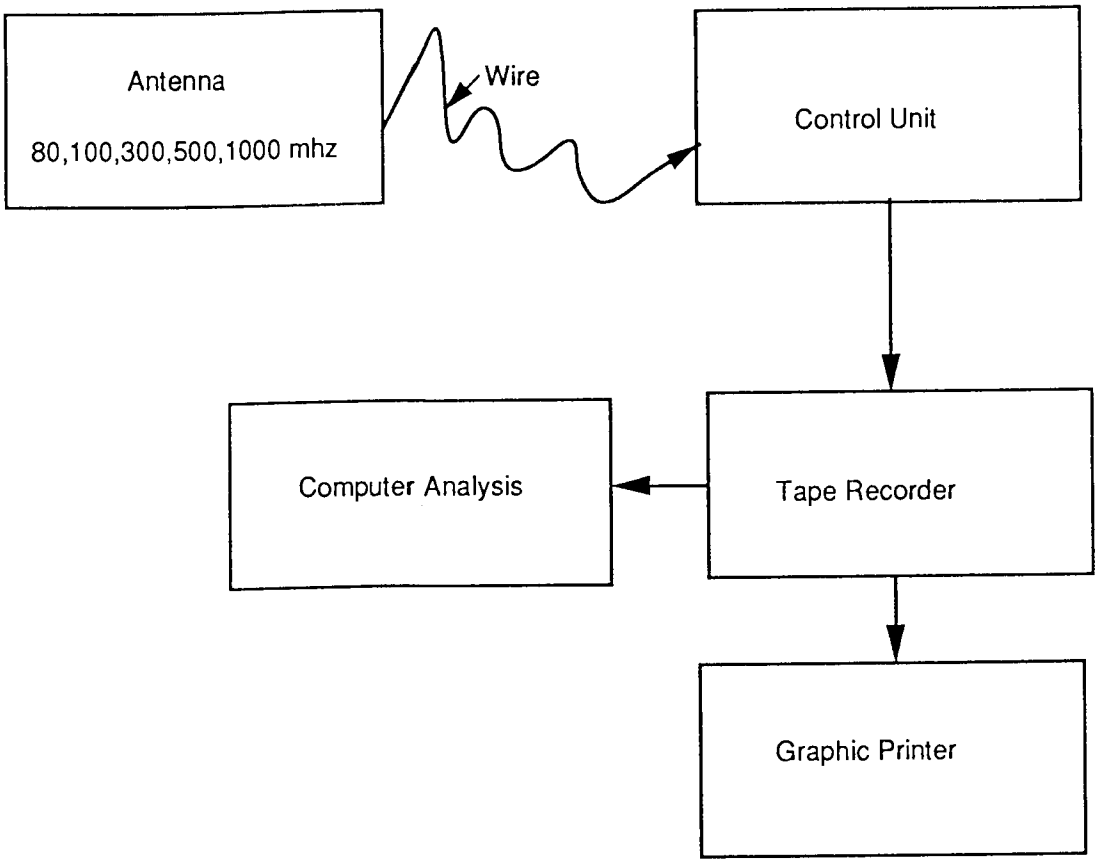

Fig. 4.3 Schematic of the impulse radar system.

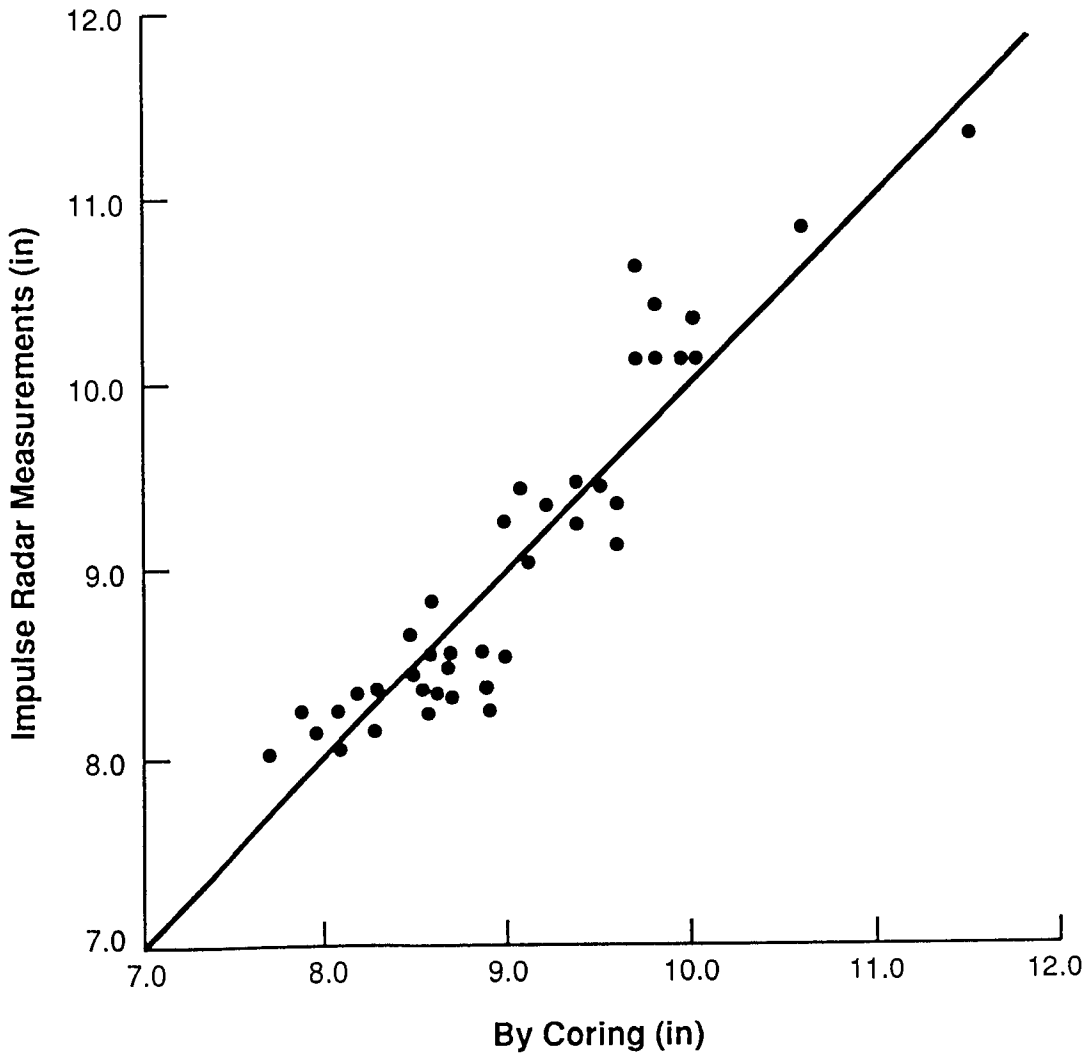

Fig. 4.4 Comparison between concrete slab thickness results as determined by coring and by impulse radar measurement (Ref. 4.13).

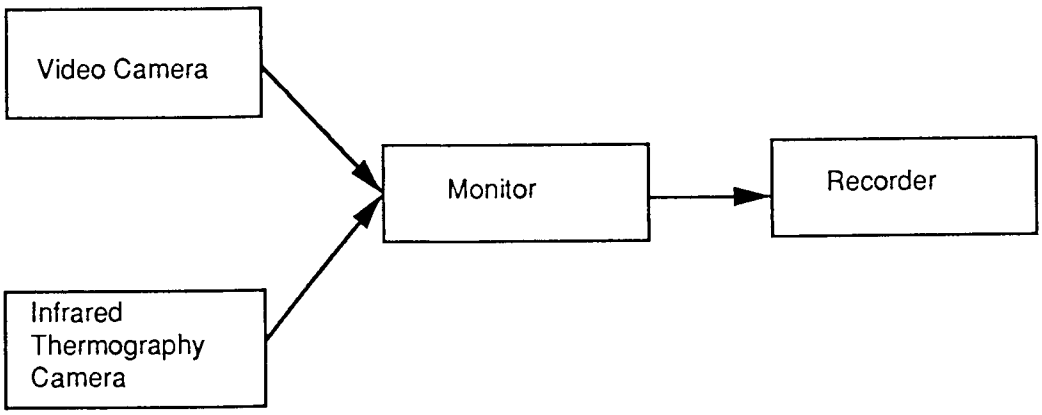

Fig. 4.5 Schematic of the infrared thermography method with video taping capability.

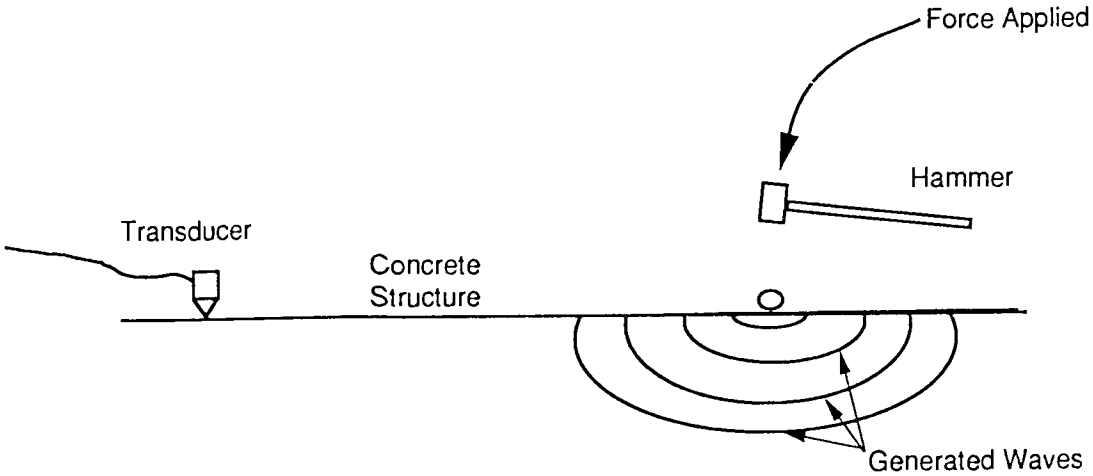

Fig. 4.6 Schematic of microseismic refraction method.

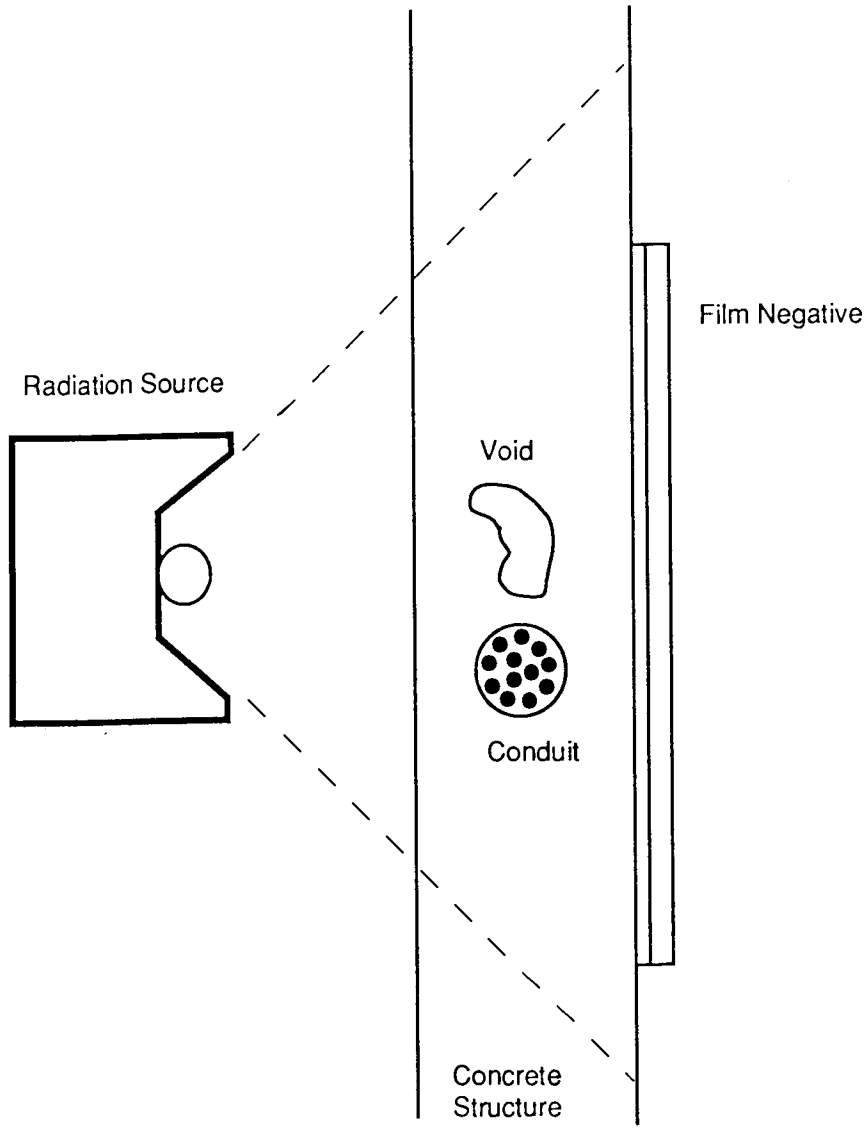

Fig. 4.7 Schematic of the radiography method.

