ACI 437R-19

# Strength Evaluation of Existing Concrete Buildings

Reported by ACI Committee 437

American Concrete Institute
Always advancing

First Printing
May 2019
ISBN: 978-1-64195-059-6

# Strength Evaluation of Existing Concrete Buildings



The technical committees responsible for ACI committee reports and standards strive to avoid ambiguities, omissions, and errors in these documents. In spite of these efforts, the users of ACI documents occasionally find information or requirements that may be subject to more than one interpretation or may be incomplete or incorrect. Users who have suggestions for the improvement of ACI documents are requested to contact ACI via the errata website at http://concrete.org/Publications/DocumentErrata.aspx. Proper use of this document includes periodically checking for errata for the most up-to-date revisions.

ACI committee documents are intended for the use of individuals who are competent to evaluate the significance and limitations of its content and recommendations and who will accept responsibility for the application of the material it contains. Individuals who use this publication in any way assume all risk and accept total responsibility for the application and use of this information.

All information in this publication is provided "as is" without warranty of any kind, either express or implied, including but not limited to, the implied warranties of merchantability, fitness for a particular purpose or non-infringement.

ACI and its members disclaim liability for damages of any kind, including any special, indirect, incidental, or consequential damages, including without limitation, lost revenues or lost profits, which may result from the use of this publication.

It is the responsibility of the user of this document to establish health and safety practices appropriate to the specific circumstances involved with its use. ACI does not make any representations with regard to health and safety issues and the use of this document. The user must determine the applicability of all regulatory limitations before applying the document and must comply with all applicable laws and regulations, including but not limited to, United States Occupational Safety and Health Administration (OSHA) health and safety standards.

Participation by governmental representatives in the work of the American Concrete Institute and in the development of Institute standards does not constitute governmental endorsement of ACI or the standards that it develops.

Order information: ACI documents are available in print, by download, through electronic subscription, or reprint and may be obtained by contacting ACI.

Most ACI standards and committee reports are gathered together in the annually revised the ACI Collection of Concrete Codes, Specifications, and Practices.

**American Concrete Institute**
**38800 Country Club Drive**
**Farmington Hills, MI 48331**
**Phone: +1.248.848.3700**
**Fax: +1.248.848.3701**

**www.concrete.org**

ACI 437R-19

# Strength Evaluation of Existing Concrete Buildings

## Reported by ACI Committee 437

Paul H. Ziehl*, Chair

Aaron K. Larosche*, Secretary

Nicholas J. Carino*
Paolo Casadei
John A. Frauenhoffer*
Nestore Galati*
Zareh B. Gregorian
Pawan R. Gupta
Frederick D. Heidbrink*
Jason S. Johnson*
Ashok M. Kakade
Danielle D. Kleinhans
Andrew T. Krauklis
Carl J. Larosche*
Daniel J. McCarthy*
Antonio Nanni*
Thomas E. Nehil
K. Nam Shiu
J. Gustavo Tumialan*
Jeffrey S. West

**Consulting Members**

Marco Arduini
Grant T. Halvorsen
Habib M. Zeim Alabideen

*Members of the committee who prepared this report.

*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.

## CONTENTS





ACI 437R-19 supersedes 437R-03 and was adopted and published May 2019.

# 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 how much 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 (Tam 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. 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 necessary 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 cost and safety 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 at the hole, and calculating the wave speed from the measured frequency and thickness.

The impact-echo method can be used to detect internal abnormalities and defects, such as delaminations, regions of honeycombing, voids in grouted tendon ducts, subgrade voids, and the quality of interfaces in bonded overlays (Sansalone and Carino 1988, 1989; Jaeger et al. 1996; Wouters et al. 1999; Lin and Sansalone 1996). The test provides information on the conditions in the region directly below the receiving transducer and impact point. Thus, an impact-echo survey typically comprises many tests on a predefined grid. Care is required to establish the optimal spacing between test points (Kesner et al. 1999). The degree of success in a specific application depends on factors such as the shape of the member, the nature of the defect, and the experience of the operator. It is important that the operator understands how to select the impact duration and how to recognize invalid waveforms that result from improper seating of the transducer or improper impact (Sansalone and Streett 1997). Standardized test methods (ASTM) have not been developed for internal defect detection using the impact-echo method.

Devices are available, or are being developed, that will speed up the data acquisition process so that many points can be tested in a short period of time. Some devices incorporate a rolling transducer and an electro-mechanical impactor whereas others use a support frame with an electric motor to move a transducer-impactor assembly along a straight line and make measurements at selected points along the line. Software uses the closely-spaced, impact-echo data to reconstruct images of reflecting interfaces.

**3.2.5.4** *Impulse-response*—The impulse-response method is similar to the impact-echo method, except that a longer-duration impact is used, and the time history of the impact force is measured. The method measures the vibrational response of the portion of the structure surrounding the impact point (Davis et al. 1997). Measured response and the force history are used to calculate the impulse response spectrum of the structure (Carino 2004b). Depending on the quantity (displacement, velocity, or acceleration) measured by the transducer, the response spectrum has different meanings. Typically, the velocity of the surface is measured and the response spectrum represents the mobility (velocity/force) of the structure, which is affected by the geometry of the structure, the support conditions, and defects that affect the dynamic stiffness of the structure. An experienced user can extract several measures of the vibrational response that can be used to compare conditions at different test points (Davis and Dunn 1974; ACI 228.2R; Davis and Hertlein 1995).

ASTM C1740 provides a standard practice for impulse response testing. Parameters extracted from the impulse response test are plotted as contour plots from which it is possible to identify testing locations that have a different response from the rest of the structure. Those anomalous points can be subjected to more detailed investigation and the rest of the structure with similar and acceptable response can be assumed to be sound. Coring or other forms of invasive probing should be performed at the good and flawed locations to confirm the interpretations.

An impulse-response test result is affected by a larger volume of the structure surrounding the test point than the impact-echo method, but the test result cannot define the exact location or depth of a hidden defect. As a result, impulse-response testing is often used as a rapid screening method in conjunction with impact-echo or ultrasonic-echo testing for detailed investigation at identified anomalous locations.

**3.2.5.5** *Ultrasonic-echo*—The ultrasonic-echo method is a time-domain, reflection-based, stress-wave method for locating reflecting interfaces within a concrete member. It is based on the pitch-catch principle in which one transducer sends out a stress-wave pulse and another receives the reflected pulse, with both transducers being located on the same surface (ACI 228.2R). The travel time from the transmitter to the receiver is measured and, based on the wave speed and distance between the transducers, the depth of the reflecting interface can be calculated as explained in ACI 228.2R.

In one commercial device that has been developed, a computer-controlled antenna composed of a 4 x 12 array of transducers permits measurements of travel times between different pairs of transducers to be made in a few seconds. The transducers are point shear-wave transducers that do not require a viscous coupling fluid. The antenna array measures 3 x 13 in. (75 x 330 mm) and, in effect, it looks into the concrete beneath the antenna. The system can measure the concrete shear-wave speed or the user can provide an approximate value. An error in the wave speed value will only affect the indicated depth of the reflecting interfaces.

The computer in the antenna controls the operation of the 12 rows of transducers so that each row functions sequentially as a transmitter while the other rows act as receivers. The multiple travel time measurements from the various transducer pairs are used as input to a signal-processing technique called synthetic aperture focusing. The end result of the signal processing is an averaged two-dimensional (2-D) image of the reflecting interfaces in the volume of concrete below the antenna.

To test a concrete member, a grid is marked on the test surface and the grid spacing is entered into the instrument. At each test location, a 2-D cross-sectional image of the reflecting interface is computed. After the entire grid has been tested, the 2-D images are transferred to a computer along with the grid geometry. The computer uses 3-D visualization software to stitch together the 2-D images and create a 3-D representation of the reflecting interfaces in the concrete below the test grid. The process is analogous to the method used in medical imaging to reconstruct 3-D images of internal organs from 2-D images at different cross sections (tomograms).

The ultrasonic-echo instrument is able to detect the same types of defects as the impact-echo method, but it can do so more rapidly. It has proven useful for locating voids in grouted tendon ducts of post-tensioned members. Compared with impact-echo, the antenna of the ultrasonic-echo system does not have to be directly over the tendon duct to obtain a clear signal. The center frequency of the transmitted pulse in the commercial system can be changed from 25 to 85 kHz, which provides control over resolution and penetration depth.

**3.2.5.6** *Ground-penetrating radar*—This method is similar in principle to the other echo techniques except that electromagnetic energy is introduced into the material rather than mechanical energy. An antenna placed on the concrete surface (surface coupled antenna) sends out an extremely short-duration radio frequency pulse. The receiver in the antenna registers a strong signal due to the direct coupling of the transmitter and receiver through the near-surface concrete. The pulse travels into the concrete, and if the member contains boundaries between materials with different electrical properties, some of the energy in the pulse is reflected back to the antenna. The electrical property of interest is called the dielectric constant, which is related to the ability of a material to store electrical charge and it affects the speed of the pulse in the material (ACI 228.2R). Knowing the speed of the electromagnetic pulse in the concrete, the depth of the reflecting interface can be determined from the arrival time of the reflected pulse (ACI 228.2R). A digital recording system generates a profile view of the reflecting interfaces within the member as the antenna is moved over the surface. Changes in the reflection patterns indicate buried items, voids, and thickness of the member. Interpretation of the recorded profiles is the most difficult aspect of using commercially available radar systems. This method has been used successfully to locate embedded items, such as reinforcing steel and ducts, to locate regions of deterioration and voids or honeycombing, and to measure member thickness if access is limited to one side. The penetrating ability of the pulse depends on the electrical conductivity of the material and the frequency of the electromagnetic radiation. As electrical conductivity increases, pulse penetration decreases. In testing concrete, higher moisture content increases conductivity and reduces pulse penetration.

There are two ASTM standards on the use of ground-penetrating radar (GPR), both of which have been developed for highway applications. ASTM D4748 measures the thickness of bound pavement layers, and ASTM D6087 identifies the presence of delaminations in asphalt-covered bridge decks. With proper adaptation, these standards can be applicable to condition assessment in building structures. The Federal Communications Commission (FCC 2002) published rules that regulate the purchase and use of GPR equipment. Purchasers of GPR equipment are required to register their devices with the FCC and indicate where the devices will be used.

