

ACI 437R-19

Strength Evaluation of Existing Concrete Buildings

Reported by ACI Committee 437



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Strength Evaluation of Existing Concrete Buildings

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Strength Evaluation of Existing Concrete Buildings

Reported by ACI Committee 437

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The strength of existing concrete buildings and structures can be evaluated analytically and supplemented where necessary with load testing. The recommendations in this report indicate when such an evaluation may be needed, establish criteria for selecting the evaluation method, and indicate the data and background information necessary for an evaluation. Methods of determining material properties used in the analytical and load test investigations are described in detail. Analytical investigations should follow the principles of strength design. Working stress analysis can supplement the analytical investigations by relating the actual state of stress in structural components to the observed conditions.

Keywords: cracking; deflection; deformation; deterioration; gravity load; load; load test; reinforced concrete; strength; strength evaluation; test.

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CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

This report defines the process of structural evaluation to determine the structural adequacy of existing concrete structures as defined by [ACI 562](#). The procedures can be applied generally to new concrete structures, provided that appropriate evaluation criteria are agreed upon before the start of the investigation. This report covers structural concrete, including conventionally reinforced cast-in-place concrete, precast-prestressed concrete, precast post-tensioned concrete, and post-tensioned cast-in-place concrete.

1.2—Scope

1.2.1 Background and limitations—Engineering judgment based on rational, scientific principles is critical in the strength evaluation of concrete structures. Such judgment of a qualified licensed design professional is needed for proper application of relevant code provisions to the case being studied. An assessment of structural safety can be achieved with the information and data from field investigations, scientific computations based on sound principles, as well as subjective engineering judgment from the licensed design professional. This is particularly true for structures deteriorated from prolonged exposure to the environment or damaged in an extreme event, such as a fire, earthquake, or explosion.

Similarly, there are no generally recognized criteria for evaluating serviceability of an existing concrete structure. Such evaluation requires engineering judgment based on scientific principles, and close consultation with the owner regarding the intended use of the structure and expected level of performance.

[ACI 562](#) discusses potential conclusions resulting from a strength evaluation; however, the determination of one or more of the following conclusions regarding the integrity of a concrete structure is possible from a strength evaluation:

- a) The structure or structural element has an adequate margin of safety according to the provisions of the applicable building code.
- b) The strength determined by evaluation is less than that required for factored loads but greater than required for service loads (load factors equal to or greater than 1.0 for all load cases). In this case, the structure or structural element is not adequate. In some cases, restricted use of the structure that limits the applied loads in recognition of the computed strength may be permitted.
- c) The design strength of the structure is less than required for service loads under the applicable building code. In such cases, the owner should be notified and consideration given to the installation of shoring, severe restriction of use, or evacuation of the structure until remedial work can be done.

1.2.2 Applications—The procedures recommended in this report apply to strength evaluation of existing concrete buildings or other structures, including the following circumstances:

- a) Structures that show damage from excess or improper loading, explosions, vibrations, fire, or other causes
- b) Structures where there is evidence of deterioration or structural weakness, such as excessive cracking or spalling

of the concrete, reinforcing bar corrosion, excessive member deflection or rotation, or other signs of distress

- c) Structures that are suspected of not satisfying building code requirements in terms of design, materials, or construction
- d) Structures where there is doubt as to the structural adequacy and the original design criteria are not known
- e) Structures undergoing expansion or a change in use or occupancy and where the new design criteria exceed the original design criteria
- f) Structures that require performance testing following remediation (repair or strengthening)
- g) Structures that require testing by order of the building official

1.2.3 Exceptions—This report does not address the following conditions:

- a) Performance testing of structures with unusual design concepts
- b) Product development testing where load tests are carried out for quality control or approval of mass-produced elements
- c) Evaluation of soil conditions
- d) Load assessment for strength evaluation of environmental engineering concrete structures (refer to [ACI 350](#) for additional information)
- e) Liquefied gas containment structures (refer to [ACI 376](#) for additional information)

1.2.4 Categories of structural evaluation—There are numerous different characteristics or levels of performance of an existing concrete structure that can be evaluated. These include:

- a) Stability of the entire structure
- b) Stability of individual components of the structure
- c) Strength and safety of individual structural elements
- d) Stiffness of the entire structure
- e) Stiffness of individual structural elements
- f) Susceptibility of individual structural elements to excessive long-term deformation
- g) Dynamic response of individual structural elements
- h) Fire resistance of the structure
- i) Serviceability of the structure
- j) Durability of the structure

This report deals with the evaluation of an existing concrete structure for stability, strength, and safety. Although not intended to be an in-depth review of durability, this report addresses durability-related aspects and notes significant features that could compromise structural performance, either at the time of the investigation or later.