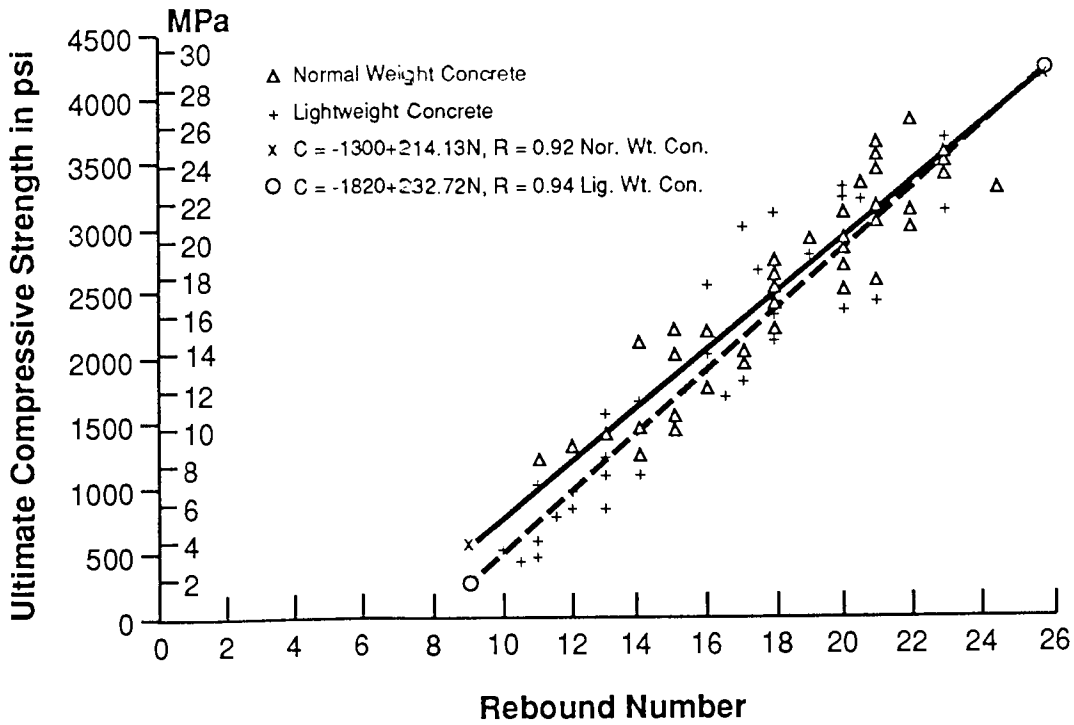

Fig. 4.8 Compressive strength versus rebound number (Ref. 4.30).

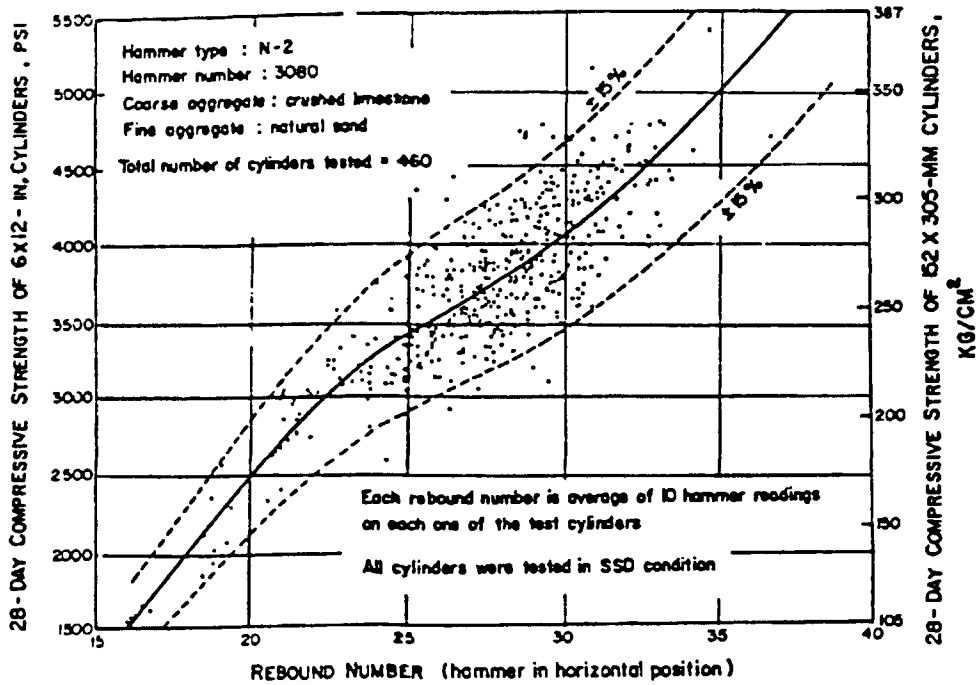

Fig. 4.9 Relationship between compressive strength and rebound number for a limestone aggregate concrete (Ref. 4.17).

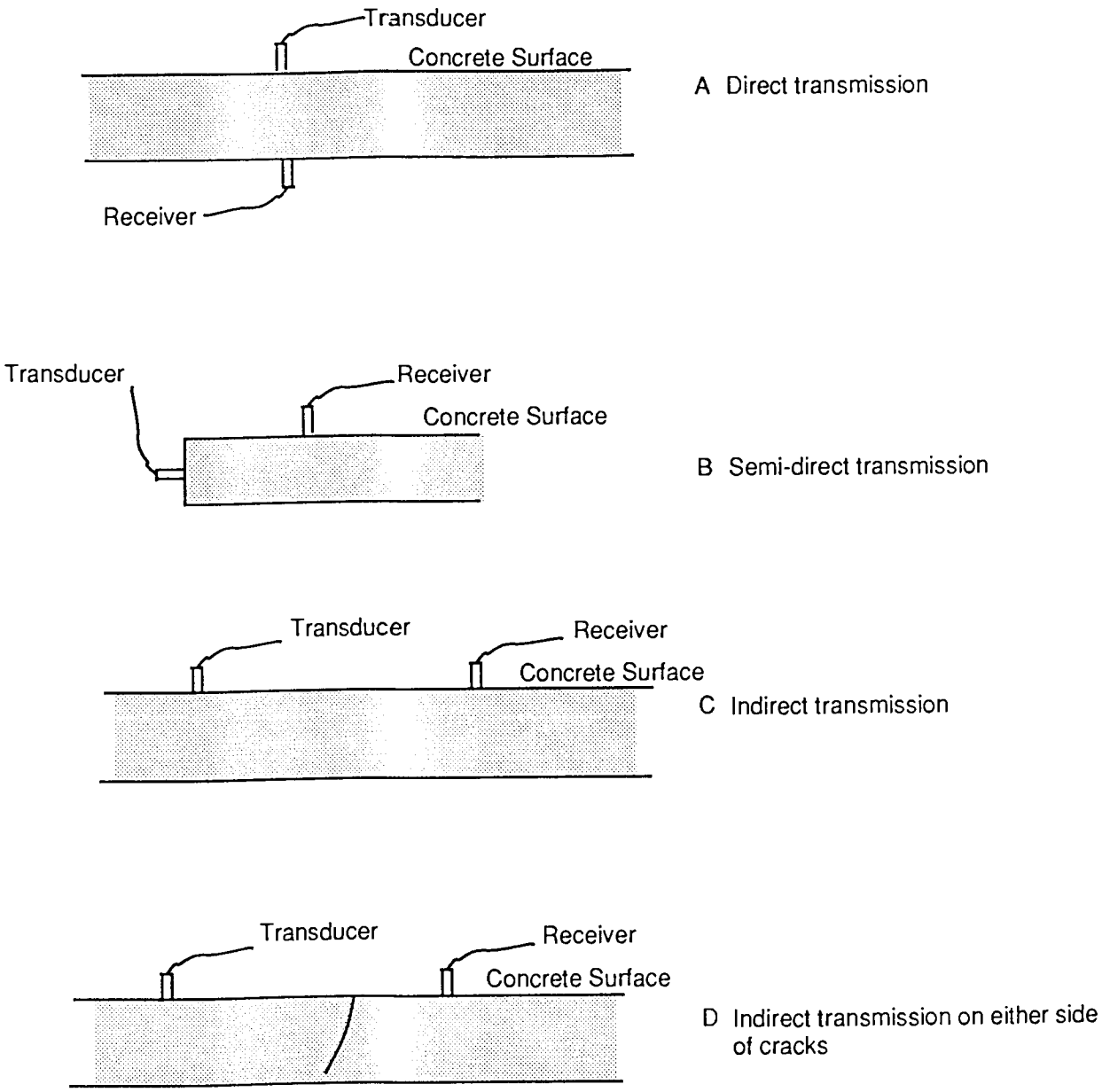

Fig. 4.10 Different methods of ultrasonic transmission.

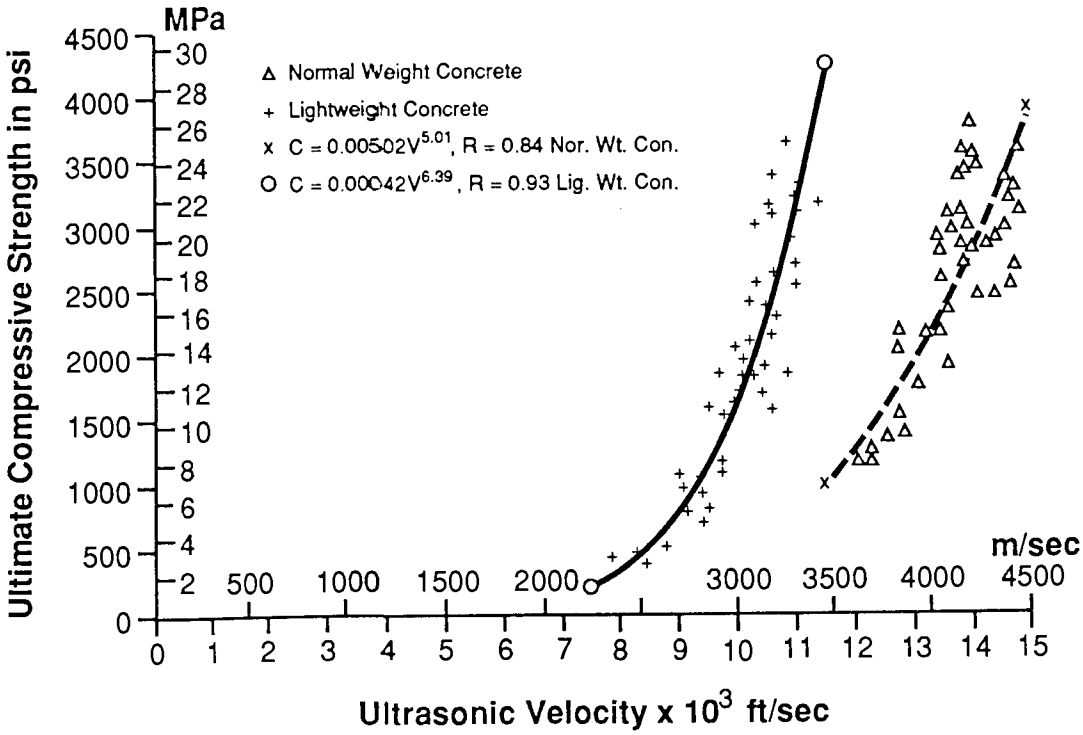

Fig 4.11 Compressive strength versus pulse velocity (Ref. 4.30).

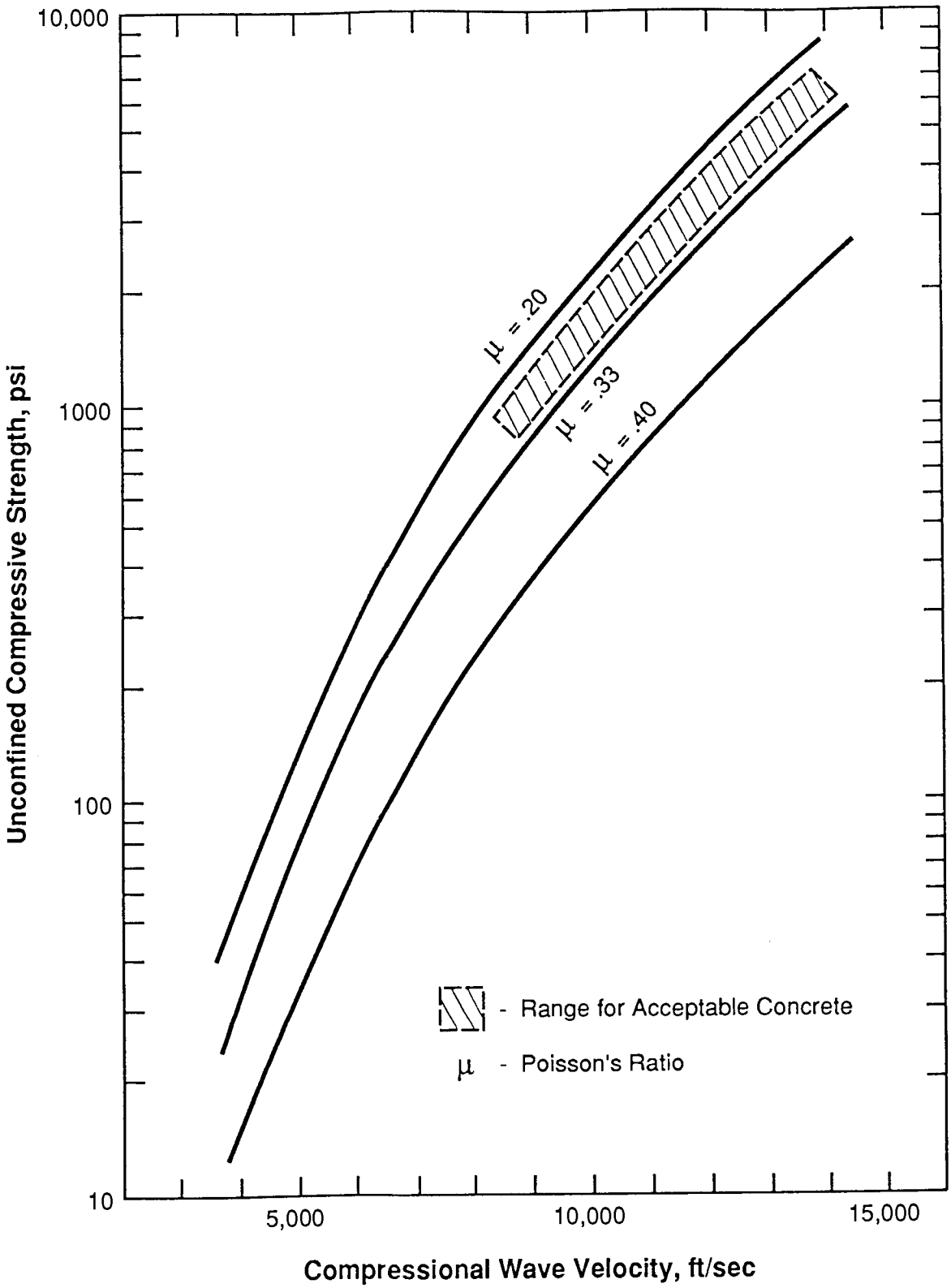

Fig. 4.12 Unconfined compressive strength versus wave velocity (Ref. 4.30).