Ground-penetrating radar is especially helpful in locating reinforcing bars so that cores can, perhaps, be obtained free of reinforcement. As described in 3.2.4.3, GPR is an option for locating reinforcing bars. This application can be used to determine the reinforcement layout so that cores can be obtained free of reinforcement. In the simplest form of this application, GPR scans are performed in the region of interest and the detected reinforcement locations are marked on the surface of the concrete. Where more complete information is required, some GPR equipment can be used to create a 3-D model of the reinforcing steel in a section of the structure. This is accomplished by placing a plastic sheet with a permanently marked grid on the concrete surface. The antenna, with a distance-measuring transducer, is scanned along the gridlines in perpendicular directions. The acquired reflection data are manipulated by software to create a 3-D model of the portion of the structure below the test grid. The 3-D model can be manipulated to show the locations of the centerlines of the reinforcing bars. The sizes of the bars shown on the reconstructed images are, however, not to be interpreted as actual bar sizes. The image provides only the centerlines of the bars. Sophisticated signal processing is needed to estimate bar sizes from GPR data (Zhan and Xie 2009). Some GPR systems include an additional sensor that measures the magnetic field associated with alternating current in a metallic conductor. Such systems are useful to avoid cutting into live electrical conductors in the process of sampling concrete.

**3.2.5.7** *Infrared thermography*—A surface having a temperature above absolute zero emits electromagnetic energy. At room temperature, the wavelength of this radiation is in the infrared region of the electromagnetic spectrum. The rate of energy emission from the surface depends on its temperature, so by using infrared detectors it is possible to notice differences in surface temperature. If a concrete member contains an internal defect, such as a large crack or void, and there is heat flow through the member, the presence of the defect can influence the temperature of the surface above the defect. A picture of the surface temperature can be created by using an infrared camera, thereby enabling the location of hot or cold spots on the surface. The locations of these hot and cold spots serve as indications of the locations of internal near-surface defects in the concrete. The technique has been used successfully to locate regions of delamination in concrete pavements and bridge decks (ASTM D4788).

Heat flow through the member must be present to use infrared thermography. This can be achieved by the natural heating from sunlight or by applying a heat source to one side of the member. In addition, the member surface should be of one material and have a uniform value of a property known as emissivity, which is a measure of the efficiency of energy radiation by the surface. Changes in emissivity cause changes in the rate of energy radiation, which can be incorrectly interpreted as changes in surface temperature. The presence of foreign material on the surface, such as paint or grease, will affect the results of infrared thermography by changing the emissivity and, therefore, the apparent temperature of the surface. It is often useful to take a photographic or video record of the areas of the concrete surface being imaged by infrared photography. By comparing the two images, surface defects with different emissivity can be eliminated from consideration as internal defects in the concrete.

**3.2.5.8** *Radiography*—As discussed in 3.2.4.2, radiography can be used to determine the position and location of embedded reinforcing steel. Radiography can also be used to determine the internal condition of a structural member. As described previously, reinforcing bars absorb more energy than the surrounding concrete and show up as light areas on exposed film or digital images. Cracks and voids, on the other hand, absorb less radiation and show up as dark areas in the images. Crack planes parallel to the radiation direction are detected more readily than cracks perpendicular to the radiation direction.

## CHAPTER 4—METHODS FOR MATERIAL EVALUATION

This chapter describes procedures to assess the quality and mechanical properties of the concrete and reinforcing steel in a structure. These procedures are often used to corroborate the results of in-place or nondestructive methods mentioned in Chapter 3. Sampling techniques, petrographic and chemical analyses of concrete, and test methods are discussed. When material properties are not available or not found on drawings or specifications, values specified in ACI 562 may be considered. If material properties obtained from project documents are used, ACI 562 notes that additional testing may be required to confirm the information if degradation has occurred.

### 4.1—Concrete

The compressive strength of concrete is the most significant concrete property with regards to the strength evaluation of concrete structures. In-place concrete strength is a function of several factors, including the concrete mixture proportions, curing conditions, degree of consolidation, location within the structure, and deterioration over time. The following sections describe the physical sampling and direct testing of concrete to assess concrete strength. The condition of the concrete and extent of distress is indirectly assessed by strength testing because deterioration results in a strength reduction. An evaluation of the condition of the concrete and causes of deterioration may be obtained directly from petrographic and chemical analyses of the concrete.

**4.1.1** *Guidelines on sampling concrete*—The licensed design professional is responsible for determining the locations and the number of samples to be taken for establishing the in-place characteristics of the concrete (ACI 562). It is essential that the concrete samples be obtained, handled, identified (labeled), and stored properly to prevent damage or contamination. Sampling techniques are discussed in this section.

Guidance on developing an appropriate sampling program is provided by ASTM C823/C823M. Samples are usually taken to obtain statistical information about the properties of concrete in the entire structure, for correlation with in-place tests covered in Chapter 3, or to characterize some unusual or extreme conditions in specific portions of the structure (Bartlett and MacGregor 1996, 1997). For statistical information, sample locations should be randomly distributed throughout the structure. The number and size of samples depends on the necessary laboratory tests and the degree of confidence desired in the average values obtained from the tests.

The type of sampling plan that is required on a particular project depends on whether the concrete is believed to be uniform or if there are likely to be two or more regions that are different in composition, condition, or quality. In

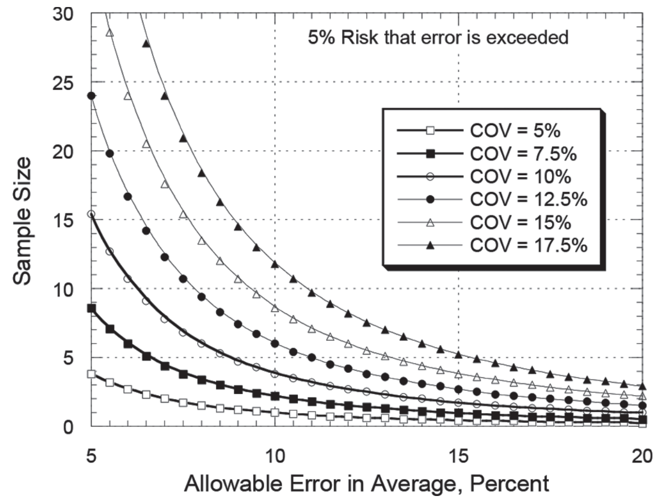


*Fig. 4.1.1—Sample size based on ASTM E122; risk = 5 percent.*

general, a preliminary investigation should be performed and other sources of information should be considered before a detailed sampling plan is prepared. Where a property is believed to be uniform, sampling locations should be distributed randomly throughout the area of interest and all data treated as one group. Otherwise, the study area should be subdivided into regions believed to be relatively uniform, with each region sampled and analyzed separately.

There are different approaches for choosing sample locations. Williams et al. (2006a,b) discuss the use of random, stratified, and adaptive sampling methods for planning nondestructive testing investigations of large concrete structures to determine the proportion of the structure that is flawed. Adaptive sampling is a method in which the next sampling location is based on the results from previous locations. From limited case studies, Williams et al. (2006a,b) determined that random sampling produced the narrowest confidence intervals, but noted that access and cost considerations may not always allow random sampling.

For tests intended to measure the average value of a concrete property, such as strength, elastic modulus, or air content, the number of samples generally depends on:

a) The maximum allowable difference (or error) between the sample average and the true average

b) The variability of the test results

c) The acceptable risk that the maximum allowable difference is exceeded

These factors can be taken into account by using ASTM E122 to estimate the sample size, as shown in Fig. 4.1.1. The vertical axis gives the number of samples needed as a function of:

a) The maximum allowable difference (as a percentage of the true average) between the sample average and true average

b) The coefficient of variation of the test results

In Fig. 4.1.1, the risk that the maximum allowable error will be exceeded is 5 percent, but other levels can be used. Because the variability of test results is usually not known in advance, an initial estimate can be made and adjusted as test results become available.

Economy should also be considered in the selection of sample sizes. In general, uncertainty in an average value is related to the inverse of the square root of the number of results used to compute that average. For large sample sizes, an increase in the sample size will result in only a small decrease in the risk that the acceptable error is exceeded. The cost of additional sampling and testing would not be justified in these situations.

Concrete is neither isotropic nor homogenous, and so its properties will vary depending on the direction that samples are taken and the position within a member. Close attention should be given to vertical concrete members, such as columns, walls, and deep beams, because concrete properties will vary with elevation due to differences in placing and compaction procedures, segregation, and bleeding. Typically, the strength of concrete at the bottom of a placement will be greater than at the top of the placement (Bartlett and MacGregor 1999).

**4.1.1.1** *Core sampling*—The procedures for removing concrete samples by core drilling are described in ASTM C42/C42M, and ACI 214.4R provides additional guidance on factors to consider in obtaining cores. The following guidelines are of particular importance in core sampling:

a) Cores should be taken using water-cooled, diamond-studded core bits. Drills should be in good operating condition and supported rigidly so that the cut surfaces of the cores will be as straight as possible.

b) The number, size, and location of core samples should be selected to permit all necessary laboratory tests. If possible, use separate cores for different tests so that there will be no influence from prior tests.

c) Cores to be tested for a strength property should have a minimum diameter of at least twice the maximum nominal size of the coarse aggregate, or 3.70 in. (94 mm), whichever is greater. The use of small-diameter cores results in lower and more erratic strengths (Bungey 1979; Bartlett and MacGregor 1994a).

d) If possible, cores to be tested for a strength property should have a length of twice their diameter.

e) Embedded reinforcing steel should be avoided in a core to be tested for compressive strength. If cores cannot be obtained without embedded steel, ASTM C42/C42M permits testing cores with pieces of reinforcing steel, but engineering judgment may be needed to interpret the test results (Gaynor 1965). The criteria that will be used to evaluate measured strengths of cores with embedded reinforcement should be agreed to by the interested parties before beginning the coring program.

f) Avoid cutting electrical conduits or prestressing steel. Use cover meters (3.2.4.1) or ground-penetrating radar (3.2.4.3) to locate embedded metal items prior to drilling.

g) Select core locations to have the least effect on member strength.

h) If possible, core drilling should completely penetrate the concrete section to avoid having to break off the core to facilitate removal. If through-drilling is not feasible, the core should be drilled approximately 2 in. (50 mm) longer than required allowing for possible damage at the base of the core.

i) If cores are taken to determine strength, the number of cores should be based on the expected uniformity of the concrete and the desired confidence level in the average strength, as discussed in 4.1.1. The strength value should be taken as the average of the cores. A single core should not be used to evaluate or diagnose a particular problem.