1.2.5 Procedure for a strength evaluation—Most strength evaluations have many basic steps in common. Each evaluation, however, should address the unique characteristics of the structure in question and the specific concerns that have arisen regarding its structural integrity. Generally, the evaluation will consist of:

- a) Defining the existing condition of the structure, including:
 - i. Reviewing available information
 - ii. Conducting a condition survey
 - iii. Determining the cause and rate of progression of existing distress

- iv. Performing preliminary structural analysis
- v. Determining the degree of repair to precede the evaluation
- b) Selecting the structural elements that require detailed evaluation
- c) Assessing past, present, and future loading conditions to which the structure has and will be exposed under anticipated use
- d) Conducting the evaluation
- e) Evaluating the results
- f) Preparing a comprehensive report including description of procedure and findings of the previous steps

CHAPTER 2—DEFINITIONS

Please refer to the latest version of ACI Concrete Terminology for a comprehensive list of definitions.

CHAPTER 3—PRELIMINARY INVESTIGATION

This chapter describes the initial work that should be performed during a strength evaluation of an existing concrete structure. The objective of the preliminary investigation is to establish the existing condition of the structure to obtain a reliable assessment of the available structural capacity. This requires estimating the condition and strength of the concrete and the condition, location, strength, and area of reinforcement. Sources of information that should be reviewed before carrying out the condition survey are discussed. Available techniques for conducting a condition survey are described. Refer to **ACI 562** for code requirements for the preliminary assessment.

3.1—Review of existing information

To learn as much as possible about the structure, the licensed design professional should research the history of the structure related to the design, construction, and service record. A thorough knowledge of the original design criteria minimizes the number of assumptions necessary to perform an analytical evaluation. The following list of possible information sources is intended as a guide. Not all of them need to be considered in a strength evaluation. The licensed design professional should exercise judgment in determining which sources need to be consulted for the specific strength evaluation being conducted.

3.1.1 The original design—Many sources of information are helpful in defining the parameters used in the original design, such as:

- a) Architectural, structural, mechanical, electrical, and plumbing contract drawings and specifications
- b) Structural design calculations
- c) Change orders to the original contract drawings and specifications
- d) Project communication records such as faxes, transcripts of telephone conversations, e-mails, and memoranda between the engineer of record and other consultants for the project
- e) Records of the local building department
- f) Geotechnical investigation reports including anticipated structure settlements

- g) The structural design standards referenced by the local code at the time of design

3.1.2 Construction materials—Project documents should be checked to understand the types of materials that were specified and used for the structure, including:

- a) Reports on the proportions and properties of the concrete mixtures, including information on the types of admixtures used and whether they contained more than negligible amounts of chlorides
- b) Reinforcing steel mill test reports
- c) Material shop drawings, including placing drawings prepared by suppliers that were used to place their products, bars, welded wire fabric, and prestressing steel; formwork drawings; and mechanical, electrical, and plumbing equipment drawings
- d) Thickness and properties of any stay-in-place formwork, whether composite or noncomposite by design; such materials could include steel sheet metal and clay tile

3.1.3 Construction records—Documentation dating from original construction may be available such as:

- a) Correspondence records of the design team, owner, general contractor, specialty subcontractors, and material suppliers and fabricators
- b) Field inspection reports
- c) Contractor and subcontractor daily records
- d) Job progress photographs, films, and videos
- e) Concrete cylinder compressive strength test reports
- f) Field slump and air-content test reports
- g) Delivery tickets from concrete trucks
- h) As-built drawings
- i) Survey notes and records
- j) Reports filed by local building inspectors
- k) Drawings and specifications kept in the trailers or offices of the contractor and the subcontractors during the construction period
- l) Records of accounting departments that may indicate materials used in construction

3.1.4 Design and construction personnel—Another source of information concerning the design and construction of the structure under investigation is the individuals involved in those processes. Interviews often yield relevant information for a strength evaluation. This information can reveal problems, changes, or anomalies that occurred during design and construction.