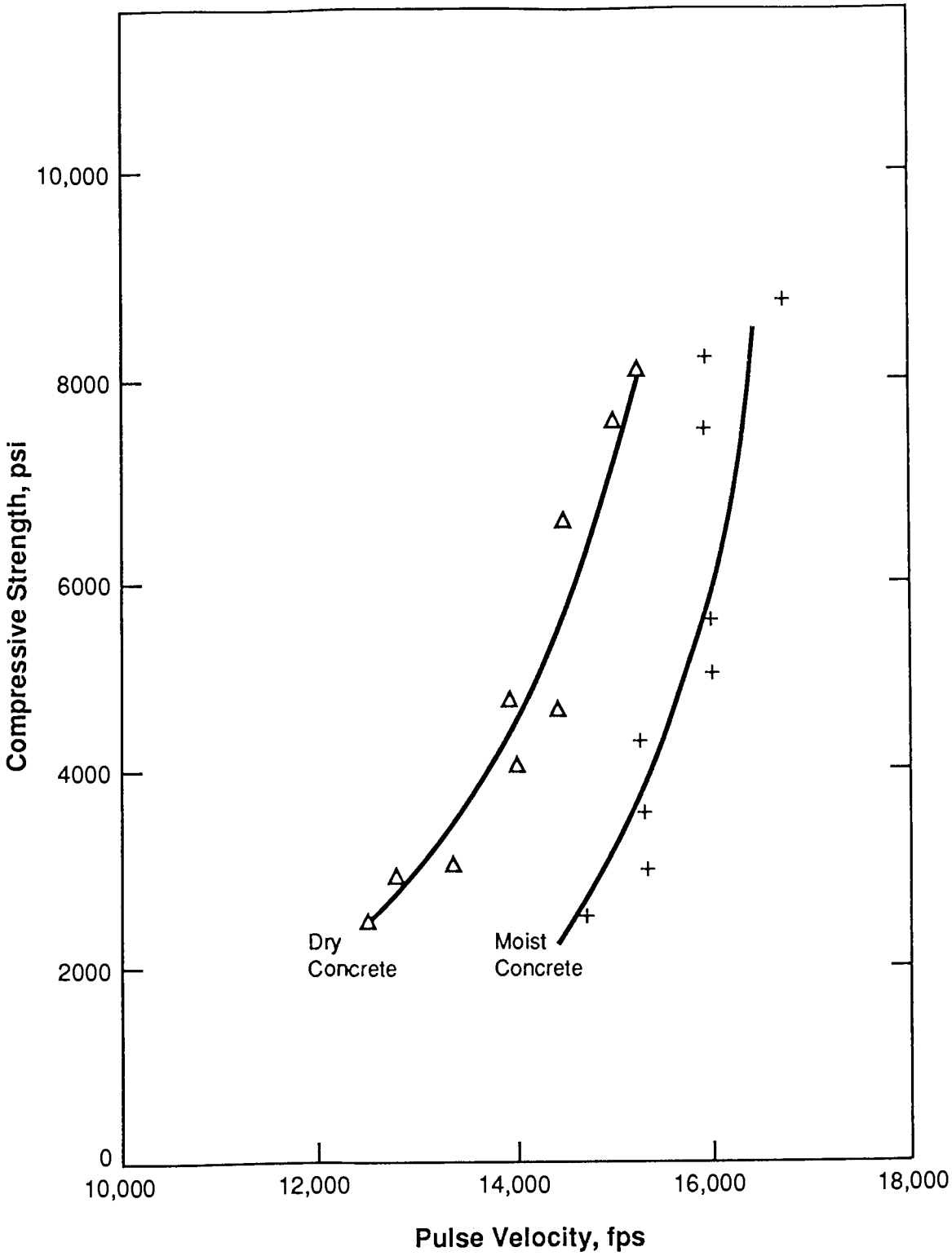

Fig. 4.13 Effect of moisture content on compressive strength versus pulse velocity results (Ref. 4.30).

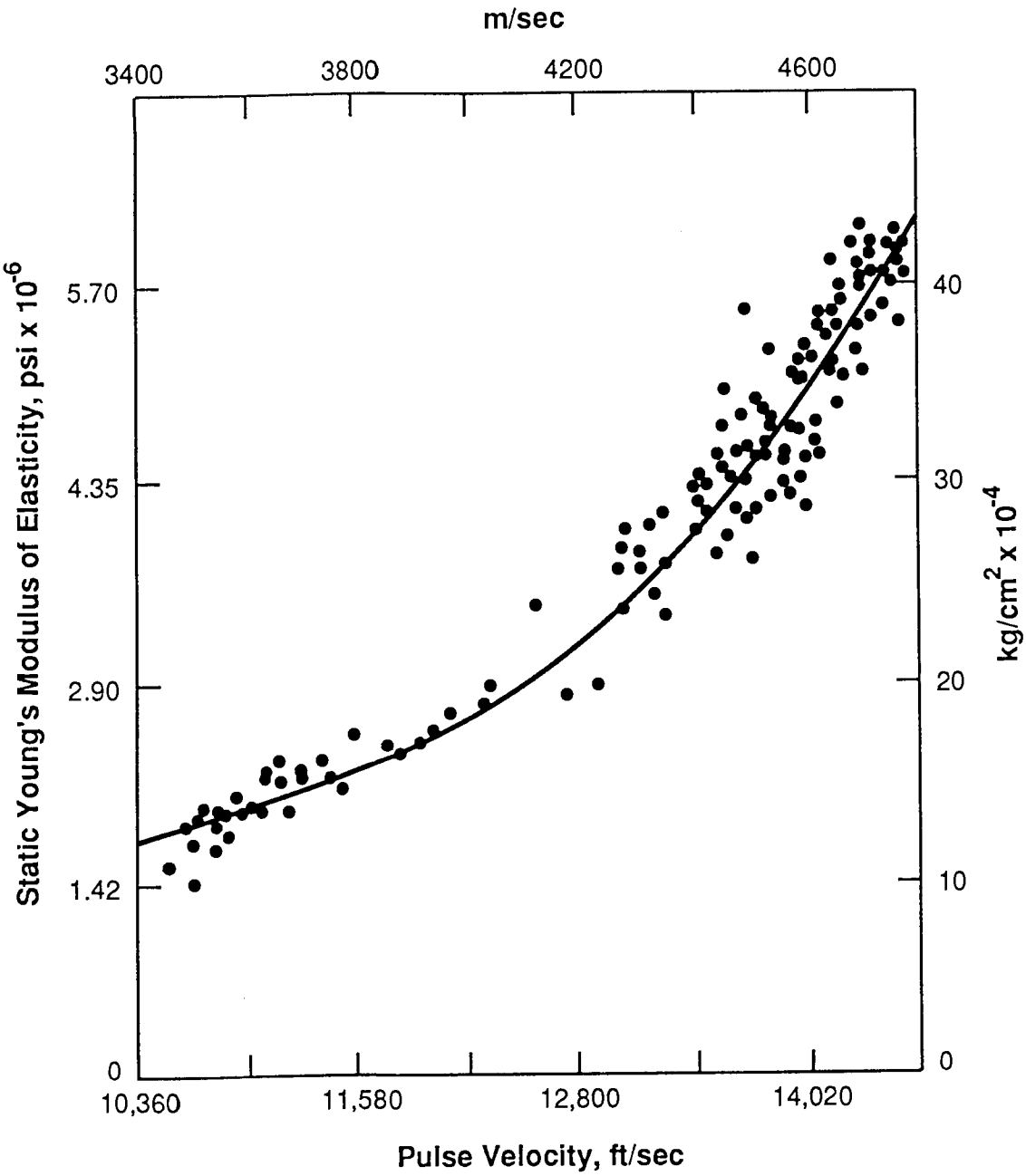

Fig. 4.14 Relationship between static Young's modulus and pulse velocity (Ref. 4.17).

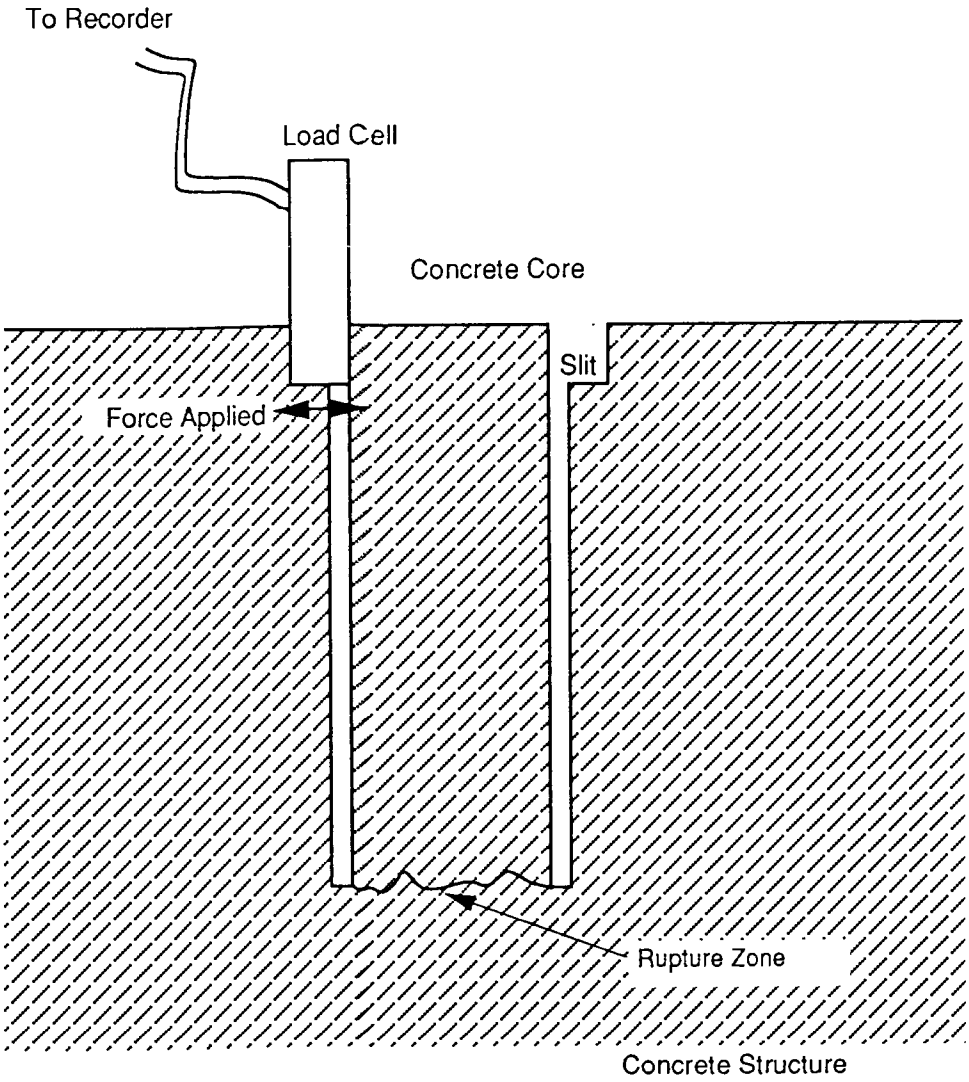

Fig. 4.15 Schematic of break-off test.