**4.1.1.2** *Random sampling of broken concrete*—Sampling of broken concrete generally should not be used if strength of concrete is in question. Broken concrete samples, however, can be used in some situations for petrographic and chemical analyses in the evaluation of deteriorated concrete members.

**4.1.2** *Petrographic and chemical analyses*—Petrographic and chemical analyses of concrete are important tools for the strength evaluation of existing structures, providing valuable information related to the concrete composition, present condition, and potential for future deterioration. The concrete characteristics and properties determined by these analyses can provide insight into the nature and forms of the distress.

**4.1.2.1** *Petrography*—The techniques used for a petrographic examination of concrete or concrete aggregates are based on those developed in petrology and geology to classify rocks and minerals. The examination is generally performed in a laboratory using cores removed from the structure. The cores are cut into sections and polished before microscopic examination. Petrography may also involve analytical techniques, such as scanning electron microscopy (SEM), X-ray diffraction (XRD), infrared spectroscopy, and differential thermal analysis. ASTM C1723 provides guidance on the use of energy-dispersive X-ray spectroscopy (EDX) to obtain information on the elemental composition of the specimens placed in the SEM.

A petrographic analysis is normally performed to determine the composition of concrete, assess the adequacy of the mixture proportions, and determine the cause(s) of deterioration. A petrographic analysis performed in accordance with ASTM C856 can provide some of the following information about the concrete:

a) Density of the cement paste and color of the cement
b) Type of cement used
c) Proportion of unhydrated cement
d) Presence of pozzolans or slag cement
e) Volumetric proportions of aggregates, cement paste, and air voids
f) Homogeneity of the concrete
g) Presence and type of fibers (fiber-reinforced concrete)
h) Presence of foreign materials, including debris or organic materials
i) Aggregate shape, size distribution, and composition
j) Nature of interface between aggregates and cement paste
k) Extent to which aggregate particles are coated and the nature of the coating substance
l) Potential for deleterious reactions between the aggregate and cement alkalis
m) Presence of unsound aggregates (fractured or porous)

n) Air content and various dimensional characteristics of the air-void system, including entrained and entrapped air
o) Characteristics and distribution of voids
p) Occurrence of settlement and bleeding in fresh concrete
q) Degree of consolidation
r) Presence of surface treatments

Petrography can also provide information on the following items to aid in the determination of causes of concrete deterioration:
a) Occurrence and distribution of fractures
b) Presence of contaminating substances
c) Surface-finish-related problems
d) Curing-related problems
e) Presence of deterioration caused by exposure to freezing and thawing
f) Presence of reaction products in cracks or around aggregates, indicating deleterious alkali-aggregate reactions
g) Presence of ettringite within cement paste (other than in pore system or voids) and in cracks indicating sulfate attack
h) Presence of corrosion products
i) Presence of deterioration due to abrasion or fire exposure
j) Weathering patterns from surface-to-bottom

The standard procedures for the petrographic examination of samples of hardened concrete are addressed by ASTM C856. Procedures for a microscopic assessment of the concrete air-void system, including the air content of hardened concrete and of the specific surface, void frequency, spacing factor, and paste-air ratio of the air-void system, are provided in ASTM C457/C457M. ASTM C295/C295M contains procedures specific to petrographic analysis of aggregates. Powers (2002), Mailvaganam (1992), and Erlin (1994) provide additional information on petrographic examination of hardened concrete. Mielenz (1994) describes petrographic examination of concrete aggregates in detail.

Concrete samples for petrographic analysis should be collected as described in 4.1.1 and following ASTM C823/C823M. If possible, a qualified petrographer who is familiar with problems commonly encountered with concrete should be consulted before the removal of samples from an existing structure. If the petrographic analysis is being used to assess observed concrete distress or deterioration in a structure, samples for analysis should be collected from locations in the structure exhibiting distress, rather than in a random manner as used in a general assessment (4.1.1).

The petrographer should be provided with information regarding the preconstruction, construction, and post-construction history and performance of the structure. Specific items of interest include:
a) Original concrete mixture proportions, including information on chemical admixtures and supplementary cementitious materials
b) Concrete surface treatments or coatings
c) Curing conditions
d) Placement conditions, including concrete temperature, air temperature, ambient humidity, and wind conditions
e) Placement and finishing techniques
f) Location and orientation of core or sample in structure


*Fig. 4.1.2.2—Depth of carbonation as indicated by color change using a pH indicator solution.*

g) Exposure conditions during service
h) Description of distressed or deteriorated locations in structure, including photographs

**4.1.2.2** *Chemical tests*—Chemical testing of concrete samples can provide information on the presence or absence of various compounds and on forms of deterioration. In addition, chemical tests can be used to gauge the severity of various forms of deterioration and, in some cases, to predict the potential for future deterioration if exposure conditions remain unchanged. Examples of chemical testing for concrete include determination of cement content, chemical composition of cementitious materials, presence of chemical admixtures, content of soluble salts, detection of alkali-silica reactions, depth of carbonation, and chloride content. One of the more common uses of chemical testing is to measure the depth of carbonation and chloride concentration to assess the risk of reinforcement corrosion (corrosion mechanisms and factors for corrosion are discussed in detail in ACI 222R, ACI 222.2R, and ACI 222.3R).

Carbonation contributes to the risk of reinforcing steel corrosion by disrupting the passivity of the steel. More specifically, concrete carbonation exists if the pH of the concrete is reduced to approximately 9 or less (ACI 222R). Chemical testing to determine the depth of carbonation can be accomplished by splitting a core lengthwise and applying a solution of phenolphthalein indicator to the freshly fractured core surface. The indicator changes from colorless to a magenta color above a pH of 9. Thus, the depth of carbonation can be measured by determining the depth of material not undergoing a color change to magenta upon application of phenolphthalein indicator. Alternatively, a pH indicator solution can be applied to the freshly fractured surface. Such a solution displays different colors depending on the pH value. For example, Fig. 4.1.2.2 shows the carbonation front on a concrete core as evidenced by the color variation after applying a pH indicator solution. The abrupt color change from yellow (light shade) to purple (dark shade) indicates the depth of carbonation. Any steel within this depth could be vulnerable to carbonation-induced corrosion.

The presence of chloride ions in the concrete at the level of the reinforcement is the most common cause of reinforcement corrosion. Chlorides can be present in the concrete from the mixture constituents or due to external sources, including exposure to a marine environment or chloride-based deicing chemicals. If the chloride concentration reaches a threshold level at the reinforcement surface, corrosion of the reinforcement may begin in the presence of adequate oxygen and moisture. Thus, testing to determine chloride ion concentration is used to determine whether chloride levels are above the corrosion threshold and to predict the time to corrosion initiation (information on service-life prediction is provided in ACI 365.1R). A full assessment of corrosion risk will include the development of a chloride concentration profile of the concrete by collecting and testing samples at multiple depths from near the surface of the concrete at or below the level of reinforcement. Chemical analysis for chloride concentration is performed on powdered samples of concrete. Samples may be collected using a rotary impact drill or using cores. In the first method, concrete powder from the drilling operation is carefully collected at several depths. If cores are used, the core is cut into 0.5 in. (13 mm) thick slices at the depths of interest, and the concrete is crushed to powder for analysis. Guidance on both collection techniques is provided in ASTM C1152/C1152M, ASTM C1218/C1218M, and AASHTO T 260. Alternatively, powder samples at carefully controlled depth increments can be obtained by a device known as a profile grinder. At each depth increment, a portion of the surface with a diameter of 2.9 in. (73 mm) is removed.

Depending on the evaluation objective(s) and criteria, the samples are tested for water-soluble or acid-soluble chloride concentration (ACI 222R provides detailed information on water- and acid-soluble chlorides). Sample preparation for water-soluble and acid-soluble chloride levels is addressed in ASTM C1218/C1218M and ASTM C1152/C1152M, respectively. The chloride concentration is determined by potentiometric titration of the prepared sample with silver nitrate, as described in ASTM C114. Commercial kits for rapid (acid-soluble) chloride concentration testing using a calibrated chloride ion probe are also available. AASHTO T 260 addresses this field method for determining acid-soluble or total chloride content. ACI 222R provides more information on chloride thresholds for corrosion and chloride testing. Also, testing for the presence of corrosion inhibitors can be important in assessing the likely effect of chloride contamination on the anticipated performance of the structure.

**4.1.3** *Testing concrete for compressive strength*—Direct measurement of the concrete compressive strength in an existing structure can only be achieved through removal and testing of cores. In-place or nondestructive test methods can be used to estimate compressive strength if used in conjunction with core testing.