3.1.5 Service history of the structure—This includes documents that define the history of the structure such as:

- a) Records of current and former owners/occupants, their legal representatives, and their insurers
- b) Maintenance records
- c) Documents and records concerning previous repair and remodeling, including summaries of condition assessments and reports associated with the changes made
- d) Records maintained by owners of adjacent structures
- e) Weather records
- f) Logs of seismic activity and activity or records of other extreme weather events, such as hurricanes (where applicable)
- g) Photographs of the structure, including aerial photographs

3.2—Condition survey of structure

Areas of deterioration and distress in structural elements should be identified, inspected, and recorded as to type, location, and degree of severity. Procedures for performing condition surveys are described in this section. The reader should also refer to [ACI 201.1R](#) and [ACI 364.1R](#). Engineering judgment should be exercised in performing a condition survey. All the steps outlined in the following may not be required in a particular strength evaluation. The engineer performing the evaluation should decide what information will be needed to determine the existing condition of structural elements of the particular structure being evaluated.

3.2.1 Recognition of abnormalities—A broad knowledge of the fundamental characteristics of structural concrete and the types of distress and defects that can be observed in a concrete structure is essential for a successful strength evaluation. Additional information on the causes and evaluation of concrete structural distress may be found in [ACI 201.1R](#), [ACI 207.3R](#), [ACI 222R](#), [ACI 222.2R](#), [ACI 222.3R](#), [ACI 224R](#), [ACI 224.1R](#), [ACI 224.4R](#), [ACI 228.2R](#), [ACI 309.2R](#), [ACI 349.3R](#), [ACI 362.2R](#), [ACI 364.1R](#), [ACI 423.4R](#), and [ACI 423.8R](#), as well as documents of other organizations such as the International Concrete Repair Institute (ICRI).

3.2.2 Visual examination—Visual distress, deterioration, and damage existing in the structure should be located by means of a thorough visual inspection of the critical and representative structural components. Liberal use of photographs, notes, and sketches to document this examination is recommended. Abnormalities should be recorded as to type, magnitude, location, and severity.

If the engineer conducting the visual examination finds defects that render a portion or all of the structure unsafe, the condition should be reported immediately to the owner or building official. Appropriate temporary measures should be undertaken immediately to secure the structure before it is placed back into service and the survey continued.

To employ the analytical method of strength evaluation discussed in [Chapter 6](#), it is necessary to obtain accurate information on the member properties, dimensions, and positioning of the structural components in the structure. If this information is incomplete or questionable, the missing information should be determined through a field survey. Verification of geometry and member dimensions by field measurement should be made for all critical members.

3.2.3 In-place tests for estimating concrete compressive strength—Numerous standard test methods are available for estimating the in-place concrete compressive strength or for determining relative concrete strengths within the structure. Traditionally, these have been called nondestructive tests to contrast them with drilling and testing core samples. A more descriptive term for these tests is in-place tests. Additional information on these methods can be found in [ACI 228.1R](#), [Malhotra \(1976\)](#), [Malhotra and Carino \(2004\)](#), and [Bungey et al. \(2006\)](#).

The common feature of in-place tests is that they do not directly measure compressive strength of concrete. Rather, they measure some other property that has been found to have an empirical correlation with compressive strength.

These methods are used to estimate compressive strength or to compare relative compressive strength at different locations in the structure.

If in-place tests are to be used for estimating in-place compressive strength, a strength relationship that correlates compressive strength and the test measurement should be developed by testing cores that have been drilled from areas adjacent to the in-place test locations. An attempt should be made to obtain paired data (core strength and in-place test results) from different parts of the structure to obtain the full range of in-place compressive strength. Regression analysis of the correlation data can be used to develop a prediction equation along with the confidence limits for the estimated strength. For a given test method, the strength relationship is influenced to different degrees by the specific constituents of the concrete. For accurate estimates of concrete strength, general correlation curves supplied with test equipment or developed from concrete other than that in the structure being evaluated should not be used unless they have been verified by comparison of estimated strengths with measured core strengths. Therefore, in-place testing can reduce the number of cores taken but cannot eliminate the need for drilling cores from the structure.

If in-place tests are to be used only to compare relative concrete strength in different parts of the structure, it is not necessary to develop the strength relationships. If the user is not aware of the factors that can influence the in-place test results, it is possible to draw erroneous conclusions concerning the relative in-place strength.