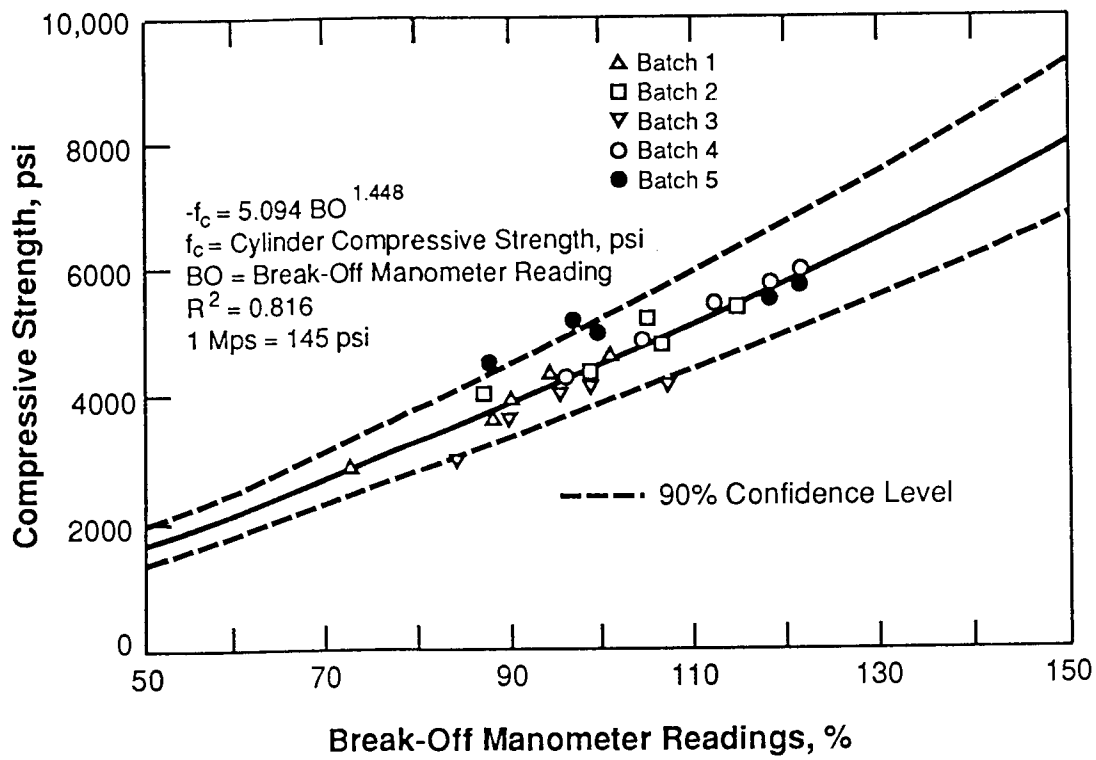

Fig. 4.16 Compressive strength versus break-off manometer readings (Ref. 4.36).

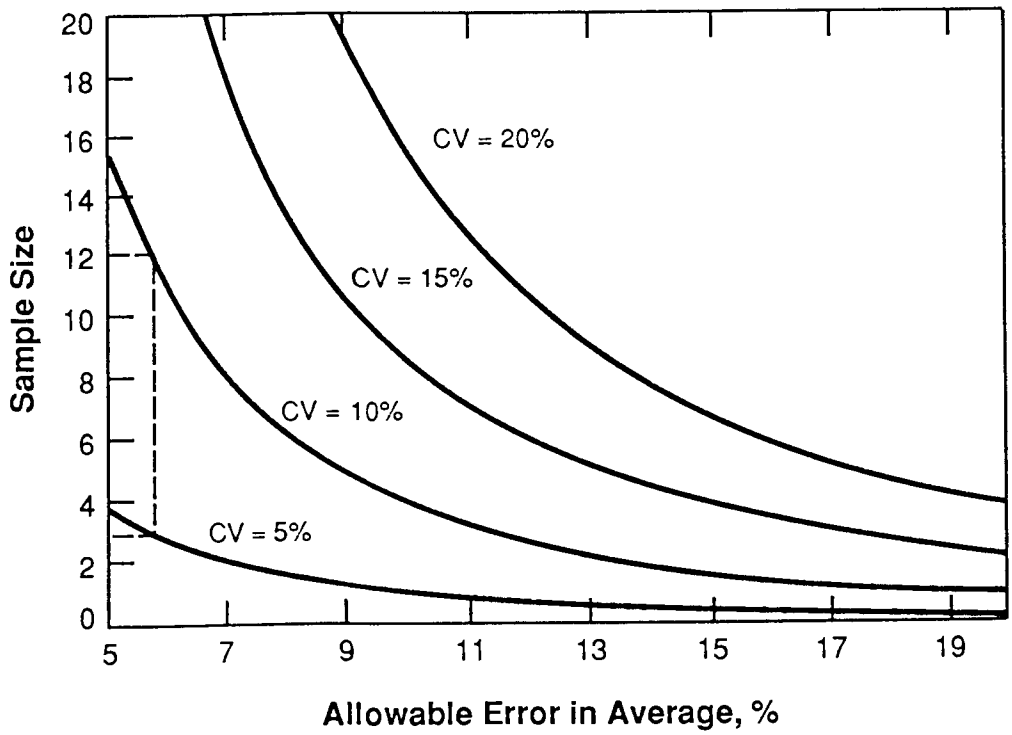

Fig. 4.17 Sample size required to obtain an allowable error that is comparable to that obtained under laboratory conditions (Ref. 4.26).

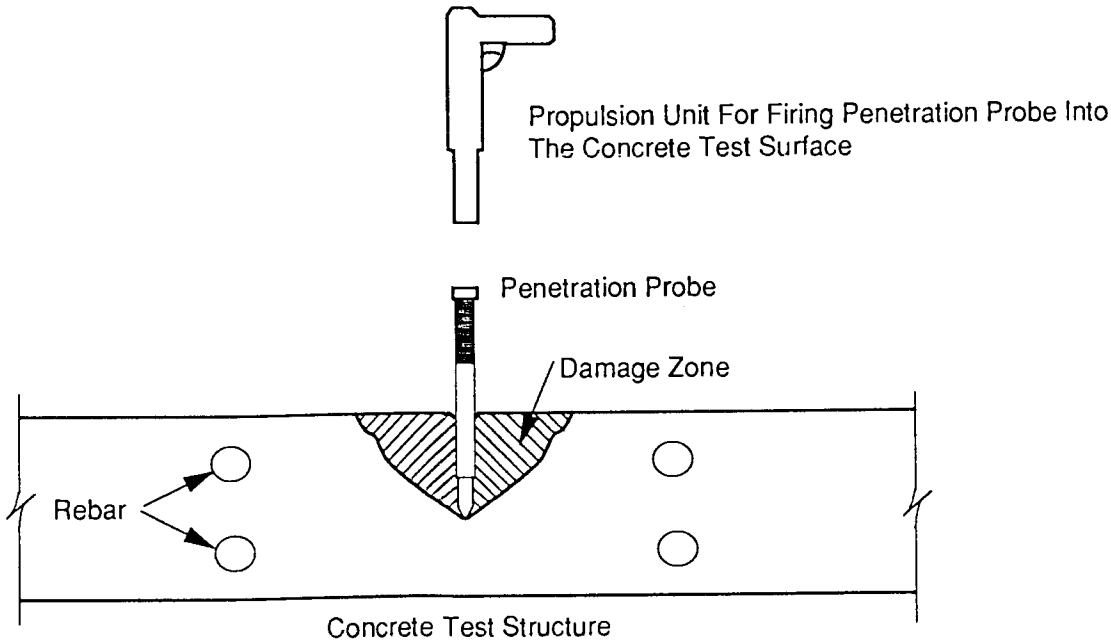

Fig. 4.18 Schematic of probe penetration test.

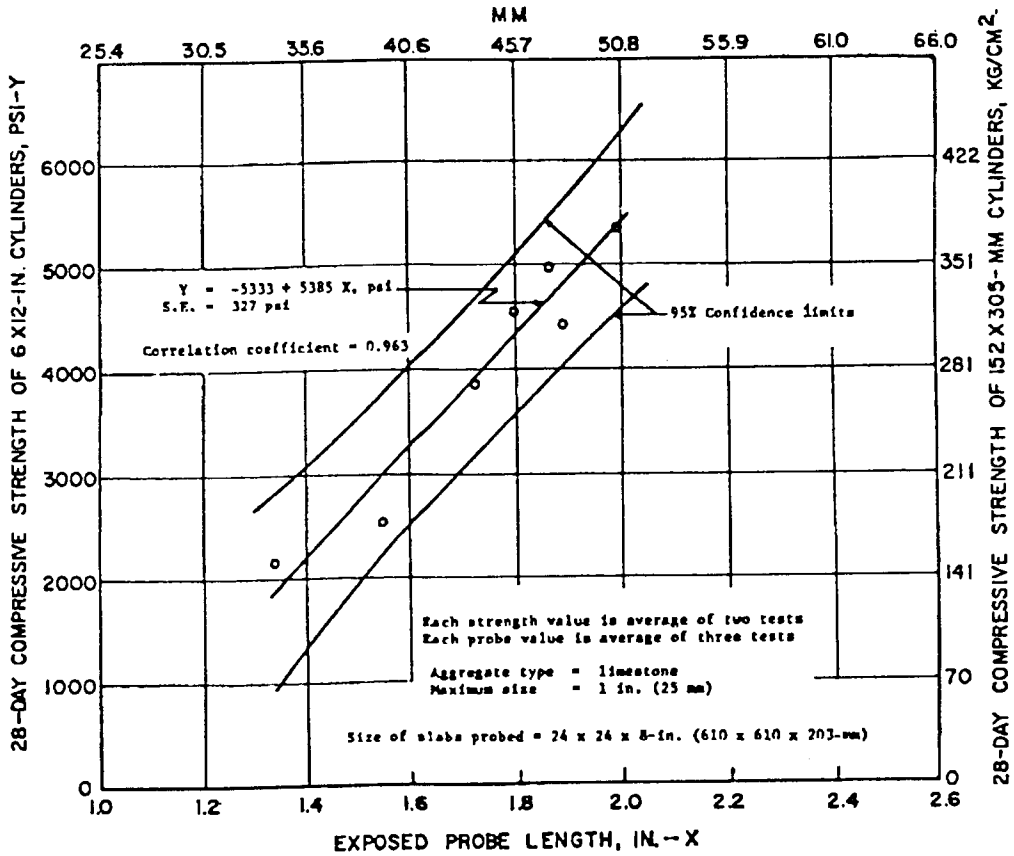

Fig. 4.19 Relationship between 28-day compressive strength of concrete and exposed probe length (Ref. 4.17).

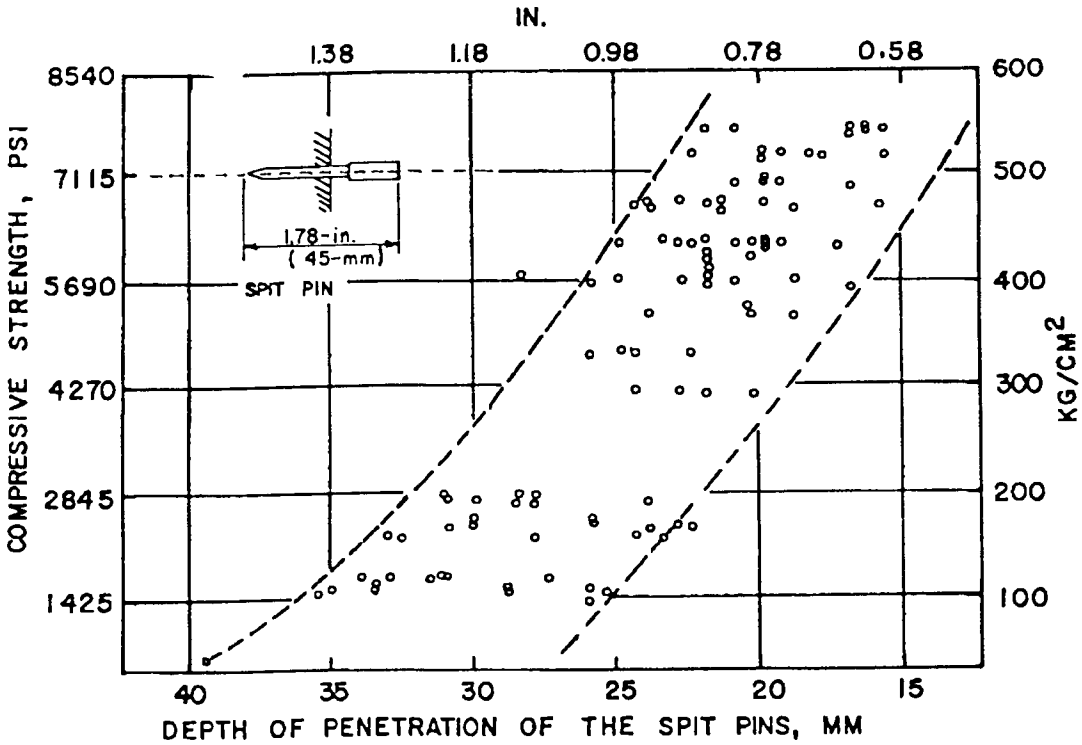

Fig. 4.20 Relationship between compressive strength of concrete and depth of pin penetration (Ref. 4.17).