**4.1.3.1** *Testing cores*—Compressive strength of concrete cores taken from an existing structure and conditioned in accordance with ASTM C42/C42M should be determined in accordance with ASTM C39/C39M. Key points in this procedure are:

a) The mass of each core tested for compressive strength should be measured in accordance with ASTM C42/C42M. The mass and measured core dimensions are used to calculate the approximate density of the core, which may be of value in the event of unexpected compressive strength results.

b) For core length-diameter ratios less than 1.75, the appropriate strength correction factors given in ASTM C42/C42M should be applied to obtain the core strength for a length-diameter ratio of 2. These correction factors are approximate and engineering judgment should be exercised in evaluating the corrected core strength (Bartlett and MacGregor 1994b).

c) Unless specified otherwise, cores should be moisture-conditioned in accordance with the default procedure given in ASTM C42/C42M. This procedure involves keeping the cores in watertight container(s) for a specified time after they were last wetted. The time of last wetting is either when the core was drilled or at the completion of the last end treatment that involved wetting the core. The intent of the conditioning procedure is to preserve the in-place moisture content and reduce moisture gradients. Excessive moisture gradients in the cores will reduce the measured compressive strength (Bartlett and MacGregor 1994c). Additional discussion on the importance of moisture conditioning is provided by Neville (2001). For core testing related to the evaluation of concrete due to low strength test results for standard-cured cylinders during construction, ACI 318 requires that the cores be tested between 48 hours and 7 days after they were obtained, unless otherwise approved by the licensed design professional. For core testing related to the strength evaluation of an existing concrete structure, careful consideration should be given to whether procedures for moisture conditioning of cores should differ from the default procedure specified in ASTM C42/C42M.

d) Care should be exercised in end preparation of cores before testing for compressive strength. If capping compound is used, its thickness is limited by ASTM C617/C617M. This is especially critical for high-strength concrete. If cores are tested using unbonded caps in accordance with ASTM C1231/C1231M, the inner diameter of the retaining rings should be between 102 and 107 percent of the core diameter. There are limitations on the flatness of the core ends to use one of these capping procedures; therefore, saw cutting of one or both ends may be required. If ends of cores are ground, verification of flatness is required to ensure that the requirements of ASTM C39/C39M are met.

e) Core compressive strengths may be expected to be lower for cores removed from the upper portions of slabs, beams, footings, walls, and columns than from lower portions of such members (Bartlett and MacGregor 1999).

f) The interpretation of core strengths is not a simple matter. Involved parties should agree on the evaluation criteria before sampling begins (Neville 2001).

**4.1.3.2** *Equivalent specified strength*—Depending on age, temperature history, moisture condition during curing, and strength level, compressive strength values obtained from core tests can either be lower or higher than those obtained from tests of standard-cured cylinders molded from samples of concrete taken during construction. It has been reported that for mature concrete, the core strength varies from 100 percent of the of the standard-cylinder strength for 3000 psi (20 MPa) concrete to 70 percent for 9000 psi (60 MPa) concrete (Mindess and Young 1981). It has been practice to divide measured core strengths by 0.85 and consider the resulting value as the equivalent cylinder strength (Hanson 2007). The basis for this practice is the work of Bloem (1965, 1968) in which it was found that, for well-cured concrete slabs, the strengths of cores drilled from the slabs were approximately 85 percent of the strength of companion standard-cured cylinders. There are many factors that affect the relationship between core strengths and the strengths of companion standard-cured cylinders; therefore, a single ratio is not applicable to all situations. In addition, for a strength evaluation, the objective is not to determine equivalent cylinder strength, but to estimate the equivalent in-place specified strength based on the strength variation measured by cores. Thus, the inappropriate practice of dividing core strengths by 0.85 is discouraged because rational procedures exist for making more reliable estimates of the equivalent in-place strength by considering the major factors that affect measured core strengths (Bartlett and MacGregor 1995).

In evaluating structural capacity, the equivalent specified strength for the concrete in the structure should be established on the basis of core strengths. The equivalent in-place specified strength represents the in-place compressive strength that is expected to be exceeded by approximately 90 percent of the concrete in the structure and is used in calculating nominal member capacities. The commentary to Chapter 27 of ACI 318-14 refers to the methods in ACI 214.4R as an acceptable procedure for determining the equivalent specified strength of the in-place concrete on the basis of cores. Section 6.4.3 of ACI 562-16, on the other hand, provides a specific statistical relationship to calculate the equivalent specified strength from the measured core strengths, which have been modified to account for core diameter and moisture condition. These procedures for estimating the equivalent in-place specified strength are based largely on work by Bartlett and MacGregor (1995).

The value of the calculated equivalent specified strength depends on the number of cores tested and the variability of the core strengths. Figure 4.1.3.2 shows the value of the equivalent specified strength as a fraction of the average in-place strength for different numbers of cores and coefficients of variation of the core test results based on Chapter 6 of ACI 562-16. The figure shows that the equivalent specified strength is a smaller fraction of the average strength with increasing variability of test results and with lower number of tests.

**4.1.3.3** *In-place tests*—Currently, there are no in-place tests that provide direct measurements of compressive strength of concrete in an existing structure. In-place tests, however, can be used to estimate in-place compressive strength, provided correlations that are applicable to the concrete in the structure have been established. In-place tests are commonly used in conjunction with tests of drilled cores to reduce the amount of coring required to estimate compressive strengths throughout a large structure. These indirect tests

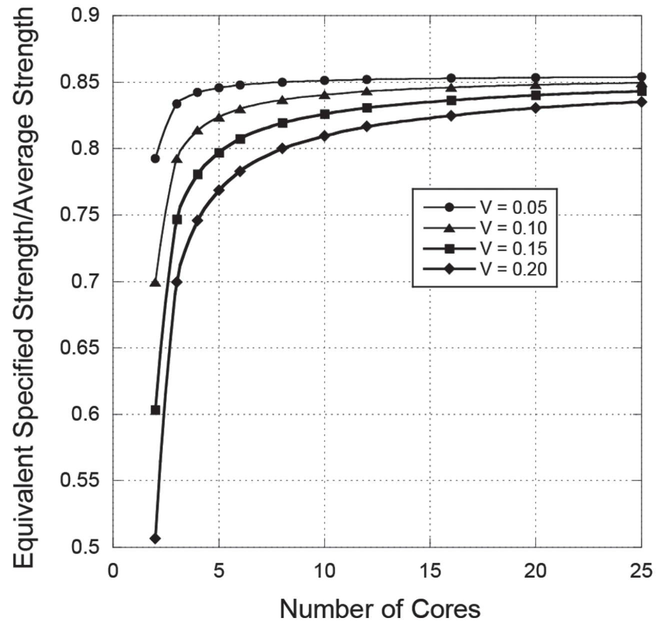


*Fig. 4.1.3.2—Ratio of equivalent specified strength to average in-place compressive strength as a function of the number of cores and coefficient of variation of core strengths based on ACI 562.*

do not have the same degree of uncertainty, and appropriate statistical methods are necessary to establish valid estimates of compressive strength based on indirect test results. Refer to ACI 228.1R and 3.2.3 for further information.

## 4.2—Reinforcing steel

**4.2.1** *Determination of yield strength*—The yield strength of the reinforcing steel can be established by one of three methods as discussed in Chapter 6 of ACI 562-16. Information from project documents, including drawings; specifications; or previous testing, including mill test reports, may be used to determine yield strength. If material properties from original construction reports are used, the commentary of ACI 562 suggests that additional testing may be required to confirm these properties if degradation has occurred. If mill test reports furnished by the manufacturer of the reinforcing steel are used, the design professional and the building official should be in agreement. Yield strengths from mill test reports tend to be greater than those obtained from tests of field samples. If existing documentation of yield strength is not available or desirable, yield strength may be determined either by the historical values presented in Chapter 6 of ACI 562 or by sampling and destructive testing of specimens taken from the structure. Guidelines for sampling are provided in 4.2.2. ACI 562 allows the design professional to use a yield strength value of 27,000 psi (186 MPa) in place of testing if historic information is not available.

The Concrete Reinforcing Steel Institute (2014) provides information on reinforcing systems in older structures, including reinforcing bar specifications, yield strengths, sizes, and allowable stresses. Note that the hardness reported in historic standards may not necessarily correlate to current metallurgical hardness measures.

**4.2.2** *Sampling techniques*—If the yield strength of embedded reinforcing steel is determined by testing, the recommendations listed in the following should be followed:

a) An equivalent yield strength should be calculated using Eq. (6.4.8) presented in ACI 562.

b) A minimum of three test samples should be used to compute an equivalent yield strength.

c) Tension test specimens should be the full section of the bar (ASTM A370-18 Annex 9). Requirements for specimen length, preparation, testing, and determination of the yield strength are provided by ASTM A370.

d) If bar samples meeting the length requirements of ASTM A370-18 Annex 9, cannot be obtained, samples may be prepared (machined) according to the general requirements of ASTM A370 for testing and determination of mechanical properties.

e) Samples should be removed at locations of minimum stress in the reinforcement.

f) To avoid excessive reduction in member strength, no two samples should be removed from the same cross section (location) of a structural member.

g) Locations of samples in continuous concrete construction should be separated by at least the development length of the reinforcement to avoid excessive weakening of the member.

h) For single structural elements having a span of less than 25 ft (7.5 m) or a loaded area of less than 625 ft$^2$ (60 m$^2$), at least one sample should be taken from the main longitudinal reinforcement (not stirrups or ties).

i) For longer spans or larger loaded areas, more samples should be taken from locations well distributed through the portion being investigated.

j) Sample locations should be chosen to determine whether the same strength of steel was used throughout the structure.

Sampling of prestressed reinforcement, whether from bonded or unbonded systems, is a complex undertaking and beyond the scope of this report. Discussion of extraction of unbonded single-strand tendons for testing can be found in ACI 423.4R.

**4.2.3** *Additional considerations*—The strength evaluation of concrete structures can require consideration of several reinforcement-related factors in addition to the yield strength, such as development length, anchorage, and reduction in cross section or bond due to corrosion.

Reinforcing bars manufactured before 1947 are sometimes smooth or have deformation patterns not meeting modern requirements. As a result, the bond and development of these bars can be significantly different from those of modern reinforcement (Concrete Reinforcing Steel Institute 2014). Similarly, changes to details and assumptions for standard hooks can affect the development of hooked bars in older structures. For structures with reinforcing bars manufactured before 1947, the Concrete Reinforcing Steel Institute (2014) conservatively recommends assuming that the required development length is twice that based on current code provisions. Concrete deterioration, especially spalling of concrete due to corrosion or other damage mechanism, will adversely affect the development of reinforcing steel.

Corrosion of reinforcement can lead to reduction in member capacity and ductility as a result of reinforcement section loss or disruption of bond. Because reinforcement corrosion normally results in disruption and cracking of the concrete surrounding the bar, bond to the concrete will be negatively affected in addition to loss of cross-sectional area. As a result, where bond is important, the reduction in structural capacity can be higher than that based solely on the reduction of the cross-sectional area of the bar. A conservative approach should be used in assessing the residual capacity of damaged or corroded reinforcement. Special consideration should be given to situations where corrosion of prestressing steel is suspected (ACI 222.2R). Tests for determining corrosion activity include measuring half-cell potential (ASTM C876) and linear polarization resistance. Refer to ACI 222R and ACI 228.2R for additional information on these types of tests.