Sections 3.2.3.1 through 3.2.3.5 summarize a number of currently available in-place tests that have been adopted as ASTM test methods and highlight some factors that have a significant influence on test results. [ACI 228.1R](#) has detailed information on developing strength relationships and on the statistical methods that should be used to interpret the results.

3.2.3.1 Rebound number—Procedures for conducting this test are given in [ASTM C805/C805M](#). The test instrument consists of a metal housing, a spring-loaded mass (the hammer), and a steel rod (the plunger). To perform a test, the plunger is placed perpendicular to the concrete surface and the instrument housing is pushed toward the concrete. This action causes the extension of a spring connected to the hammer. When the instrument is pushed to its limit, a catch is released and the hammer is propelled toward the concrete where it impacts a shoulder on the plunger. The hammer rebounds, and the rebound distance is measured on a scale numbered from 10 to 100. The rebound distance is recorded as the rebound number indicated on the scale, which represents the percentage of the original stretched length of the spring. A new instrument has been developed that measures a rebound index as the ratio of the speed of the hammer at rebound to the speed at impact. The rebound index determined in this manner is not affected by the orientation of the instrument during testing. For the same concrete, the two types of instruments do not result in the same value of the rebound index.

The rebound distance or speed depends on the value of the initial hammer energy is absorbed by the interaction

of the plunger with the concrete. The greater the absorbed energy, the lower the rebound number will be. A simple, direct relationship between rebound number and compressive strength does not exist. It has been shown empirically, however, that for a given concrete mixture, there is good correlation between the gain in compressive strength and the increase in the rebound number.

The concrete in the immediate vicinity of the plunger has the greatest effect on a measured rebound number. For example, a test performed directly above a hard particle of coarse aggregate will result in a higher rebound number than a test over mortar. To account for the variations in local conditions, **ASTM C805/C805M** requires averaging 10 rebound readings for a test result. Procedures for discarding abnormally high or low individual rebound values are also given.

The rebound number reflects the properties of the concrete near the surface and may not be representative of the rebound value of the interior concrete. A surface layer of carbonated or deteriorated concrete results in a rebound number that does not represent interior concrete properties. Carbonation densifies the surface and will result in high rebound values. Heavily textured, soft, or surfaces with loose mortar require surface grinding before testing. Rebound number increases as the moisture content of concrete decreases, and test results on a dry surface will not be representative of interior concrete that is moist. For instruments based on measuring rebound distance, the direction of the instrument (sideward, upward, downward) affects the rebound distance, so this should be considered if comparing readings or using correlation relationships. Manufacturers provide correction factors to account for varying hammer positions, but it is good practice to verify their accuracy if possible.

The rebound number is a simple and economical method for quickly obtaining information about the near-surface concrete properties of a structural member. Factors identified in **ASTM C805/C805M** and **ACI 228.1R** should be considered when evaluating rebound number results. Unless a project-specific correlation is developed using cores, this method is recommended only for assessing uniformity and locating regions with abnormal rebound values.

3.2.3.2 Probe penetration—The procedures for this test method are given in **ASTM C803/C803M**. The device is known commercially as the Windsor probe. The test involves the use of a special powder-actuated gun to drive a hardened steel rod (probe) into the surface of a concrete member. The penetration of the probe into the concrete is taken as an indicator of concrete strength.

The probe penetration test is similar to the rebound number test, except that the probe impacts the concrete with a much higher energy level. A theoretical analysis of this test is complex. Qualitatively, it involves the initial kinetic energy of the probe and absorption of that kinetic energy by friction and failure of the concrete. As the probe penetrates the concrete, crushing of mortar and aggregate occurs along the penetration path and extensive fracturing occurs within a conical region around the probe. Hence, the strength properties of aggregates and mortar influence penetration depth. This contrasts with the behavior of ordinary strength

concrete in a compression test, in which aggregate strength plays a secondary role compared with mortar strength. Thus, an important characteristic of the probe penetration test is that the type of coarse aggregate strongly affects the relationship between compressive strength and probe penetration.

Because the probe penetrates concrete, test results are not highly sensitive to local surface conditions such as texture and moisture content. The exposed lengths of the probes are measured, and a test result is the average of three probes located within 7 in. (180 mm) of each other. The probe penetration system has provisions to use a lower power level or a probe with larger tip diameter for testing relatively weak (less than 3000 psi [20 MPa]) or low-density (lightweight) concrete. Relationships between probe penetration and compressive strength are only valid for that specific power level and probe type.