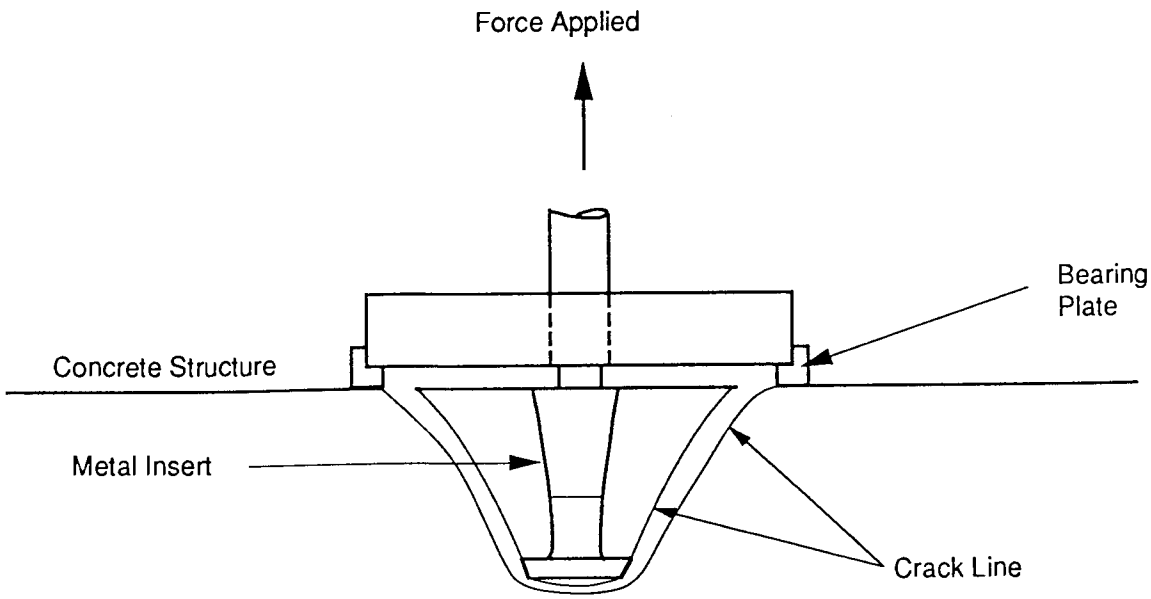

Fig. 4.21 Schematic showing the pullout test.

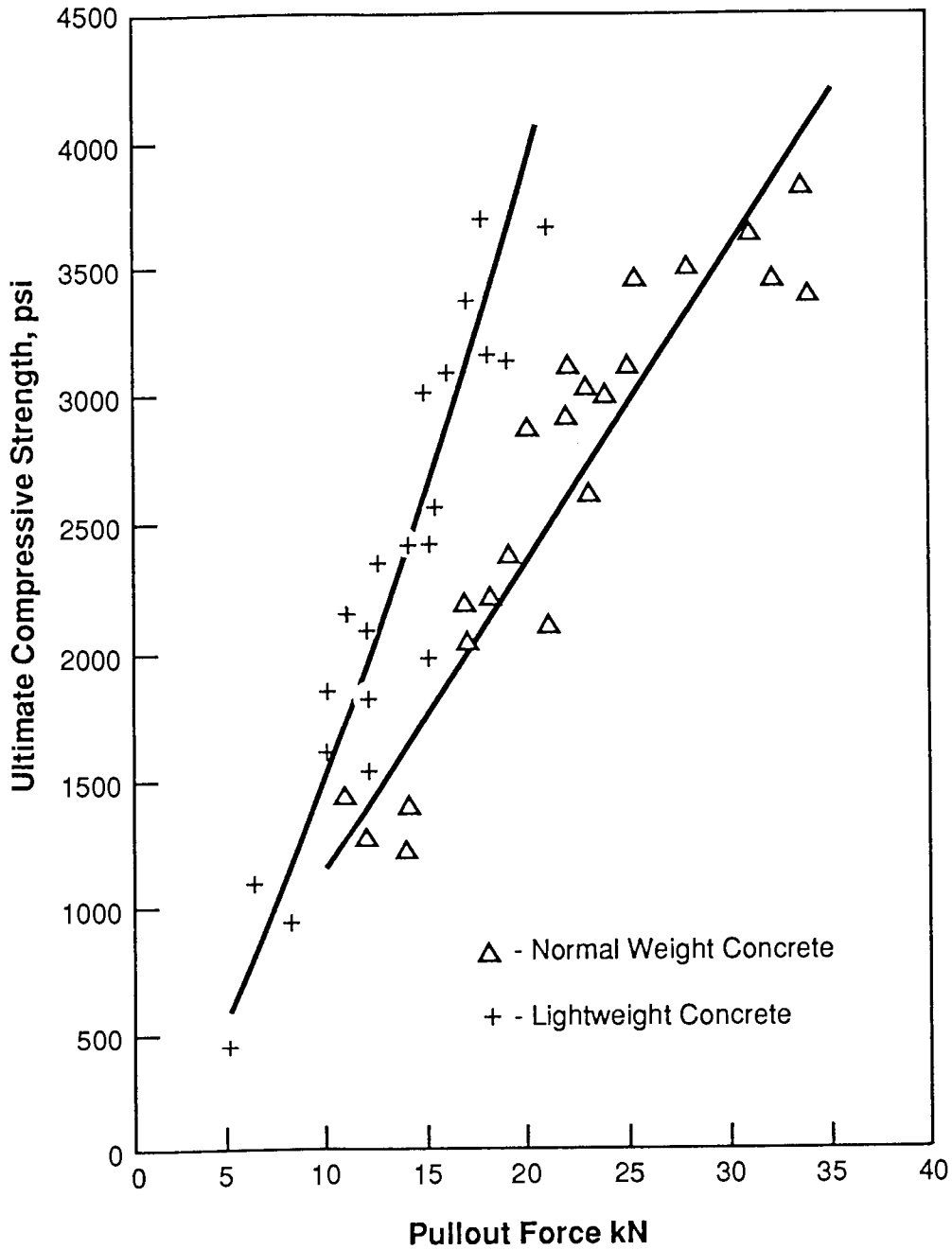

Fig. 4.22 Compressive strength versus pullout force (Ref. 4.30).

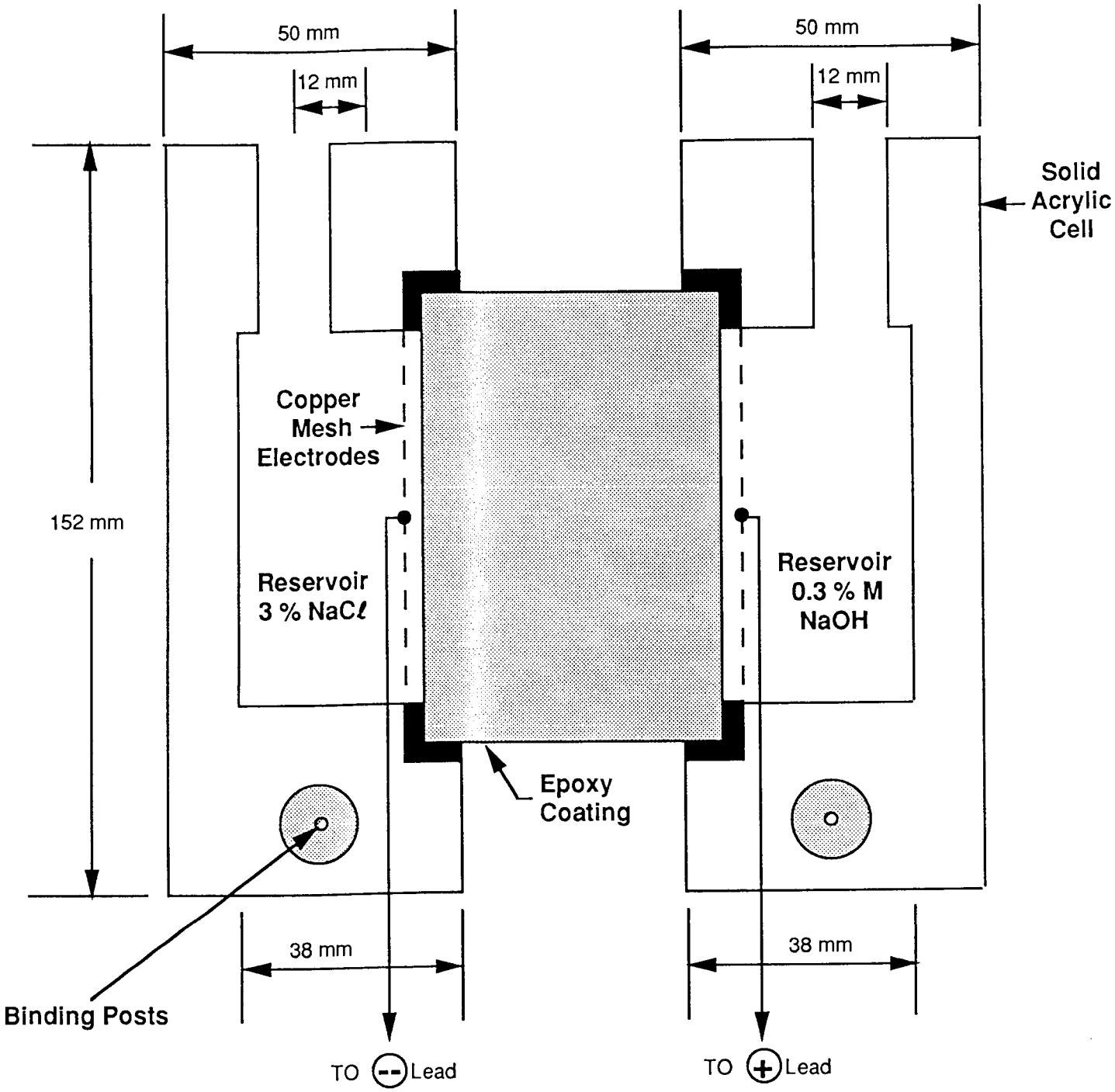

Fig. 4.23 Schematic of applied voltage cell for core specimen (Ref. 4.47).

5. CURRENT IN-SERVICE INSPECTION METHODOLOGIES

5.1 INTRODUCTION

The objective of this chapter is to review the inspection methodologies currently utilized in the assessment of general concrete structures. The inspection techniques discussed also can be applicable to assessment of concrete components in nuclear power plants. The common approach used for inspection and evaluation of an existing concrete structure involves, at first, a general or detailed condition survey of the concrete surface. This may be followed by nondestructive and/or destructive testing to determine material properties and assess extent of degradation. Quite often, the structure or structural component is analyzed under the existing condition of degradation in order to determine its load carrying capacity and investigate necessary repair measures. A final product, a report, is usually prepared documenting findings and results.

5.2 GENERAL METHODOLOGY

One of the objectives of the in-service inspection of concrete structures is to detect any retrogression in physical properties of concrete which could affect the capability of the concrete to meet design requirements in the future (Ref 5.1). The performance of nuclear containment vessels is usually monitored after completion of construction to assure that they function safely and in accordance with the design (Ref. 5.2). The monitoring may be part of the owner's operation and maintenance program.

5.2.1 Routine Inspections

Routine inspections of general civil engineering structures are generally made at a frequency of 6 months to 2 years. These inspections commonly consist of a visual examination of all exposed and accessible surfaces of a structure. In some cases, visual examination may be supplemented by nondestructive tests to indicate properties and conditions of the in-situ concrete at that particular time. These tests may include determination of compressive strength, presence of voids, and cracking. Data from instrumentation embedded in the concrete may also be available. A comparison of the concrete properties, conditions, and instrumentation at each inspection interval can be a useful analysis tool to indicate abnormal changes. To detect aging phenomena and deterioration of concrete, inspections should be made at regular intervals with the interval determined according to existing conditions. These inspections are directed at identifying any changes in condition of the concrete or concrete properties which may affect the integrity of the structure and its future serviceability. A report describing the findings of each routine inspection should be prepared noting any changed conditions and recommending corrective actions. Photographs of changed conditions are an important part of the report. Further in-depth investigations should be initiated if for any reason problems are suspected.

5.2.2 Periodic Inspections

Periodic inspections are similar in nature or objective to routine inspections, except that they may involve a more detailed study. They are usually conducted at a frequency of 2 to 10 years. Periodic inspections generally are associated with higher risk structures such as dams. However, extensive periodic inspections may not be necessary unless changes in the outward appearance of the concrete or concrete structure are noted.

Periodic inspections may include considerable preparation such as dewatering or arranging means for inspecting submerged portions of a structure and excavating inspection trenches. Comprehensive review of instrumentation data, design, and operating criteria may be required for a complete evaluation. In addition, the periodic inspection may include sampling of seepage water,

nondestructive testing, and determination of stress conditions. The amount of investigative work necessary usually depends on the condition of the concrete. The scope of work of the inspection should include identification of causes of any deterioration present.

5.3 CONDITION SURVEYS

As defined in Ref. 5.1, a condition survey is a visual examination of exposed concrete for the purpose of identifying and defining areas of distress. Condition surveys are usually performed prior to conducting any nondestructive or destructive testing of concrete structures. However, sometimes it is necessary to include destructive testing in the condition survey program. The following types of condition surveys are commonly used to assess the general physical condition of concrete.

5.3.1 Cracking Surveys

A cracking survey is an examination of a concrete structure for the purpose of locating, marking and identifying cracks and relating cracks to destructive phenomena. A cracking survey is significant in evaluating the future serviceability of a structure since, in most cases, cracking is the first visual symptom of concrete distress. Monitoring of cracks to determine whether cracks are active or inactive can assist in evaluating a structure's condition. Cracks may occur at later ages of the structure and increase in magnitude with time, e.g., alkali-aggregate reactions. Also, cracks may occur following some unusual event indicating signs of distress, e.g., overload condition.

5.3.2 Surface Mapping

Surface mapping may consist of detailed drawings produced from hand mapping, photographic or movie film mapping, or a combination of these or similar techniques. Crack maps provide a permanent record of the condition of the concrete at the time each survey is made. Items most often identified and mapped include cracking, spalling, popouts, honeycombing, exudation, distortion, unusual discoloration, erosion, cavitation, seepage, condition of joint and joint materials, corrosion of exposed reinforcement, and soundness of surface concrete.