## 4.3—Fiber-reinforced polymer reinforcement systems

Fiber-reinforced polymer (FRP) systems used for strengthening of existing concrete structures come in a variety of forms, including wet layup systems; pultruded (shop-fabricated) systems, commonly referred to as laminates; and near-surface-mounted (NSM) FRP systems. ACI 440.2R provides information on different commercially available FRP systems for strengthening of concrete structures. Wet layup FRP systems consist of fiber sheets or fabrics (typically made of carbon and glass) impregnated with a saturating resin on site and bonded to the concrete surfaces. The fiber sheets can be bound together in multiple plies. Pultruded FRP systems are typically thin composite shapes manufactured off site and are also bonded to the concrete surfaces with resins. Near-surface-mounted FRP systems consist of circular or rectangular bars or plates installed and bonded into grooves cut in the concrete surface.

Material properties of installed FRP used to strengthen concrete elements can be obtained from project documents such as drawings, specifications, or results of material testing performed as quality control during construction, which can include FRP properties from tests of field sample panels or witness panels. It is not practical to extract samples to determine mechanical properties such as tensile strength and modulus of elasticity because FRP systems are adhered to the concrete substrate with resins. Therefore, samples cannot be easily retrieved without damaging the FRP itself. Small core samples, typically 0.5 in. (13 mm) in diameter, can be taken to measure the thickness of cured laminate thickness or number of plies. The licensed design professional should specify the sampling frequency. ICRI 330.2 provides guidance on sampling frequency.

The performance and effectiveness of FRP over time can be evaluated as one or more of the following procedures:

a) Visual survey of the FRP system for signs of debonding of FRP laminates or near-surface-mounted FRP bars, peeling or blistering of the FRP laminates, changes in color, and cracking of the resin used to bond the FRP to concrete

b) Visual survey of the strengthened concrete element for signs of distress such as cracking or excessive deflections, and underlying damage such as corrosion of the internal steel reinforcement

c) Sounding (hammer tap) of laminates for hollow-sounding areas to identify areas of debonding (delaminations) from the concrete surfaces. The licensed design professional should evaluate the effect of delaminations on the structural integrity and durability of the FRP system. The size, locations, and quantity of delaminations should be considered in the evaluation. ACI 440.2R provides guidance for acceptance guidelines. In general:

i. Small delaminations, less than 2 in.$^2$ (1290 mm$^2$), are acceptable if the delaminated area is less than 5 percent of the total laminate area and there are no more than 10 such delaminations per 10 ft$^2$ (0.93 m$^2$).

ii. Large delaminations, greater than 25 in.$^2$ (16,130 mm$^2$), should be repaired by removing delaminated areas and installing and overlapping a laminate patch of equivalent strength.

iii. Delaminations less than 25 in.$^2$ (16,130 mm$^2$) may be repaired by epoxy injection or laminate replacement, depending on the size and number of delaminations and their locations.

d) Pull-off tension tests evaluate the bond strength of FRP laminates to the concrete substrate. ASTM D7522/D7522M describes test procedures for evaluating the pull-off strength of FRP laminates bonded to concrete surfaces. ICRI 210.3 provides additional guidance regarding pull-off tests. Testing should not be performed at critical areas (for example, area of maximum flexural demand for FRP flexural strengthening) and splice areas should be avoided. Typical sample areas are at the ends of the FRP strengthening. The licensed design professional should specify the testing frequency (ICRI 330.2). Tension adhesion strengths should exceed 200 psi (1380 kPa) and should fail in the concrete substrate. Lower strengths or failure between the FRP laminate and the concrete or between laminate plies should be evaluated by the licensed design professional for acceptance. Cored samples from bond testing can also be used to measure the laminate thickness or number of plies.

e) Ultrasonic or thermographic tests for indications of progressive delamination.

f) For near-surface-mounted strengthening, cores can be extracted to measure the bar size and visually verify the consolidation of the adhesive around the FRP bar. As in the case of laminates, the sample location should not be at critical areas.

g) Load testing to evaluate the overall performance of the element strengthened with FRP. ACI 437.2 provides requirements for load testing of concrete elements.

# CHAPTER 5—ASSESSMENT OF LOADING CONDITIONS AND SELECTION OF EVALUATION METHOD

## 5.1—Assessment of loading and environmental conditions

A fundamental aspect of any strength evaluation is the assessment of the loads and environmental conditions, past, present, and future. These should be accurately defined so that the results of the strength evaluation process will be realistic.

**5.1.1** *Dead loads*—Dead loads consist of the self-weight of the structure and any superimposed dead loads.

**5.1.1.1** *Self-weight of structure*—The self-weight of the structure can be estimated using field-measured dimensions of the structure and material densities as presented in ASCE 7. Dimensions obtained solely from design drawings should be used with caution because significant differences can exist between dimensions shown on design drawings and actual, as-built dimensions. Similarly, differences can exist between material densities obtained from ASCE 7 and actual in-place densities due to variations in moisture content, material constituents, and other reasons. If differences in densities are suspected, field samples should be analyzed.

**5.1.1.2** *Superimposed dead loads*—Superimposed dead loads include the weight of all materials incorporated into the structure, exclusive of the self-weight of the structure. Examples include the weight of architectural floor and ceiling finishes; nonstructural topping slabs or overlays; partitions; mechanical systems; fixed service equipment such as cranes and exterior cladding; and landscaping such as fixed planters, soils, and plantings. The magnitude of superimposed dead loads can be estimated by performing a field survey of the structure for such items and using appropriate values for loads as presented in ASCE 7 or other reference sources. Consideration should be given to superimposed dead loads that may not be present at the time of the evaluation but may be applied over the life of the structure.

**5.1.2** *Live loads*—The magnitude, location, and orientation of live loads on a structural component depend on the intended use of the structure. Past, present, and future usage conditions should be established accurately so that appropriate assumptions can be made for the selection of live loads. For evaluation, refer to Chapters 1, 4, and 5, and Appendix A of ACI 562-16 regarding which building code edition should be used in establishing the live loads. In cases where the structure's occupancy does not align with the prescribed live load categories, the live load can be established if a rational assessment of past, present, or future usage is prudent and reasonable.

If the serviceability of a structure is to be evaluated in addition to strength, the live loads that will be present during normal occupancy of the structure should be estimated. Estimates of live loads can be obtained from field surveys and measurements of loads in other structures with similar occupancies. In many instances, the day-to-day live loads are much lower than the design live loads prescribed in the local building code. Data from surveys of live loads in buildings are presented in the commentary to ASCE 7. Data from surveys of live loads in parking structures are presented in Wen and Yeo (2001).

**5.1.3** *Wind loads*—ASCE 7 provides guidance to determine wind loads. Site-specific historical wind-speed information can be obtained from the National Climatic Data Center of the National Oceanic and Atmospheric Administration (NOAA).

**5.1.4** *Rain loads*—When evaluating roofs, loads that result from ponding or pooling of rainwater due to the nature of the roof profile, deflections of framing members, or improper roof drainage should be considered.

**5.1.5** *Snow and ice loads*—Consider the possibility of partial snow loading, unbalanced roof snow loads, drifting snow loads, and sliding snow loads as defined in ASCE 7. When estimating ground snow loads, consider local and regional geographical locations. In the absence of specific requirements in the local building code, refer to ASCE 7 and information available from NOAA. Special attention is required if the structure is located near a large body of water that could be the source of moisture leading to ultra-heavy snow storms.

**5.1.6** *Seismic loads*—Seismic loading conditions are presented in local building codes. In addition, detailed seismic load information is presented in ASCE 7 and ASCE 41. If the ability of the structure to resist seismic loads is of concern, the evaluation of the structure should also follow criteria contained in appropriate Building Seismic Safety Council and Federal Emergency Management Agency documents.

**5.1.7** *Thermal effects*—Where restraint exists, expansion and contraction of a concrete structure due to daily and seasonal variations in ambient temperature can cause significant forces in the structural elements. The engineer should consult local weather records or NOAA to determine the range of temperatures that the structure has experienced. Approximate data regarding seasonal temperature variations are available in the *PCI Design Handbook* (Prestressed/Precast Concrete Institute 2004).

If there is a sudden change in ambient temperature, large concrete sections will respond more slowly than smaller sections. Therefore, effects of rate of heat transfer in individual concrete elements can also be important. It may also be appropriate to consider the effect of absorption of radiant heat due to the reflective properties of concrete coatings exposed to direct sunlight.

Variations in the temperature within a building can influence the magnitude of thermal effect forces. Consider conditions such as areas of the building where heating or cooling is turned off at night, inadequately or overly insulated areas, and existence of cold rooms.

**5.1.8** *Creep and shrinkage*—The effects of long-term creep and shrinkage are important considerations for concrete elements (ACI 209R). Cracks or other distress can be caused by restrained shrinkage (ACI 224R). In a concrete structure, internal stresses result from restrained shrinkage and long-term creep of concrete elements. These stresses, when combined with other stresses produced by applied loading, prestressing forces, or restraint of deformations caused by prestressing forces, can be significant. An example of this effect is a reinforced concrete column under sustained loading where stresses in the embedded reinforcing steel can increase over time due to creep of the concrete. Another example occurs in unrestrained prestressed structures, such as pretensioned beams, where creep and shrinkage will reduce the tensile force in the prestressing steel over time. In restrained prestressed structures, such as post-tensioned floor systems, restrained shrinkage may result in significant tension stresses that counteract the initial concrete precompression due to post-tensioning. The complex mechanisms associated with creep and shrinkage often makes quantifying their effects with precision difficult. Guidance for estimating the effects of creep and shrinkage can be found in ACI 209R, ACI 224R, and the *PCI Design Handbook* (Prestressed/Precast Concrete Institute 2004).

**5.1.9** *Soil and hydrostatic pressure*—Significant loads can be imposed on a structure from soil and hydrostatic pressure. Soil densities and the lateral soil pressure vary significantly. It is often prudent to sample and establish actual soil densities and properties such as the internal angle of friction. Variations in water table and moisture content can result in large variations in the lateral pressure. Overall stability should be checked in structures that are built on a slope, due to unbalanced soil pressure. Hydrostatic uplift forces can occur due to elevated groundwater conditions, defects or failures in pressurized water piping, and at high flood elevations. Consider possible loads or damage caused by frost heaving of soil, soil shrinkage or swelling, differential soil settlement, and improper drainage. The loads imposed on the structure due to these conditions can be determined through collaboration with a geotechnical engineer.