In a manner similar to the rebound number test, this method is useful for comparing relative compressive strength at different locations in a structure. Strengths of cores taken from the structure and the statistical procedures detailed in **ACI 228.1R** are required to develop the correlation to permit compressive strength estimation on the basis of probe penetration results.

3.2.3.3 Pulse velocity—The procedures for this method are given in **ASTM C597**. The test equipment includes a transmitter, receiver, and electronic instrumentation. The test consists of measuring the time required for a pulse of ultrasonic stress-wave energy to travel through a concrete member. The ultrasonic energy is introduced into the concrete by the transmitting transducer, which is coupled to the surface with viscous acoustic couplant, such as petroleum jelly, water-soluble jelly, vacuum grease, or automotive grease. The pulse travels through the member and is detected by the receiving transducer, which is coupled to the opposite surface. The pulse transit time is measured by and displayed on associated instrumentation. The distance between the transducers is divided by the transit time to obtain the pulse velocity through the concrete under test. Most instruments allow the user to input the measured path length, and the pulse velocity is displayed along with the transit time.

The pulse velocity is proportional to the square root of the elastic modulus and inversely proportional to the square root of concrete density. The elastic modulus of concrete varies approximately in proportion to the square root of compressive strength. Hence, as concrete matures, large changes in compressive strength are accompanied by only minor changes in pulse velocity (**ACI 228.1R**). In addition, other factors affect pulse velocity, and these factors can easily overshadow changes due to strength. One such factor is moisture content. An increase in moisture content increases the pulse velocity, and this could be incorrectly interpreted as an increase in compressive strength. The presence of reinforcing steel aligned with the pulse travel path can also significantly increase pulse velocity. The user needs to be aware of the factors that can affect the measured pulse velocity and needs to ensure proper coupling to the concrete to obtain accurate values of the pulse velocity.

Under laboratory conditions, excellent correlations have been reported between pulse velocity and compressive strength development for a given concrete. These findings, however, should not be interpreted to mean that highly reliable estimates of in-place strength can be made routinely. Reliable strength estimates are possible only if correlation relationships include those characteristics of the in-place concrete that have a bearing on pulse velocity. Of paramount importance is the type and content of aggregate in the concrete, which have a strong effect on pulse velocity but generally have little effect on compressive strength for typical normal-density concrete. It is for this reason that the pulse velocity method is not generally recommended for estimating in-place strength. It is suitable for locating regions in a structure where the concrete is of a different quality or where there may be internal defects, such as cracking and honeycombing. It is not possible, however, to determine the nature of the defect based solely on the measured pulse velocity (3.2.5.2).

3.2.3.4 Pullout test—The pullout test consists of measuring the force required to pull an embedded metal insert out of a concrete member (refer to [ACI 228.1R](#) for illustration of this method). The force is applied by a jack that bears against the concrete surface through a reaction ring concentric with the insert. As the insert is extracted, a conical fragment of the concrete is also removed. The test produces a well-defined failure in the concrete and measures a static strength property. There is, however, no consensus on which strength property is measured and so a strength relationship should be developed between compressive strength and pullout strength ([Stone and Carino 1983](#); [Carino 2004a](#)). The relationship is valid only for the particular test configuration and concrete materials used in the correlation testing. Compared with other in-place tests, strength relationships for the pullout test are affected little by details of the concrete proportions. The strength relationship, however, depends on aggregate density (lightweight or normal density) and maximum aggregate size if greater than 1-1/2 in. (40 mm).

[ASTM C900](#) describes two procedures for performing pullout tests. In one procedure, the inserts are cast into the concrete during construction and the pullout strength is used to assess early-age in-place strength for timing of construction operations. The second procedure deals with post-installed inserts that can be used in existing construction. A commercial system is available for performing post-installed pullout tests ([Petersen 1997](#)), and the use of the system is described in [ACI 228.1R](#).

Other types of pullout-type test configurations are available for existing construction ([Mailhot et al. 1979](#); [Chabowski and Bryden-Smith 1979](#); [Domone and Castro 1987](#)). These typically involve drilling a hole and inserting an anchorage device that will engage in the concrete and cause fracture in the concrete when the device is extracted. These methods, however, do not have the same failure mechanism as in the standard pullout test, and they have not been standardized by ASTM.