5.3.3 Down Hole TV Camera

The down hole TV camera is a destructive method used for visual inspection of the interior concrete because it requires drilling a hole to the depth of interest. It is often used as a part of the condition survey program. The condition of interior concrete can be examined directly and video-taped, if desired, using this technique (Ref. 5.1). Small transistorized TV cameras are housed in 8-cm and 15-cm-diam probes. These TV cameras are capable of remote focusing and aiming axially as well as in a radial direction. The transmitted picture is continuously displayed on a scanner screen, and can be supplemented by video recording for a permanent record. Reference 5.1 indicates that both the Bureau of Reclamation and Corps of Engineers have used this technique with satisfactory results.

5.4 APPLICATIONS OF NONDESTRUCTIVE TESTING METHODS

Construction Technologies Laboratories, Inc. (CTL) has been involved in evaluation and assessment of different types of concrete structures. Quite often the evaluation work involves using various nondestructive testing techniques. The following tasks are usually included in typical evaluation projects that involve nondestructive testing.

1. Meeting with client to discuss project requirements.

2. Reviewing available project documents.
3. Performing a preliminary site visit to perform initial inspection and select representative structural members for testing.
4. Performing detailed condition surveys that may include cracking survey and/or surface mapping.
5. Performing nondestructive and/or destructive tests.
6. Conducting laboratory tests on samples of structural material.
7. Analyzing field and laboratory test data.
8. Preparing conceptual repair methods.
9. Documenting findings and conclusions in a report.

A summary of the approaches utilized and results obtained from several in-service inspections and nondestructive testing applications performed by CTL is presented in the following sections.

5.4.1 Copper-Copper Sulfate Half Cell

The copper-copper sulphate half-cell technique was used in the evaluation of reinforced concrete foundations at the Umm Said Refinery located in Umm Said, Qatar (Ref. 5.3). Concern for the plant structures was due to the presence of cracks, spalls, and delamination of the concrete and corrosion of embedded steel reinforcement. The testing work was performed within the following scope.

1. Detailed visual observations of test structures were performed, including sounding for delaminations and sketching areas of deterioration. Significant features were documented with photographs.
2. Concrete core and powder samples were removed from test structures for visual observation of internal concrete condition and laboratory testing.
3. Half-cell potential measurements were performed to evaluate probability of corrosion activity of embedded steel reinforcement.
4. Cover measurements were made to determine concrete cover to embedded steel reinforcement.
5. Carbonation tests were made to assess the depth of carbonation in the concrete.
6. Water, aggregate, and soil samples were obtained for analysis.
7. Laboratory tests of samples were performed to evaluate the quality and strength of materials and to determine the mechanism of deterioration.

Half-cell electrical potential testing in accordance with ASTM C 876 (Ref. 5.4) was performed on a grid pattern marked on surfaces of selected concrete foundations. Results indicated a greater than 90% probability that corrosion of reinforcement was active at some test areas and inactive at other test areas. Measurements indicating uncertain corrosion activity were also obtained. Based on other tests such as visual inspection and sounding of concrete, it was concluded that deterioration in the form of cracking, spalling, and delamination had generally resulted from corrosion of embedded steel reinforcement.

5.4.2 Impulse Radar

Impulse radar testing was utilized in an attempt to locate contamination (mudballs) and evaluate overall quality of a 25.4-cm-thick unreinforced concrete slab (Ref. 5.5). The testing work was performed within the following scope.

1. General visual surveys of the slab and nondestructive impulse radar surveys were performed on selected areas to identify presence and distribution of mudballs in the slabs.

2. Concrete core samples and samples of mudballs present at the exposed concrete surface were removed for visual and laboratory testing.
3. Laboratory examination and testing were performed.

Results of the impulse radar testing indicated the presence of randomly distributed mudballs in the concrete slab. It is important to note that small size mudballs (less than approximately 50 mm diam) or mudballs which were dispersed within the concrete could not be detected using the impulse radar.

The impulse radar has also been used in locating steel reinforcement in structural concrete members. In an evaluation of a 76.2-cm-thick reinforced concrete wall at lower Wacker Drive in Chicago (Ref. 5.6), impulse radar was used to determine the presence and location of embedded steel reinforcement. The results were in the form of maps showing locations of reinforcement on interior and exterior surfaces of the wall. Test results were confirmed by visual inspection of test openings at selected locations.

Impulse radar testing was also used to evaluate the presence of a second layer of steel reinforcement and to estimate thickness of structural elements at Cintichem Nuclear Station (Ref. 5.7). Results of radar surveys revealed the presence of two layers of vertical and horizontal reinforcement in approximately 76.2-cm-thick concrete walls.

5.4.3 Magnetic Methods

Commonly known as the R-meter, it has been used as a nondestructive method to locate steel reinforcement in concrete members and estimate concrete cover. The R-meter was used with impulse radar to evaluate the presence of reinforcement in structural concrete members at Cintichem Nuclear Station (Ref. 5.7). It should be noted, however, that the impulse radar results were more reliable than that of an R-meter when two layers of reinforcement were present in a concrete member. No quantitative information regarding the reliability is available. However, the use of an R-meter in locating reinforcement in concrete members is more convenient and easier than the use of impulse radar. Difficulties were encountered in an attempt to use an impulse radar to locate vertical reinforcement in cores of concrete masonry walls (Ref. 5.8) due to the presence of a rough stucco layer on the exterior surface of the walls. The use of a digital R-meter was much more successful. The digital R-meter also was calibrated and used to evaluate the depth of bent steel reinforcement in floor slabs (Ref. 5.9). A typical floor slab consisted of precast, prestressed planks topped with a 5.1-cm-thick concrete. It was necessary to determine whether the bent steel reinforcement was located in the cores of the planks or embedded in the topping layer. The use of an R-meter was preferred over the use of an impulse radar since the purpose was locating steel reinforcement only. Experience with the use of an R-meter indicates that the capability of an R-meter to detect zones of spliced bars is limited, especially if the spacing between the two adjacent spliced bars is less than 50 mm.

5.4.4 Pulse-Echo

This technique has been utilized in evaluating the presence of discontinuities within structural concrete members. Column and shear walls of the Kenwood Towers in Ohio were evaluated for the presence of voids (Ref. 5.10). Largest columns investigated measured approximately 71 x 71 cm in size. Shear walls investigated were approximately 30 cm in thickness. The scope of work involved the following:

1. nondestructive testing using the pulse-echo method on a predetermined grid pattern,
2. interpretation and analysis of test results and recommendations, and
3. completion of repair followed by nondestructive testing (pulse-echo).

Results of the nondestructive investigation revealed the presence of three types of internal deficiencies: (l) voids and nonbonded areas, (2) low density concrete, and (3) disruptions at steel reinforcement. Repair of voids was conducted using epoxy injection. Retesting of the columns and shear walls following completion of repair did not detect any voids. It should be noted that the preliminary testing of the columns and shear walls underestimated the extent of voids within the concrete. This was attributed to the coarse testing grid used. It was concluded that the use of a finer testing grid would improve the detection of voids and nonbonded areas in columns and shear walls. This was proven by the detection of a greater number of defective areas in the later phases of the test program.

Pulse-echo was also used to perform a quantitative assessment of the quality of concrete in a 25-cm-thick seawall in Marina del Ray, Calif., (Ref. 5.11). Defects in the form of discontinuities and corrosion of reinforcement were evaluated and classified according to their severity. The approach used in the investigation was based on gathering nondestructive testing data and correlating this data to data obtained from petrographic examinations and compressive strength testing of cores selected at nondestructive test locations. The nondestructive testing data was in the form of pulse velocity. Quantitative evaluation of the condition of concrete was based on the pulse velocity values. Numbers ranging from 1 to 5 were assigned to different locations in seawalls based on pulse velocity values, where 1 was the worst and 5 the best condition. Concrete rated 2 through 5 was considered of acceptable quality according to the calibrated rating system used (pulse velocity > 3,930 m/sec). Experience with this calibration technique, however, indicated that pulse-echo alone should not be used to evaluate the concrete condition. Additional information obtained from visual and petrographic examinations should also be included in the assessment.

5.4.5 Impact-Echo

The sonic impact-echo method was used in a nondestructive investigation to evaluate the integrity of shotcrete walls of a swimming pool at the Quayside Tower IV in North Miami, Fla., (Ref. 5.12). The walls were approximately 25-cm-thick. Concern with the walls was due to evidence of defects in the form of sand lenses, sand pockets, and voids in the walls. In addition to the nondestructive testing, the scope of work included visual inspection of cores, laboratory testing of cores to determine compressive strength and chloride-ion content, analysis of all test data, and report preparation. Results of the impact-echo testing were given in terms of peak frequencies, amplitudes, compression wave velocities, and reflector depths. A general guideline was developed for rating the shotcrete condition. Two parameters were used to establish this rating guideline, frequency echo peak and wave velocity. The following summarizes the guideline used for shotcrete rating.

Condition Rating A - Sound Shotcrete. Single frequency echo peak from the backside of the wall with wave velocity greater than or equal to 3,270 m/sec.
Condition Rating B - Minor Anomalies. Multiple frequency echo peaks including the echo peak from the backside of the wall with wave velocity greater than or equal to 3,270 m/sec.
Condition Rating C - Shotcrete with Severe Internal Anomalies. No echo from backside of wall was identified. Reflector depths were based on an assumed average velocity of 3,606 m/sec.
Condition Rating D - Questionable to Poor Shotcrete. Wave velocity less than 3,270 m/sec with or without shallow echo peak(s).
Condition Rating E - Possible Thickened Wall. No apparent echo peaks in the test record. The absence of reflections indicated no significant flaws were present.

The typical grid pattern used in the impact-echo testing was approximately 121 x 76 cm with 71 test locations per grid. In the computer calculations of the compression wave velocity, the lowest frequency echo peak was assumed to be from the backside of the wall at each impact-echo test location. This calculated velocity was used to predict reflector depths for any higher frequency

echo peaks. It should be noted, however, that information regarding thicknesses of the wall were preassumed. If the calculated wave velocity was much faster than 4,242 m/sec (the upper limit of the velocity for sound/minor anomaly shotcrete), the lowest frequency echo was interpreted not to be from the backside of the wall but from an internal anomaly. For this case, an average wave velocity of 3,606 m/sec was used to calculate the depth of the crack(s). On the other hand, if an unusually low velocity was calculated, it could be related to the poor quality of concrete or the wall thickness was greater than the assumed value.

Four cores were removed for visual examination. Each core was taken at or near locations of pulse-echo testing. The coring investigations revealed the presence of voids, isolated sand pockets, and cracks. Results of three cores correlated with results of nondestructive testing. However, results of the fourth core which was taken at a location where multiple reflector echoes were obtained did not reveal the presence of major anomalies.

The sonic impact-echo technique was also used in testing mass concrete of the pumpwell for Drydock 3 located at the Norfolk Shipyard, Portsmouth, Virg. (Ref. 5.13). The objective of testing was to evaluate quality and integrity of concrete at 90 test locations. A summary of the impact-echo test results follows.

1. Backside echoes indicative of sound concrete conditions were identified at two test locations. Wall thickness at these locations were approximately 38 and 114 cm.
2. No echoes indicative of backside reflections or defects were identified at 65 test locations. This result was attributed to the massive thickness of concrete (over 300 cm) which damped out the stress wave energy echoes. Based on experience with this testing method, it was concluded that no significant defects existed to distances on the order of 150 to 240 cm from the inside face of the pumpwell walls.
3. Echoes indicative of shallower reflectors and potentially damaged concrete conditions were identified at 23 test locations. Detected damage was believed to be in the form of multiple cracks or voids that did not permit wave energy to be transmitted to the backside of the wall and reflected back. Only 3 of the 23 points had backside echoes.

5.4.6 Ultrasonic Pulse Velocity

The ultrasonic pulse velocity method was also used at Drydock 3 located at the Norfolk Shipyard, Portsmouth, Virg., (Ref. 5.13) to evaluate the quality of concrete. The test involved passing an ultrasonic wave, or pulse, through concrete cores of known diameters that had been extracted from the pumpwell walls. Pulse velocities ranged from 4,182 to 4,606 m/sec with an average of 4,303 m/sec.

Impact-echo tests were also conducted at the drydock. Based on experience with nondestructive testing, pulse velocities obtained from ultrasonic pulse velocity tests are typically 10% faster than those obtained from the impact-echo test. Therefore, a compression wave velocity of 3,787 m/sec was selected as the nominal wave velocity used in the impact-echo depth calculations in the absence of backside reflections. The 3,787 m/sec velocity is 88% of the average ultrasonic pulse velocity obtained from the cores above and is considered conservative. This nominal velocity selection was also supported by test results which showed a backside thickness reflection and thus allowed direct velocity calculation.

The ultrasonic pulse velocity tests of the concrete indicated overall good quality concrete in areas where no shallow echoes were detected by the impact-echo tests, with no significant differences in test results throughout the depths of cores. The ultrasonic pulse velocity test results were further validated by the generally good velocities measured in impact-echo tests where a backside reflection was observed.

Another application of the ultrasonic pulse velocity is to evaluate concrete strength. The compressive strength of selected concrete columns and walls in Building No. 765 at Argonne National Laboratories (West) in Idaho Falls, Idaho, was investigated using the ultrasonic pulse velocity method (Ref. 5.14). The work involved gathering pulse velocity data from selected

concrete columns and walls, removing cores and performing laboratory compressive strength tests, analysis of all data, and preparation of a report. To estimate the strength, a pulse velocity-compressive strength empirical relationship was established by correlating core pulse velocity to compressive strength. Using least-square regression analysis, an equation relating strength to pulse velocity was obtained. Knowing the pulse velocity reading obtained from testing the structural members, it was possible to estimate the strength of concrete through the use of this equation. It should be noted, however, that the accuracy in estimating strength from a pulse velocity reading depends on the correlation curve which incorporates a number of factors such as type of aggregate, aggregate to paste ratio, moisture condition, proximity to steel reinforcement, etc., for the member investigated. Therefore, it is essential to establish a correlation curve (calibration curve) that would incorporate these factors in estimating concrete strength for each concrete mix design.