**5.1.10** *Fire*—If the structure being evaluated has been exposed to fire, consider the effects of localized damage caused by the heat of the fire or by the firefighting efforts. Volume changes of concrete elements during a fire can cause significant damage. Restrained thermal expansion may lead to high internal stresses, which can result in concrete section loss by spalling and loss of bond. Additionally, concrete material properties, including compressive strength and elastic modulus, may deteriorate and can be permanently altered as a result of fire exposure. Potential damage to reinforcing steel or prestressing tendons should also be considered in the evaluation process. Additional information on damage due to fire is found in ACI 216.1. Petrographic analysis and in-place tests can be used to assess the extent of fire damage.

**5.1.11** *Loading combinations*—For purposes of analytical strength evaluation, load combinations should conform to the provisions of ACI 562. If serviceability is to be evaluated, load factors equal to 1.0 may be appropriate. Multiple load combinations may be necessary to fully assess the performance of the structure.

## 5.2—Selecting the proper method of evaluation

The evaluation method selected depends on factors such as the structural framing system, information known about its existing condition, and logistical and economic considerations. The typical choices are:

a) Evaluation solely by analysis

b) Evaluation by analysis and in-place load testing

**5.2.1** *Evaluation solely by analysis*—Evaluation solely by analysis is recommended if:

a) Sufficient information is available, or obtainable by field investigation, about the physical characteristics, material properties, and anticipated loadings and structural behavior

b) There is no evidence of latent defects

c) The number of unknowns is small

Analytical evaluation is appropriate if all the following conditions are satisfied:

a) There exists an accepted methodology for analyzing the type of structural system under consideration

b) Characteristics of the structural elements can be determined and modeled within acceptable limits of error

c) The distress is limited in magnitude or nature, so that the uncertainties introduced into the analysis do not render the application of the theory excessively difficult

d) Nonlinear behavior in materials and systems, if present under the loading conditions imposed, is adequately modeled. Examples of nonlinear behavior include concrete cracking, bond slip, and reinforcement yielding. Impact or blast loads can also induce nonlinear behavior.

**5.2.2** *Evaluation by analysis and in-place load testing*—Considerable experience has been assembled and reported on the subject of in-place load tests of existing structures. Refer to ACI 437.2, Tumialan et al. (2012), Ziehl et al. (2008), Galati et al. (2008), Nehil et al. (2007), Anderson and Popovic (1988), Barboni et al. (1997), Bares and FitzSimons (1975), Bungey (1989), Concrete Innovation Appraisal Service (2000), Elstner et al. (1987), FitzSimons and Longinow (1975), Fling et al. (1989), Guedelhoefer and Janney (1980), Hall and Tsai (1989), Ivanyi (1976), Kaminetzky (1991), Mettemeyer et al. (1999), Nanni and Gold (1998a,b), Nanni and Mettemeyer (2001), Nanni et al. (1998), Popovic et al. (1991), and Raths and Guedelhoefer (1980). Two load testing methods, cyclic and monotonic, are described in ACI 437.1R and ACI 437.2.

Evaluation by analysis and in-place load testing is recommended in the following cases:

a) The complexity of the design concept and lack of experience with the types of structural elements present make evaluation solely by analytical methods impractical or uncertain.

b) The loading and material characteristics of the structural element(s) cannot be readily determined.

c) The existing distress introduces significant uncertainties into the parameters necessary to perform an analytical evaluation.

d) The degree of suspected deficiencies in design, material, or construction cannot be readily determined.

e) If there is doubt concerning adequacy of structural elements for new loading that exceeds the allowable stresses calculated using the original design.

## CHAPTER 6—EVALUATION AND INTERPRETATION OF RESULTS

This chapter provides guidelines for performing and interpreting results of the evaluation. The evaluation should be designed with sufficient breadth and scope to allow meaningful conclusions to be developed regarding the suitability of the structure for its intended use. The evaluation may be performed solely by analytical methods or by a combination of analytical and in-place load testing methods.

Regardless of the method of evaluation, it is essential that the evaluation include all suspected defects detected in the preliminary investigation. More than one portion of the structure may need to be evaluated if multiple defects are suspected or if large areas of a structure are being evaluated. The following items should be considered for determining the extent of the evaluation:

a) Variations in the condition of the structure and material properties

b) Variation in type of structural framing systems

c) Differences in loading intensity required by intended use

d) Presence of other conditions that can affect load-carrying capacity, such as large floor openings or atypical bay sizes

Economic, schedule, and logistical considerations limit the number of specific members or the portions of the structure that can be evaluated in detail. Therefore, it is important to identify the specific critical members or portions of the structure in assessing the overall structural performance before undertaking the evaluation. Further information related to structural assessment, structural analysis, structural serviceability, and strength evaluation by load testing can be found in ACI 562-16 Sections 6.1 through 6.8.

### 6.1—Analytical evaluation

The information gathered from the preliminary investigation and material evaluations should be used in the analysis to determine the safe load-carrying capacity of the structure or portion of the structure being evaluated.

**6.1.1** *Forms of analysis*—In the evaluation of concrete structures by analytical methods, analysis has two different meanings. One deals with finding the values of forces and moments that exist in the structure. The second uses the characteristics of the structure or member to predict how it will respond to the existing load effects.

A structure should be analyzed to determine the bending moments, torsional moments, shear forces, and axial forces at the critical sections. Many engineers will limit this part of the analysis to linear-elastic response even though this is generally not realistic for reinforced concrete, particularly for loads in excess of service level.

The alternative, nonlinear analysis may require special capabilities not found in most engineering offices. An analysis done by elastic methods, however, often provides a reasonable estimate for the values of important load effects.

For nonlinear analysis, an assumption is made about the behavior of structures. For an evaluation of structural performance at service loads, it may be reasonable to assume that concrete and reinforcing steel behave in a linearly-elastic manner. It is necessary, however, to account for the low tensile strength of concrete, and cracked section properties are often used. Understanding the working stress properties of a structure can be valuable for assessing conditions between incremental stages of loading. A working stress analysis can be beneficial when relating observed conditions (such as cracking, deflection, or camber) to the actual state of stress in the structural components.

If structural safety is the principal concern, the strength of the member or structure needs to be established. The principles of strength design, as applied in ACI 318, provide a basis for establishing a nominal strength for structural members (ACI 562).

**6.1.2** *Levels of analysis*

**6.1.2.1** *Rigorous analysis*—Analysis based on experimentally verified theories of structural mechanics is useful under the following conditions:

a) Loading conditions for the structure are known with a high degree of certainty after examining existing data.

b) Detailed structural engineering drawings and material specifications are available, and are believed to be reliable or have been confirmed or supplemented with data obtained by the condition survey, for example:

i. Dimensions of the structure and its members can be determined by field measurements and are used to establish dead loads.

ii. The location, size, and depth of concrete cover of embedded reinforcing steel can be determined by field investigation.

iii. Material characteristics essential to the analysis can be determined, or estimated reasonably, by the use of invasive or nondestructive tests.

iv. Estimates of the strength of the foundations can be obtained by conducting appropriate geotechnical explorations and soil tests.

c) Sufficient data can be collected to make an adequate assessment of the existing physical condition of the structure, including estimation of the effects of distress, deterioration, and damage.

**6.1.2.2** *Finite-element analysis*—Linear finite element analysis and nonlinear finite element analysis provide a solution for cases where conventional methods of analysis are not sufficient. The latter method can be used to evaluate the effects of nonlinear material properties on structural response to levels of loading that produce inelastic behavior, such as concrete cracking, bond slip, and yielding of reinforcement. Nonlinear finite element analysis should be performed by experienced structural engineers competent in verifying and documenting the results.

**6.1.2.3** *Approximate analysis*—Use of approximate methods of analysis requires considerable experience with the type of structural system under evaluation and its behavior. Most importantly, approximate methods require the exercise of sound engineering judgment. Two basic guidelines should be followed:

1. All assumptions necessary for performing the structural analyses should be clearly documented. If inelastic models embedded in finite element analysis software are used, the models should be well understood, properly verified, and clearly documented. Care should be taken to describe those assumptions made in the strength evaluation by accounting for existing distress, deterioration, or damage.

2. All assumptions necessary to conduct the theoretical structural analysis should provide a conservative lower-bound value for the safe load-carrying capacity of the structure.

**6.1.3** *General considerations*—The assumed behavior of the structure and the results of the theoretical analyses should be consistent with the observed behavior of the structure. The analysis should model characteristics of the structure such as:

a) The effects of nonprismatic members on the relative stiffness of components in the structure

b) Torsional characteristics of structural members

c) Two-way load response in slab systems

d) Column support and structural fixities in terms of moment-rotation characteristics

e) Column base characteristics as influenced by soil conditions

**6.1.4** *Acceptance criteria*—The structure or structural component being evaluated is deemed to have sufficient strength if the analytical evaluation demonstrates that the predicted design capacity of the elements satisfies the requirements and the intent of ACI 562.

Uncertainty about the structure is reduced where field work has established the material strengths of steel and concrete; the size, location, and configuration of reinforcement; and structural dimensions. This supporting work can serve as justification for using a different strength-reduction factor for evaluation as opposed to design (ACI 562-16 Chapter 5), if allowed by the local building code. Experience and engineering judgment are important in this case.

If the analytical evaluation indicates that the structure does not satisfy the intent of ACI 562, the building official may approve a lower load rating for the structure based on the results of such evaluation if allowed by the local building code.

**6.1.5** *Findings of analytical evaluation*—An analytical strength evaluation has three possible findings:

1. Analyses show that the structure or structural element has an adequate margin of safety according to the provisions of the applicable building code. In this case, the design strength (nominal strength multiplied by strength-reduction factor) exceeds that required for factored loads.