3.2.3.5 Pull-off test for assessing in-place tensile strength of concrete and bond of overlay materials—The procedures

for the pull-off test are given in [ASTM C1583/C1583M](#), and additional guidance on the use and interpretation of tests results may be found in [ICRI 210.3](#). The test can be used to determine the near-surface tensile strength of concrete, the bond strength of a repair or overlay material to a concrete substrate, or the tensile strength of a repair or overlay material. This is a stand-alone test method and does not require the use of a preestablished strength relationship. The test is performed by drilling a shallow core perpendicular to the surface, leaving the intact core attached to the concrete substrate ([Bungey and Madandoust 1992](#)). After bonding a steel disk to the top surface of the core, a tensile load is applied to the disk until failure occurs. Alternatively, the disk can be bonded to the surface first and then the partial-depth core can be drilled. The tensile failure will occur in the weakest of four planes: in the concrete substrate; the interface between concrete and overlay; in the overlay material (if present); or at the interface of the steel disk and test surface. In preparing for the test, it is important that the disk axis be aligned with the direction of the applied tensile load and that the loading system does not introduce a bending moment during loading.

3.2.4 In-place tests for locating reinforcing steel—The size, number, and location of steel reinforcing bars need to be established to make an accurate assessment of structural capacity. In addition, embedded reinforcement needs to be located before drilling cores. A variety of electromagnetic devices, known as cover meters, are used for these purposes.

These devices have inherent limitations and it may be necessary to resort to radiographic methods for a reliable assessment of the reinforcement layout. Ground-penetrating radar (3.2.5.6) is also capable of locating embedded metallic objects, such as reinforcement. The following sections summarize these tools. Additional information can be found in [ACI 228.2R](#), [Malhotra and Carino \(2004\)](#), and [Bungey et al. \(2006\)](#).

3.2.4.1 Electromagnetic devices—There are two general types of electromagnetic devices for locating reinforcement in concrete. One type is based on the principle of magnetic reluctance, which refers to the resistance in creating magnetic flux in a material and is analogous to electrical resistance in an electric circuit. These devices incorporate a U-shaped search head (yoke) that includes two electrical coils wound around an iron core. One coil supplies a low-frequency alternating current that results in an alternating magnetic field and an alternating magnetic flux flowing through the bar between the ends of the yoke. The other coil senses the magnitude of the flux. If a steel bar is located within the path of the flux, the reluctance decreases and the magnetic flux is increased. The sensing coil monitors the increase in flux. Thus, as the yoke is scanned over the surface of a concrete member, a maximum signal is noted on the meter display when the yoke lies directly over a steel bar. [ACI 228.2R](#) provides additional discussion for these types of meters. With proper correlation, these meters can estimate the depth of a bar if its size is known or estimate the bar size if the depth of cover is known. Bar size can also be estimated by a dual measurement technique ([Bungey et al. 1977](#); [Dixon](#)

(1987) and Snell et al. (1988) report additional details on using cover meters. Magnetic reluctance meters are affected by the presence of iron-bearing aggregates or the presence of strong magnetic fields from nearby electrical equipment.

The other type of cover meter is based on the principle of eddy currents. This type of cover meter employs a probe that includes a coil excited by a high-frequency electrical current. The alternating current sets up an alternating magnetic field. The alternating current sets up an alternating magnetic field. If this magnetic field encounters a metallic object, circulating currents are created in the surface of the metal. These are known as eddy currents. The alternating eddy currents, in turn, give rise to an alternating magnetic field that opposes the field created by the probe. As a result, the current through the coil decreases. By monitoring the current through the coil, the presence of a metal object can be detected. These devices are similar to a recreational metal detector. More advanced eddy-current instruments are based on the pulse-induction technique. In this case, a voltage pulse is applied to the coil. The decaying magnetic field, which is created when the voltage pulse is turned off, induces decaying eddy currents in the surface of the bar. The decaying eddy currents, in turn, generate a decaying magnetic field that induces a current in the coil. The amplitude of the measured induced current depends primarily on the depth of the bar. Smaller probes are available to discriminate individual closely-spaced bars, and larger probes are used to increase the maximum cover that can be measured, which is typically approximately 4 in. (100 mm). Bar sizes can be estimated, generally to within ± 1 bar size, using special diameter probes or by making dual measurements over a bar with the sensor oriented alternately in two orthogonal directions. In the latter case, the ratio of the signal amplitudes from the two measurements can be related to the bar size.