5.4.7 Vibration/Modal Analyses

Experimental observations related to structural vibrations have been made with two major objectives (Ref. 5.15): (1) determining the nature and extent of vibration response levels and (2) verifying theoretical models and predictions. These two objectives correspond to two types of tests (Ref. 5.15): (1) vibration forces, or more usually, responses (accelerations) are measured during operation of the structure under study and (2) the structure is vibrated with a known excitation and the response is recorded. The second type of test including both the data acquisition and its subsequent analysis is usually referred to as "Modal Testing."

CTL performed an evaluation of the Century Shopping Center parking garage in Chicago, Ill., (Ref. 5.16). The evaluation included vibration monitoring at selected locations of the parking garage. Concern with the structure was due to potential excessive vibration of the garage structure caused by aerobic exercise activities within the health club occupying the top floor above the parking floors. The purpose of the monitoring program was to obtain qualitative information regarding structural vibration response to aerobic exercise activity. A comparison was made between vibrations at points in the health club directly on the floor where exercise was taking place to points in the parking garage several levels below. To obtain a true vertical profile of response through the structure it would have been necessary to take simultaneous readings at a number of locations so that changes in exercise intensity could be compensated for in comparing one point to another. However, based on the client's request, vibration data were measured one location at a time during aerobic classes. Vibration level (in units of acceleration) was measured utilizing accelerometers that produce electrical signals proportional to acceleration (in units of gravity). The accelerometers were attached directly to locations to be monitored with a wax-based bonding agent and, therefore, sensed local acceleration at the attachment point. The accelerometer signals were monitored and recorded by a portable single channel vibration monitor. Floor responses were obtained on graphs as a function of acceleration and time. The average peak acceleration responses of the floors were compared to data obtained for human response perception to continuous vibration for active occupancies (Ref. 5.17). It was concluded that measured vibration levels were in the perceptible range as defined by recognized human vibration response criteria. However, structural calculations, based on measured vibration levels, indicated that stress levels in structural members were within acceptable ranges.

CTL also performed an evaluation of the Kishwaukee River Bridge using the technique of modal analysis (Ref. 5.18). The Kishwaukee River Bridge is located in Winnebago County, Ill. It consists of two identical parallel bridges. Each bridge has three main spans of 75.6 m and two end spans of 51.4 m. A typical cross-section consists of a continuous single-cell box girder constructed with precast concrete segments post-tensioned together. The project was carried out to obtain information relating the behavior of the bridge to its dynamic characteristics. The reasons for selecting dynamic testing are as follows.

1. Dynamic testing has demonstrated its capability to discover defects or deterioration through comparison of measured dynamic behavior with expected behavior (Ref. 5.19). Anomalies in measured characteristic shapes can quite often pinpoint specific areas with potential defects. Further detailed testing and inspection are then focused on these specific areas of concern.
2. Another purpose of dynamic testing is to verify assumptions made in analytical models. Certain assumed properties, such as material stress-strain characteristics, support conditions at piers, and effect of cracking are not usually known to a high degree of accuracy. The actual properties of a bridge and its support conditions are reflected in the measured dynamic characteristics and can be compared to an analytical model to check the accuracy of the model.
3. The set of dynamic characteristics established for the bridge can serve as a baseline for rapid future assessment of the bridge's overall condition. In the future, a set of operational frequencies and characteristic shapes can be measured at regular intervals and the measurements compared to the baseline or fingerprint set. Structure defects or deterioration over time would be indicated by change in dynamic response measured in successive modal tests.

An analytical model was developed to estimate the dynamic characteristics of the bridge. In addition, actual dynamic characteristics were determined by modal analysis from field measurements. Operational frequencies and associated characteristic shapes were determined for the bridge subjected to normal traffic loading. One stationary (driver) and three movable (rover) accelerometers were used to measure the bridge response. The accelerometers were connected by cables to an analyzer located in an instrument van. The first setup located all four accelerometers at the same test point for calibration. For each subsequent test setup the driver remained at the first test point while the three rovers were moved to different points in a predetermined sequence. At each test setup, the relative response between each of the rovers and the driver was used to determine transfer functions required for modal analysis. The field data were then analyzed and the dynamic characteristics of the bridge were developed.

Results of the investigation of the Kishwaukee Bridge indicated the following.

1. The overall dynamic behavior was classified as normal for a bridge of its age and configuration.
2. The characteristic shapes of measures modes of vibration through a repaired area were different from similar unrepaired areas on the bridge. This clearly indicated nonlinear behavior and lack of smooth, continuous load transfer through the repaired area.
3. A good baseline or fingerprint set of dynamic characteristics was obtained for the bridge. It was recommended, however, that dynamic characteristics be determined periodically and compared to the baseline set. Minor or no change would indicate minor or no change in the bridge's condition. A significant change, substantial local change, or a continuous rate of change would highlight the need for closer inspection.

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6. RECOMMENDED TESTING METHODS TO DETECT CONCRETE DEGRADATION

6.1 INTRODUCTION

During evaluation of concrete structures, a visual inspection is important because it may reveal a substantial amount of information with regard to cause of deterioration, required method of testing, and potential remedies needed. This section of the report provides a list of the testing methods recommended for assessment of the primary degradation factors which potentially can impact the performance of nuclear power plant concrete structures. Modal analysis testing can be used to identify areas of distress. It can also be used to indicate changes in the condition of the structure. However, use of this technique requires a baseline for comparison. A baseline can be established by developing a theoretical model or measuring dynamic characteristics of a similar structure that is in a good condition. For the case of monitoring changes in a structure's condition, the baseline can be established from the first set of measurements of dynamic characteristics. A summary and ranking of the testing techniques relative to their ability to detect and assess the effects of the primary degradation factors noted in Section 3 is presented in Table 6.1. It should be noted, however, that only a testing guideline is provided, and it is up to the evaluation engineer or inspector to decide on the suitable method of testing.

6.2 CONCRETE MATERIAL DEGRADATION

As indicated in Section 3 of this report, the degradation factors pertinent to aging of the concrete in nuclear power plant structures are classified into two categories: (1) chemical attack and (2) physical attack.

6.2.1 Chemical Attack

Degradation factors included under chemical attack are: (1) alkali-aggregate reactions; (2) sulfate attack; (3) efflorescence/leaching; and (4) bases, acids, and salt crystallization.

6.2.1.1 Alkali-Aggregate Reactions

The two types of alkali-aggregate reactions, alkali-silica and alkali-carbonate, form a gel which can imbibe water to expand and crack the concrete. In both cases, a visual inspection and a crack map can provide some information on the extent of the deterioration. Cores also should be taken from the structure in order to allow a petrographic examination to be performed. This may identify the source of degradation, that is, whether it is alkali-silica reaction, alkali-carbonate reaction, or due to some other mechanism. References 6.1 to 6.4 present guidelines for performing petrographic examinations of concrete and concrete-making materials. The Osmotic Test (Ref. 6.5) also has been used as a means of rapid screening for determination of alkali-aggregate reactivity. Finally, a relatively new technique, the ultraviolet radiation method, has been developed which can identify the presence of alkali-silica reactivity. This technique is discussed in Section 7.4.

6.2.1.2 Sulfate Attack

Deterioration of concrete due to sulfate attack can be in the form of pattern cracking as well as general disintegration of concrete. The formation of calcium sulfate and sulfoaluminate (ettringite) results in an increase of concrete volume. The increase results in pattern cracking. A visual inspection can reveal surface cracking. A core would have to be taken for laboratory testing. Petrographic examination (Ref 6.1) may reveal the cause of the expansion occurring and evaluate

extent of crackling. Corrosion of reinforcement can be expected when concrete of low sulfate resistance is exposed to sea water. The standard chemical test ASTM C 114 (Ref 6.6) can be used to evaluate the presence of sulfate in concrete, soil, and/or waters (rain, groundwater, seawater).

6.2.1.3 Efflorescence and Leaching

Visual inspection of exposed concrete surfaces can indicate the occurrence of efflorescence. Laboratory testing performed on a sample of the material can be used to confirm the presence of calcium carbonate. The testing technique used is x-ray diffraction (Ref 6.7). Petrographic examination (Ref 6.1) can also assist in the detection of efflorescence and assess the extent of reaction that has occurred.

6.2.1.4 Bases/Acids/Salt Crystallization

Damage to concrete in the form of loss of cement paste or disintegration may occur as a result of chemical attack or salt crystallization. Visual examination of a core extracted from the area of concern can provide some information regarding the extent of damage. Petrographic examination (Ref. 6.1) of the core sample can reveal more detailed information in evaluating the extent of deterioration. In order to detect the specific type of attack, however, detailed chemical analyses need to be performed. Selection of a specific chemical test depends on the nature of the environment surrounding the concrete member. In general, the standard test ASTM D 4763 (Ref. 6.8) is included in this type of investigation.

6.2.2 Physical Attack

Degradation factors defined in Section 3 under physical attack include: (1) moisture changes, (2) freeze/thaw cycling, (3) thermal exposure/thermal cycling, (4) irradiation, (5) abrasion/erosion/cavitation, (6) fatigue/vibration, and (7) creep.

6.2.2.1 Moisture Changes

Cracking of concrete is the major defect that may result from moisture changes associated with restrained volume changes. Cracking pattern and type can provide helpful information on the problem under investigation. The three major types of cracks are shrinkage cracks, shear cracks, and flexural cracks (Ref. 6.9). A visual inspection can determine the approximate cause of the deterioration. Cracks occurring in a direction parallel to the side where the structural member is restrained can be related to shrinkage of the concrete. Additional information, such as the nature of environmental exposure and water-cement ratio of the concrete, can assist in the detection of this form of degradation. Petrographic examination (Ref. 6.1) can be used to estimate the water-cement ratio. It can also assist in evaluating the presence of microcracking and extent of damage in concrete.

Infrared thermography, ultrasonic pulse velocity, impact-echo, and pulse-echo can be utilized to detect cracks in concrete. Concrete core samples tested in accordance with ASTM C 157 (Ref. 6.10) can reveal the resistance of concrete to drying shrinkage, i.e., length changes.

6.2.2.2 Freeze/Thaw Cycling

Damage to concrete due to freeze/thaw cycling can be in the form of scaling, spalling and cracking. Visual testing of cores extracted from deteriorated areas can provide an indication of the extent of the freeze/thaw damage. Information regarding the air-entrainment system obtained from petrographic examination (Ref. 6.1) can assist in the detection of this degradation factor. Also, information regarding the exposure condition and the type of deterioration of concrete is usually considered in the evaluation of cause of damage.

In most cases, a visual inspection of the concrete structure can reveal the freeze/thaw deterioration. The nondestructive testing methods that can be used to assess extent of damage are impact-echo and pulse-echo. For evaluation of large horizontal structural concrete members, infrared thermography can be used.

Zaman (Ref. 6.11) has suggested prediction techniques such as surface weight loss, depth of scaling, percentage area scaled, visual description, compressive, and flexural strength measurements. The common standard ASTM tests used to investigate the resistance of concrete to freezing and thawing are ASTM C 457, C 666, C 672 (Refs. 6.12–6.14).

6.2.2.3 Thermal Exposure/Thermal Cycling

Deterioration of concrete due to thermal exposure and/or thermal cycling can be in the form of reduction of mechanical properties including compressive and tensile strength and modulus of elasticity. Severe thermal exposure can cause cracking or spalling of concrete at the exposed surface. Visual examination of the concrete surface as well as petrographic examination (Ref. 6.1) of cores extracted from damaged areas can reveal or indicate the type(s) of deterioration that have occurred. Additional information regarding the exposure condition assists in identifying this degradation factor.

In a more comprehensive assessment of the extent of damage pertinent to thermal effects, nondestructive testing may be utilized. The pulse velocity, impact-echo, and pulse-echo testing techniques can be used to assess the in-place strength. This may require testing cores in compression to obtain calibration curves that relate wave velocity in concrete to compressive strength. Alternately, the modal analysis testing method can be used to assess the structural integrity of the concrete member. Evaluation of mechanical properties of concrete may also require performing appropriate laboratory tests on representative samples removed from the structure.

6.2.2.4 Irradiation

The effect of irradiation on concrete is similar to the thermal effects in that a reduction in mechanical properties may occur. In addition, deterioration in the form of spalling and cracking may result from excessive exposure to irradiation. Therefore, testing methodologies utilizing nondestructive and destructive techniques would be the same as discussed in Section 6.2.2.3 of this report..

6.2.2.5 Abrasion/Erosion/Cavitation

Damage to concrete due to these degradation factors is in the form of loss of material at exposed concrete surface. Visual inspection of the structural member may provide an indication of the surface condition. Surfaces subjected to abrasion may appear worn and polished. Erosion and cavitation may produce severely pitted and extremely rough surfaces. Petrographic examination (Ref 6.1) performed on concrete cores extracted from damaged areas can reveal information regarding the extent of damage. The standard ASTM tests that can be used to evaluate concrete resistance to abrasion are C 418, C 779, and C 944 (Refs. 6.15-6.17). The standard test ASTM C 32 (Ref. 6.18) can be used to evaluate concrete resistance to erosion and cavitation.