2. Analyses show that the design strength 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.

3. Analyses show that 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.

## 6.2—Supplementing the analytical evaluation with load tests

**6.2.1** *Conditions for use*—In-place load testing is recommended if the following conditions are met:

a) The test results will permit rational interpretation of the structural strength of the element to be tested.

b) The influence of adjacent structural members, components, or entire structures can be accounted for during the load test and when evaluating the results of the tests. This influence includes full accounting of alternative load paths that are available in the structure.

c) The structure can be monitored adequately and safely by appropriate instrumentation to provide the necessary data to make an evaluation of the structural strength and, where appropriate, serviceability.
d) All participants in the test and all passersby are safe during setup and performance of the test.

An analysis should always be done before conducting a load test. This analysis can employ approximate methods. The analysis should be performed to allow for a reasonable prediction of the performance of the structure during the load test. Calculated deflections of concrete structural elements can, in many cases, be inaccurate. Care and engineering judgment are required when comparing calculated deflections with those measured during a load test. Reports are available to assist the engineer in calculating deflections of reinforced concrete structures (ACI 435R; ACI 435.8R).

ACI 423.4R describes the limitations of full-scale load testing when evaluating structures with unbonded post-tensioned tendons damaged by corrosion and highlights the need for caution related to load testing for structural systems with unbonded post-tensioned tendons. Further information on the corrosion protection of bonded and unbonded prestressing materials and prestressing system components is addressed in Chapter 8 of ACI 562-16.

**6.2.2** *Identifying the form of test to be conducted*—Evaluation of structural adequacy may be aided by one or both of the following forms of load testing:
a) Static tests
b) Dynamic tests, using special test procedures developed specifically for the characteristics of the structure to be tested; dynamic test procedures are beyond the scope of this report.

**6.2.3** *General requirements*—The following general requirements are applicable when conducting a load test:
a) A qualified licensed design professional, acceptable to the building official, should design and directly supervise the tests.
b) The structure should be loaded to adequately test the suspected source of weakness.
c) On environmentally exposed structures, load tests should be conducted at a time when the effects of temperature variations, wind, and sunlight on the structure and the monitoring devices are minimized; for example, early morning, late evening, or at night.
d) Load tests on exposed concrete structures should preferably be conducted at temperatures above 32°F (0°C).
e) On environmentally exposed structures, the environmental conditions, especially the ambient temperatures and wind, should be recorded at frequent intervals during the load test.

**6.2.4** *Test loads*—The following guidelines may be useful for selecting the type of test load or loading device in conducting a load test of a concrete structure:
a) If the test load is applied by using separate elements, such as iron bars, bricks, sandbags, or concrete blocks, the elements should be arranged throughout the duration of the test to prevent arching action. The largest base dimension of the separate elements or stacks of elements should be less than one-sixth of the span of the structural element being tested. These elements or stacks should be separated by a clear lateral distance of at least 4 in. (100 mm).
b) Separate pieces should be of uniform shape, and the weight of each piece should not differ by more than 5 percent from the average weight. The average weight should be determined by weighing at least 20 pieces taken at random.
c) If nonuniform loading elements are used, each element should be measured to determine surface contact area, weighed, and marked appropriately.
d) The load devices should be readily removable.
e) Materials that readily absorb moisture should not be used as loading elements.
f) Test load devices applied to sloping surfaces should be securely anchored to prevent shifting. Load components in all directions should be accounted for to prevent movements.
g) Water, loose sand, or other similar materials should be contained within small compartments to prevent ponding effects or shifting during significant deformation of the structure that may occur during the test.
h) If using hydraulic or pneumatic load-application systems, adequate supports should be provided to transfer the reactions, except where these reactions are part of the loading scheme. These loading devices should continue to function in a uniform fashion, even under significant deformation of the structure.

The total accumulated test load should be within 5 percent of the intended value. Arrangement of the test load should consider the following:
a) Care should be taken in the loading scheme not to unintentionally damage any other element of the structure that is not part of the test.
b) The test load should be arranged as close as possible to the load arrangement that the structure is intended to support.
c) If the test load cannot be arranged as described previously, it should be arranged to produce load effects similar to those that would be produced by the design load.
d) If uniform loads are approximated with concentrated loads, stress concentrations at the points of load application should not be significant.
e) The test load should be designed to produce the maximum load effect in the area being tested. This includes use of checkerboard or similar pattern loads, if required by the applicable building code.

**6.2.5** *Instrumentation*—The following guidelines are applicable to instrumentation systems for monitoring a load test:
a) Instrumentation should monitor deflections, lateral deformations, support rotations, and support settlement or shifting during application of the test load.
b) Measurement devices should be mounted to determine relative changes in the shape of the structure or structural element during the test.
c) During the load test, instrumentation should be protected from environmental influences such as direct sunlight, significant temperature variations, and wind.
d) Before the start of the load test, instrumentation should be installed to determine the effects of thermal changes on the deformations of the structure and on the instruments. If

necessary, compensation factors should be developed for application to the data obtained from the load test.

e) Strain measurements, if applicable, should be made at critical locations.

f) Deflection and strain measurement devices should be duplicated in critical areas.

g) Resolution of deflection measurement devices should not exceed 1/100 of the expected deflection (ACI 437.2).

h) Deflection of structural members can be measured with electronic or mechanical devices or with conventional surveying equipment. As an example, deflections can be measured using linear variable differential transformers (LVDTs).

i) Displacement transducers and resistance strain gauges can allow rapid electronic collection of data from numerous points when connected to a data acquisition system. Their installation, however, can be time-consuming, particularly if instrumentation has to be protected from the weather.

j) Inclinometers can be used to measure the rotation or slope of a test member. Because values of slope can be correlated to deflections, these instruments can be a good resource if displacement transducers are difficult to mount. Inclinometers can be mounted on a variety of vertical and horizontal surfaces.

k) Mechanical devices, such as dial gauges, are typically sturdy and simple to operate, but collection of data can be slow and often requires that someone enter the structure during performance of the test, which can be dangerous. These devices are valuable for measuring small deflections in stiff structures.

l) Large deflections can be measured easily by suspending graduated scales from critical points and reading them with a surveyor's level from a remote location.

m) Deflection measurement devices should be placed at the point(s) of maximum expected deflection. Devices should also be placed at the supports to detect column shortening, if deemed appropriate by the engineer.

n) Crack width can be measured by using graduated magnifying glasses or crack comparators. Their use during a load test is often restricted for safety reasons. If they are used, marks should be placed at each point on the cracks where readings are to be taken so that subsequent readings are taken at the same positions.

o) Crack movement (opening or closing) can be measured with dial gauges and displacement transducers. Crack movement can also be measured accurately by using gauge points and an extensometer.

p) Crack extension and the formation of new cracking can be tracked with acoustic emission sensors in combination with appropriate data acquisition systems and software (El Batanouny et al. 2014).

q) In deteriorated structures, cracks are often present either on the top or bottom surfaces of slabs and beams or the sides of columns. Some of these cracks may have meaning with respect to the structural behavior, while others may simply be the result of deterioration. For example, cracking caused by corrosion of embedded reinforcement may not directly relate to movement of structural elements during a load test. Engineering judgment should be exercised when monitoring and measuring crack movement, particularly if the structure contains numerous existing cracks or exhibits deterioration.

r) Thermometers or thermocouples should be used to measure the ambient temperature during a load test. Temperature readings should be taken in all areas of a structure that are affected by the load test. For structural slabs, thermometers should be placed above and below the slab surface. Records of variations of sunlight should be maintained for roof slabs and other areas of the structure that are exposed to direct sunlight during performance of a load test.

s) In a variety of shapes, sizes, and capacities, load cells are used to measure the load applied by hydraulic or pneumatic jacks. Pressure transducers can also be used to measure fluid pressures in the hydraulic system, which can be calibrated to a specific level of load.

t) Data acquisition systems can be used for simultaneously collecting readings from several devices as the load is being applied. Such devices include pressure transducers, load cells, LVDTs, inclinometers, extensometers, acoustic emission sensors, and strain gauges. This allows for real-time monitoring of the measured structural response. If acoustic emission sensors are used, high-speed data acquisition equipment is needed.

**6.2.6** *Shoring*—Shoring should be provided before a load test, whether the entire structure or only a portion is involved, to support the structure in case of failure during the test. The shoring should be designed to carry the existing dead load and all additional superimposed test loads on the portion of the structure for which collapse is possible. The effects of impact loading on the shoring, which is likely if a structure or member fails suddenly during the test, should be considered in the selection of shoring elements. Refer to the Commentary of ACI 562-16 Chapter 9 for guidance on shoring design. This may be accomplished by designing the shoring to support at least twice the total test load plus the existing dead load. Some engineers design the shoring to support at least twice the total test load plus the existing dead load. This approach should be evaluated for appropriateness on a case-by-case basis. Additional guidance on how to address shoring loads is available in ACI 562-16 Chapter 5.

Shoring more than one level to prevent progressive collapse in the event of failure should be considered. For example, if all floors below the test floor cannot support the weight of the test element, the loads it supports, and the imposed test loads, then the shoring should be extended to the foundation level. For horizontal members, shoring should clear the underside of the structure by not more than the maximum expected deflection plus an allowance not to exceed 2 in. (50 mm). Similar arrangements should be made for other types of members. Shoring should not influence or interfere with the free movements of the structure under the test load and should be designed and constructed to protect all people working on, below, or beside the structure to be tested in case of excessive deformation or collapse.

**6.2.7** *Static load tests of flexural members*—Static load testing of flexural members is addressed in ACI 437.2. Two loading protocols—monotonic and cyclic—are described.

The monotonic protocol involves the application of load for a period of 24 hours followed by a recovery period under no applied load. The cyclic protocol involves loading and unloading in incremental steps and generally requires less time to perform than the monotonic protocol. The cyclic protocol also provides the opportunity to better understand end fixity and load transfer characteristics of the tested component by comparing actual with calculated responses as the load test progresses. In addition to the information provided in ACI 437.2, the following guidelines are presented:

a) Shoring and instrumentation should be installed before any test load is applied. A series of base readings should be taken immediately before the application of the test load to serve as a datum for making measurements on the various elements of the structure during the load test.

b) No portion of the load that represents live loads should be applied before the deflections due to the dead load and superimposed dead load have effectively reached constant values.

c) After total dead load deflections have stabilized, existing cracks and other defects should be observed, marked, and recorded.

d) The licensed design professional should closely inspect the structure following application of each load increment for the formation or worsening of cracking and distress, as well as for the presence of excessive deformations or rotations. The licensed design professional should analyze the significance of any distress and determine whether it is safe to continue with the test.

e) If the cyclic protocol is followed, load-deflection curves should be developed for all critical points of deflection measurements during the load test. Electronic data-gathering and plotting equipment is available to automatically plot such curves. These curves should be closely monitored during the load test. They are a valuable tool in determining the load-deflection response of the structure and for determining if the structure is behaving elastically as the test load is increased. If the monotonic protocol is followed, deflection measurements should be recorded at each load increment, at the beginning and end of the 24-hour sustained load period, and at least 24 hours after the removal of the load. If the structure has met the acceptance criteria at the conclusion of the loading cycle, it is not necessary to wait the full 24-hour period to make the final set of response measurements.