An important distinction between these two types of meters is that reluctance meters detect only ferromagnetic objects, whereas eddy-current meters detect any type of electrically conductive metal. Cover meters are limited to detecting reinforcement located within approximately 4 in. (100 mm) of the exposed concrete surface. They are usually not effective in heavily reinforced sections, particularly sections with two or more adjacent or nearly adjacent layers of reinforcement. The ability to detect individual closely spaced bars depends on the design of the probe. Probes that can detect individual closely spaced bars, however, have limited depth of penetration. It is advisable to create a specimen composed of a bar embedded in a nonmagnetic and nonconductive material, such as dry sand, to verify that the device is operating correctly.

The accuracy of cover meters depends on the meter design, bar spacing, and thickness of concrete cover. The ratio of cover to bar spacing is an important parameter in terms of the measurement accuracy, and the manufacturer's instructions should be followed. It may be necessary to make a mockup of the member being tested to understand the limitations of the device, especially if more than one layer of reinforcement is present. Such mockups can be made by supporting bars in a plywood box or embedding bars in sand.

Results from cover meter surveys should be verified by drilling or chipping a selected area or areas as deemed neces-

sary to confirm the measured concrete cover and bar size or develop improved project-specific correlations (3.2.4.4).

3.2.4.2 Radiography—By using penetrating radiation, such as X-rays or gamma rays, radiography can determine the position and configuration of embedded reinforcing steel, post-tensioning strands, and electrical wires (ACI 228.2R). As the radiation passes through the member, its intensity is reduced according to the thickness, density, and absorption characteristics of the material. The quantity of radiation passing through the member is recorded on film similar to that used in medical applications, or radiation amplitude can be recorded using special image plates that produce a two-dimensional digital array of the radiation amplitude (Mariscotti et al. 2009). The length of exposure is determined by the sensitivity of the recording media, strength of radiation, distance from the source to detector, and thickness of concrete. Reinforcing bars absorb more energy than the surrounding concrete and show up as light areas on the exposed film. Cracks and voids, on the other hand, absorb less radiation and show up as dark zones on the film. Crack planes parallel to the radiation direction are detected more readily than crack planes perpendicular to the radiation direction. With digitally recorded data, signal-processing tools can be used to extract quantitative information such as loss in cross-sectional area due to reinforcement corrosion (Mariscotti et al. 2009). If multiple images are obtained with the source at different locations, three-dimensional (3-D) images of the internal structure can be reconstructed (Mariscotti et al. 2009).

Due to the size and large electrical power requirements of X-ray units to penetrate concrete, the use of X-ray units in the field is limited. Therefore, radiography of concrete is generally performed using the man-made isotopes, such as Iridium 192 or Cobalt 60. Gamma rays result from the radioactive decay of unstable isotopes. As a result, a gamma ray source cannot be turned off, and extensive shielding is needed to contain the radiation when not in use for inspection. The shielding requirements make gamma ray sources heavy and bulky, especially if high penetrating ability is required.

The penetrating ability of gamma rays depends on the type and activity (age) of the isotope source. Iridium 192 is practical up to 8 in. (200 mm) and can be used on concrete up to 12 in. (300 mm) thick, if time and safety permit. Cobalt 60 is practical up to approximately 20 in. (0.5 m) thickness. Additional penetration depth up to approximately 24 in. (0.6 m) can be obtained by the use of intensifying screens next to the film. For thicker structural elements, such as beams and columns, a hole may be drilled and the source placed inside the member. The thickness that can be penetrated is a function of the time available to conduct the test. The area to be radiographed needs access from both sides.

Radiographic inspection poses health hazards and needs to be performed only by licensed and trained personnel. One drawback to radiography is that it can interrupt tenant or construction activities should the exposure area need to be evacuated during testing. Because of the high gamma ray

concerns, radiography is usually the method of last resort if other methods are not successful.

As with other methods, results from radiographic tests should be verified by drilling or chipping selected areas as deemed necessary to confirm locations of reinforcing steel.

3.2.4.3 Ground-penetrating radar—Pulsed radar systems (3.2.5.6) can be used to locate embedded reinforcement. This method offers advantages over magnetic methods as a result of its greater penetration, which depends on the antenna frequency and the concrete moisture content. Access to one side of a member is all that is generally needed to perform an investigation. It is difficult to estimate bar size unless advanced processing of data is carried out (Zhan and Xie 2009). For a given antenna, the ability to discern individual bars in the received signal depends on the depth of the bars and their spacing. As cover increases, the bars have to be further apart to be discerned (Bungey et al. 1994). Interpretation of the results of a radar survey requires an experienced operator and should always be correlated to actual field measurements made by selected drilling or chipping.