6.2.2.6 Fatigue/Vibration

The symptoms identified for fatigue distress are cracking, microcracking, and/or excessive deflection. Cracking, if severe, can be noted by simple visual inspection. Microcracking can be evaluated by testing concrete for homogeneity using the ultrasonic pulse velocity method in addition to performing petrographic examinations (Ref. 6.1) on representative concrete samples. The impact-echo and pulse-echo methods can also be used in evaluating the cracks and microcracks within the concrete member. Modal analysis is an alternative method of testing to determine the change in structural integrity over time.

6.2.2.7 Creep

As indicated in Section 3.2.2.7 of this report, degradation of concrete due to creep can be in the form of cracking, if the mass concrete is restrained.

Creep of concrete can be detected and assessed through the use of strain gages installed on the member which are monitored periodically. Visual and petrographic examination (Ref. 6.1) of concrete can reveal the extent of cracking, if such cracking occurs. The modal analysis method of testing can be used to indicate any change of physical characteristics at different ages of the structure. Comparison of data at different times can assist in the evaluation of the creep phenomenon.

6.3 MILD STEEL DEGRADATION

According to Section 3.3 of this report, there are four potential degradation factors that can affect mild steel reinforcement used in nuclear power plant concrete structures: (1) corrosion, (2) elevated temperature, (3) irradiation, and (4) fatigue. Of these, corrosion is the most likely factor affecting degradation of mild steel reinforcement in nuclear power plant concrete structures.

Corrosion of mild steel reinforcement may be indicated by coring, chemical method, air permeability method, electrical method, nuclear, and visual inspection. Deterioration of concrete as a result of corrosion may be detected by visual inspection, infrared thermography, audio methods, ultrasonic transmission, and radiography. The chemical and electrical methods can be used to indirectly measure the potential for corrosion.

The location of the reinforcement can be determined by using one of the magnetic methods such as the R-meter. After locating the reinforcement, a core sample through the reinforcement can be taken and visually examined to determine if corrosion is present. Chemical tests can determine the amount of chloride ions and moisture content present in concrete. This can provide an indication of the potential for corrosion. The air permeability method can be used in the assessment of the resistance of concrete to carbonation and the penetration of aggressive ions.

The electrical methods, four electrode and copper-copper sulfate half cell, are used in the assessment of corrosion of the steel reinforcement embedded in concrete. The standard test ASTM C 876 (Ref. 6.19) is utilized to evaluate the potential for steel reinforcement corrosion . It does not provide an indication of the actual amount of steel corrosion present, only the potential for corrosion. A visual inspection can reveal rust stains on the surface of the concrete. Rust stains usually provide a positive sign that corrosion is occurring. The nuclear method provides an indication of the amount of moisture present in the concrete. The infrared thermography, audio method, ultrasonic transmission, and radiography can be used to detect deterioration of concrete caused by corrosion of reinforcement. These test methods locate delaminations, spalling, cracking, etc.

6.4 PRESTRESSING STEEL DEGRADATION

As identified in Section 3.4 of this report, the degradation factors that may influence the performance of prestressing steel systems utilized in nuclear power plant concrete structures include: (1) corrosion, (2) elevated temperature, (3) irradiation, and (4) loss of prestressing.

The primary symptoms of degradation due to these factors are the loss of steel cross-section due to corrosion and/or loss of prestressing forces due to other factors. The presence of corrosion can be evaluated in ungrouted prestressing steel systems by visual inspection (Refs. 6.20 and 6.21). This usually includes examination of tendon anchorage areas (areas most susceptible to corrosion). The guidelines contained in Regulatory Guide 1.35, Rev. 3 (Ref. 6.21) can be used in monitoring prestressing tendon-stress levels. This is accomplished by performing the lift-off test or other equivalent tests. These tests include measurements of the tendon-force level with properly calibrated jacks and the simultaneous measurements of elongations (Ref. 6.22).

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Table 6.1 Recommended testing methods to assess concrete degradation.

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

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

Table 6.1 (cont.)

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

* Rating is based on simplicity of the method and practical experience in applying the method.

7. POTENTIAL NEW TECHNIQUES TO ASSESS CONCRETE DEGRADATION

7.1 INTRODUCTION

Several new nondestructive testing techniques, although still in the development stage, possess potential for application to the evaluation of reinforced concrete structures in nuclear power plants. These techniques are briefly discussed below.

7.2 MAGNETIC METHODS

7.2.1 Leakage Flux

When ferromagnetic materials are magnetized, magnetic lines of force (flux) flow through the material and complete a magnetic path between the poles. When the magnetic lines of flux are contained within a ferromagnetic object, it is difficult to detect them in the air space surrounding the member. If the surface of the member contains a crack or defect, however, its magnetic permeability is changed and leakage flux will emanate from the discontinuity (Ref. 7.1). Measurement of the intensity of the leakage flux provides a basis for nondestructive identification of such discontinuities.

The United States Federal Highway Administration (Ref. 7.2) is developing a magnetic field method based on leakage flux for use in detecting loss of area (>10%) due to corrosion and fracture of reinforcing bars and prestressing strands. A steady-state magnetic field is applied to the member under inspection, and a scanning magnetic field sensor (Hall-Effect device) is used to detect protuberances in the applied field caused by anomalies such as deterioration or cracks. The technique has been demonstrated in the laboratory relative to its ability to detect varying degrees of deterioration, influence of adjacent unflawed steel elements, type of tendon duct, type of reinforcing steel, transverse rebar configuration, etc. (Ref. 7.1). The laboratory results indicated good overall sensitivity to loss-of-section and excellent sensitivity to fracture. Field tests have produced similar results. Additional work at the Ferguson Laboratory of the University of Texas (Austin) confirmed the ability of the method to detect fractures in reinforcing strands developed due to fatigue loadings (Ref. 7.1). The capability of this technique in testing concrete components with congested reinforcement has not been reported.

7.2.2 Nuclear Magnetic Resonance

Nuclear Magnetic Resonance (NMR) is an electromagnetic method capable of determining the amount of moisture present in a material by detection of a signal from the hydrogen nuclei in water molecules. The method exposes the material examined to a static magnetic field and to a pulsed radio frequency (RF) magnetic field corresponding to the NMR frequency of the nuclei of interest (Ref. 7.3). A prototype NMR moisture measurement system has been developed by Matzkanin et al. (Ref. 7.3), which utilizes a two-pulse sequence to provide the capability of distinguishing between NMR signals from free moisture and signals from bound hydrogen. The system utilizes a sensor assembly in which the test article is external to both the RF coil and magnet structure. The system specifications indicate a moisture content range from 1 to 6% by weight of material with an error of ±0.2% moisture for depths up to 70 mm and ±0.4% for depths from 70 to 95 mm. No information regarding the accuracy of this method in massive concrete structures is available.

7.3 ELECTRICAL METHODS

7.3.1 Capacitance Instruments

Experimental investigations at frequencies up to 100 MHz have shown that components (real and imaginary) of the dielectric constants of building materials increase significantly with increasing moisture content (Refs. 7.4-7.6). The dielectric constant provides a measure of the ability of a material to store a charge. Several capacitance instruments are available, having various electrode configurations, to measure moisture content of building materials. The electrodes are attached to a constant frequency alternating current source and establish an electric field in the material to be tested. The current flow power loss is used to indicate moisture content. Reference 7.5 indicates that moisture content of laboratory concrete specimens could be determined to ±0.25% for values less than 6% using a 10 MHz frequency. Knab et al. (Ref. 7.7) suggests that further study is required to establish the reliability of the method. Also, for test results, the instruments should be calibrated to the material.

7.3.2 Polarization Resistance

The National Institute of Standards and Technology (NIST) has developed a system for measuring corrosion rate of steel in concrete (Refs. 7.8 and 7.9). The system is computer controlled and uses the polarization resistance to compute the corrosion current which indicates the rate at which steel is corroding. The polarization technique employs a three electrode system using the specimen as one electrode, a voltage reference as the second electrode, and a counter electrode from which polarizing current is applied to the specimen. The reliability and accuracy of the system are being assessed.

Nippon Steel (Ref. 7.10) has developed an electrochemical impedance system, somewhat similar to the NIST system, which can be used for locating deteriorated areas and for corrosion rate measurements of rebar embedded in concrete. Corroded and noncorroded rebars are distinguished based on different impedances in a low-frequency region. Corrosion reaction of rebars is indicated by differences in impedances between the low-frequency region and the high-frequency region. The method was investigated using a 20-year-old precast concrete slab reinforced by welded wire fabric. Results showed that the electrical potential best serves as an indicator of a threshold value for corrosion to occur, and that the reciprocal value of polarization resistance of the corrosion reaction correlates well with actual corrosion rate, e.g., at high corrosion rates the potential is low and reciprocal value of polarization resistance is high. The Nippon Steel system is undergoing evaluation and is not presently available.

7.3.3 Half-Cell Potential Test Using Impulse Radar

As described in Section 4 of this report, the copper-copper half cell has the capability of determining the potential for corrosion of reinforcement. When performing half-cell potential tests, one end of the cell is connected to the reinforcement and the other end is placed in direct contact with the test area. In this configuration a "closed" circuit is formed so that a reading can be taken. The most recent addition to this technique is the capability to measure the resistance to passing a charge through the concrete.

Because of the "electric potential" of the concrete, it is of interest to correlate the return data from the impulse radar to that of the copper-copper sulphate half cell. The theory behind this concept is that as the resistance of the concrete increases, the dielectric constant of the concrete decreases. The opposite occurs when the resistance of concrete decreases. The change in dielectric constants can be detected by the impulse radar. This method is still in the experimental stage, and at present, no literature is available to assess its application to massive concrete structures.

7.4 ULTRAVIOLET RADIATION METHOD

The ultraviolet radiation method is a new method that was recently developed to identify alkali-silica reaction (ASR) products in in-situ concrete structures (Ref. 7.11). This method identifies ASR by staining the concrete with uranyl ions which have the characteristic of greenish yellow fluorescence under short-wave ultraviolet light.

The fluorescence test was developed at Cornell University and most of the concrete specimens used in the research study were obtained from CTL. The results of this test are available for a wide variety of aggregates and alkalis. The test can be used successfully to determine whether the products of ASR are present in a structure and thus provide clues to the potential for further deterioration of the structure. No quantitative information can be obtained by this test. Another limitation about this test is that it requires a controlled lighting system, i.e., short-wave ultraviolet light in a darkened room.

7.5 FINITE-ELEMENT ANALYSIS METHOD

7.5.1 Performance Monitoring Using Finite-Element Analysis

This potential technique was presented by CTL in 1989 (Ref 7.12). It proposes to utilize field data obtained through permanently installed instrument sensors in conjunction with Finite-Element Analysis (FEA) to assess the structural behavior of nuclear containment structures.

This new monitoring technique, through the use of sensors and associated digital technology, can indicate changes in the physical condition of a structure due to aging or influence of degradation factors. The method can also provide a means to measure any deleterious effects on the structure from an accident or seismic event. Furthermore, it provides a means to evaluate the post-accident condition of the structure by comparing strains, deflections, and accelerations before and after an extreme event. The technique requires establishing a baseline condition. For a new structure, a baseline condition can be obtained from initial field measurements. However, for an aged structure, a baseline condition can be obtained by modeling the structure using finite-element methods. Results of the finite-element analysis will represent the baseline condition.

7.5.2 Field Measurements

The type of field measurement and choice of sensor may vary. It depends on the objectives of the monitoring. Field measurements may include applied loads or reactions, deformations, motions, or environmental conditions. Applied loads or reactions can be measured by means of load cells, calibrated strain gages, or pressure cells. Deformations can be monitored in terms of strain monitoring, deflection measurements, or rotation measurements. Movement characteristics of the structures such as velocity and acceleration can be measured by seismometers or accelerometers.

For proper monitoring, it is recommended in Ref. 7.12 that sensors be very durable, should have long-term signal stability, and be easy to install. Sensors should not be grossly affected by the change of environment. For a more practical and convenient method of collecting field data, a microcomputer attached to data acquisition systems can be used to scan sensors and record readings. A personal computer can control up to 700 sensors with recorded readings stored on magnetic media or transmitted back to the main computer for reduction and analysis. A more practical number of sensors can be used by defining critical parameters at critical locations. This can be accomplished by performing a brief analysis of the containment.

7.5.3 Finite-Element Analysis

As indicated in Ref. 7.12, the objective in developing a concept for structural monitoring of nuclear containments is to use the field measurements as a part of the boundary condition data required in the finite-element analysis. In addition, static and dynamic test data acquired in the field can be integrated with the analytical model for verification and model refinement. A simple example to illustrate the use of field data in the finite-element analysis is as follows:

$$F = [K]D$$

where F represents the force vector, [K] represents the stiffness matrix, and D represents the displacement vector. Using the substructuring technique, as indicated in Ref. 7.12, additional equations that relate element forces to boundary displacements can be obtained. If all boundary displacements are known from field measurements, the element forces can be computed. It should be noted, however, that the linear-elastic behavior originally assumed in the model, can be changed to nonlinear behavior when evidence of deterioration (i.e., cracking) is indicated. Structural integrity can be monitored by periodic measurements of the installed sensors that will provide a quick check on the performance of key structural elements. Information can forewarn pending abnormalities or structural distresses. Also, measurement of structural performance during extreme events can assist in judging the capabilities of the existing structure prior to restart. Additional investigative work needs to be conducted in order to demonstrate the suitability of application of this method to nuclear concrete structures.

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