3.2.4.4 Removal of concrete cover—This method removes the concrete cover to locate and determine the depth and size of embedded reinforcing steel, either by chipping or power drilling. This method is used primarily for verification of test results or for developing correlations with the results of the nondestructive methods outlined previously. Removal of concrete cover is the only reliable technique available to determine the condition of embedded reinforcing steel in deteriorated structures.

3.2.5 Nondestructive tests for identifying internal abnormalities—A strength evaluation may also involve determining if internal abnormalities exist that may reduce structural capacity, such as internal voids, cracks, or regions of inferior concrete quality. Compared with methods of strength estimation, some techniques for locating internal defects require complex instrumentation and specialized expertise to perform the tests and interpret the results. Refer to ACI 228.2R, Malhotra and Carino (2004), and Bungey et al. (2006) for additional information.

3.2.5.1 Sounding—Hollow areas or planes of delamination below the concrete surface can be detected by striking the surface with a hammer or a steel bar. A hollow or drum-like sound is heard when the surface over a hollow region, a delamination, or a region with shallow depth is struck. This compares with a higher-frequency ringing sound when the surface of undamaged and relatively thick concrete is struck. For slabs, such areas can be detected by dragging short lengths of steel chain over the concrete surface, unless the slab has a smooth, hard finish, in which case inadequate vibration is set up by the chain segments. Sounding is a simple and effective method for locating regions with subsurface fracture planes, but the sensitivity and reliability of the method decreases as the depth of the defect increases. For overhead applications, there are commercially available devices that use rotating sprockets on the end of a pole as a sounding method to detect delaminations. Procedures for using sounding in pavements and slabs can be found in ASTM D4580/D4580M.

3.2.5.2 Pulse velocity—The principle of pulse velocity is described in 3.2.3.3. Pulse travel time between the transmitting and receiving transducers is affected by the concrete properties along the travel path and the actual travel path distance. If there is a region of low-quality concrete between the transducers, the travel time increases and a lower velocity value is computed. If there is a relatively small void between the transducers, the pulse diffracts around the void as it travels through the concrete. This increases the actual path length and a lower pulse velocity is computed. While the pulse velocity method can be used to locate abnormal regions, it cannot identify the depth or the nature of the abnormality. Cores are often taken to determine the nature of the indicated abnormality.

3.2.5.3 Impact-echo—In the impact-echo method, a short-duration mechanical impact is applied to the concrete surface (Sansalone and Carino 1986). The impact generates stress waves that propagate away from the point of impact. The stress wave that propagates into the concrete is reflected if it encounters an interface between the concrete and a material with different acoustic properties. If the interface is between concrete and air, almost complete reflection occurs. The reflected stress wave travels back to the surface, where it is again reflected into the concrete and the cycle repeats. A receiving transducer located near the impact point monitors the surface movement resulting from the periodic arrival of the reflected stress wave. The transducer signal is recorded as a function of time from which the depth of the reflecting interface can be determined. If there is no defect, the thickness of the member can be determined, provided the thickness is small compared with the other dimensions. In general, the thickness has to be less than 20 percent of the smallest lateral dimension for the response to be dominated by reflections from the back wall. If this condition is not satisfied, reflections from the side boundaries will interfere with reflections from the back wall and interpretation of results becomes complicated.

Because the stress wave undergoes multiple reflections between the test surface and the internal reflecting interface, the recorded waveform is periodic. If the waveform is transformed into the frequency domain, the periodic nature of the waveform appears as a dominant peak in the amplitude spectrum (Carino et al. 1986). The frequency of that peak can be related to the depth of the reflecting interface by a simple relationship (Sansalone and Streett 1997).

Impact-echo can be used to measure the thickness of plate-like elements if there is access to only one face (ASTM C1383). A plate-like element is one in which the smallest lateral dimension is at least five times the thickness to be measured. Two procedures are required to measure the thickness. The first is to determine the stress-wave speed in the concrete, and the second is to measure the thickness frequency of the plate and calculate thickness. The wave speed can be established by a surface measurement technique using two transducers (Sansalone and Streett 1997), or it can be determined by measuring the thickness frequency, drilling a hole at the test point, measuring the thickness