**6.2.8** *Static load tests of elements in shear*—Load testing in direct shear to proof test the shear capacity of structural elements (such as beams, two-way flat plates and slabs, and corbels) is not recommended because only limited deflections are expected before failure, providing limited warning. A great deal of reliance is placed on the judgment of a licensed design professional conducting a load test for shear capacity. If possible, analysis of shear-flexure interaction, such as at a beam-column joint, is recommended. Each test is unique in terms of the characteristics of the structural elements being evaluated. Therefore, specific guidelines for conducting such tests cannot be listed as for load tests of flexural members. The following guidelines are presented for consideration by a licensed design professional who determines that a load test for evaluation of shear capacity should be conducted:

a) The structure should be thoroughly examined before and during the test. It is important to establish the concrete strength, aggregate type, and the shear reinforcement details as constructed. These parameters significantly impact the shear capacity of a structural element.

b) The load test should be preceded by a structural analysis to predict the performance of the structure.

c) Shoring of the structure is imperative. Provide shoring similar to that discussed for testing flexural members. Shoring should be designed for impact loading in the event of a sudden failure during the load test.

d) Instrumentation of the structure should concentrate on shear crack-width monitoring and shear crack extension in addition to deflections. Electronic instrumentation should be used to monitor crack widths to avoid the need for workers to read mechanical instruments during the load test.

e) The critical components of the structure should be monitored continuously during the test.

f) If load testing is planned for two-way slab systems, attention should be paid to the effect that the transfer of unbalanced moments has on punching shear at columns supporting unequal spans or unequally loaded spans, particularly for structures designed before modern code requirements.

g) If testing an element that is likely to fail in shear, but has significant flexural contribution (for example, the shear-flexure interaction at a beam-column joint), the loading to produce the expected failure should be calculated. Shear strength is typically, although not always, assessed at a distance $d$ away from a support. Load placed less than a distance $d$ away from a support may provide confinement and increase the shear capacity.

h) If minimum requirements are not specified by the code, acceptance criteria for the load test should be developed based on the judgment of a qualified licensed design professional with concurrence of the building official. Such acceptance criteria may be based on crack formation and movements at and along existing crack planes.

**6.2.9** *Interpretation of load test results*—Engineering judgment should be exercised in developing an appropriate interpretation of the results of a load test conducted on a concrete structure or elements within the structure. Sometimes, a concrete structure is believed to be deficient but passes a load test. This behavior can be the result of one or more of the following reasons:

a) The concrete structure has been designed conservatively. There are numerous reasons for a high degree of conservatism in reinforced concrete construction. These include the use of supplemental reinforcing steel placed arbitrarily in the structure to minimize cracking, using bar layouts with larger areas than required by calculation, use of conservative design theories, overestimation of dead loads, and inaccurate modeling of boundary and support conditions.

b) Actual concrete compressive strengths may exceed the specified design strengths.

c) The structural analyses do not accurately model the load-sharing characteristics of the structure.
d) Membrane forces may play a significant role in increasing the load capacity of reinforced and prestressed concrete slabs (Vecchio and Collins 1990).

## CHAPTER 7—REFERENCES

Committee documents are listed first by document number and year of publication followed by authored documents listed alphabetically.

*American Association of State Highway and Transportation Officials*

AASHTO T 260-11—Sampling and Testing for Total Chloride Ion Content in Concrete and Concrete Raw Materials

*American Concrete Institute*

ACI 201.1R-08—Guide for Conducting a Visual Inspection of Concrete in Service

ACI 207.3R-18—Report on Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions

ACI 209R-92(08)—Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

ACI 214.4R-10(16)—Guide for Obtaining Cores and Interpreting Compressive Strength Results

ACI 216.1-14—Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies

ACI 222R-01(10)—Protection of Metals in Concrete against Corrosion

ACI 222.2R-14—Report on Corrosion of Prestressing Steels

ACI 222.3R-11—Guide to Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures

ACI 224R-01(08)—Control of Cracking in Concrete Structures

ACI 224.1R-07—Causes, Evaluation, and Repair of Cracks in Concrete Structures

ACI 224.4R-13—Guide to Design Detailing to Mitigate Cracking

ACI 228.1R-19—Report on Methods for Estimating In-Place Concrete Strength

ACI 228.2R-13—Report on Nondestructive Test Methods for Evaluation of Concrete in Structures

ACI 309.2R-15—Guide to Identification and Control of Visible Effects of Consolidation on Formed Concrete Surfaces

ACI 318-14—Building Code Requirements for Structural Concrete

ACI 349.3R-18—Report on Evaluation and Repair of Existing Nuclear Safety-Related Concrete Structures

ACI 350-06—Code Requirements for Environmental Engineering Concrete Structures

ACI 362.2R-00(13)—Guide for Structural Maintenance of Parking Structures

ACI 364.1R-19—Guide for Evaluation of Concrete Structures Before Rehabilitation

ACI 365.1R-17—Report on Service-Life Prediction

ACI 376-11—Code Requirements for Design and Construction of Concrete Structures for the Containment of Refrigerated Liquified Gases and Commentary

ACI 423.4R-14—Report on Corrosion and Repair of Unbonded Single-Strand Tendons

ACI 423.8R-10—Report on Corrosion and Repair of Grouted Multistrand and Bar Tendon Systems

ACI 435R-95(00)—Control of Deflections in Concrete Structures

ACI 435.8R-85(97)—Observed Deflections of Reinforced Concrete Slab Systems, and Causes of Large Deflections

ACI 437.1R-07—Load Tests of Concrete Structures: Methods, Magnitude, Protocols, and Acceptance Criteria

ACI 437.2-13—Code Requirements for Load Testing of Existing Concrete Structures and Commentary

ACI 440.2R-17—Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures

ACI 562-16—Code Requirements for Assessment, Repair, and Rehabilitation of Existing Concrete Structures and Commentary

*American Society of Civil Engineers*

ASCE 7-16—Minimum Design Loads for Buildings and Other Structures

ASCE 41-17—Seismic Evaluation and Retrofit of Existing Buildings

*ASTM International*

ASTM A370-18—Standard Test Methods and Definitions for Mechanical Testing of Steel Products

ASTM C39/C39M-18—Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

ASTM C42/C42M-18—Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

ASTM C114-18—Standard Test Methods for Chemical Analysis of Hydraulic Cement

ASTM C295/C295M-18—Standard Guide for Petrographic Examination of Aggregates for Concrete

ASTM C457/C457M-16—Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

ASTM C597-16—Standard Test Method for Pulse Velocity through Concrete

ASTM C617/C617M-15—Standard Practice for Capping Cylindrical Concrete Specimens

ASTM C803/C803M-18—Standard Test Method for Penetration Resistance of Hardened Concrete

ASTM C805/C805M-18—Standard Test Method for Rebound Number of Hardened Concrete

ASTM C823/C823M-12(2017)—Standard Practice for Examination and Sampling of Hardened Concrete in Constructions

ASTM C856-18—Standard Practice for Petrographic Examination of Hardened Concrete

ASTM C876-15—Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel

ASTM C900-15—Standard Test Method for Pullout Strength of Hardened Concrete

ASTM C1152/C1152M-04(2012)—Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete

ASTM C1218/C1218M-17—Standard Test Method for Water-Soluble Chloride in Mortar and Concrete

ASTM C1231/C1231M-15—Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders

ASTM C1383-15—Standard Test Method for Measuring P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method

ASTM C1583/C1583M-13—Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)

ASTM C1723-16—Standard Guide for Examination of Hardened Concrete Using Scanning Electron Microscopy

ASTM C1740-16—Standard Practice for Evaluating the Condition of Concrete Plates Using the Impulse-Response Method

ASTM D4580/D4580M-12(2018)—Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding

ASTM D4748-10(2015)—Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar

ASTM D4788-03(2013)—Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography

ASTM D6087-08(2015)—Standard Test Method for Evaluating Asphalt-Covered Concrete Bridge Decks Using Ground Penetrating Radar

ASTM D7522/D7522M-15—Standard Test Method of Pull-Off Strength for FRP Laminate Systems Bonded to Concrete Substrate

ASTM E122-17—Standard Practice for Choice of Sample Size to Estimate a Measure of Quality for a Lot or Process

*International Concrete Repair Institute (ICRI)*

ICRI 210.3-13—Guide for Using In-Situ Tensile Pull-off Tests to Evaluate Bond of Concrete Surface Materials

ICRI 330.2-16—Guide Specification for Externally Bonded FRP Fabric Systems for Strengthening Concrete Structures

### Authored documents

Anderson, N., and Popovic, P., 1988, "Load Testing of Long Span Prestressed Single Tee Beams," *PCI Journal*, V. 33, No. 2, Mar.-Apr., pp. 42-51. doi: 10.15554/pcij.03011988.42.51

Barboni, M.; Benedetti, A.; and Nanni, A., 1997, "Carbon FRP Strengthening of Doubly Curved Precast PC Shell," *Journal of Composites for Construction*, V. 1, No. 4, Nov., pp. 168-174. doi: 10.1061/(ASCE)1090-0268(1997)1:4(168)

Bares, R., and FitzSimons, N., 1975, "Load Tests of Building Structures," *Journal of the Structural Division*, V. 101, May, pp. 1111-1123.

Bartlett, F. M., and MacGregor, J. G., 1994a, "Effect of Core Diameter on Concrete Core Strengths," *ACI Materials Journal*, V. 91, No. 5, Sept.-Oct., pp. 460-470. doi: 10.14359/4160

Bartlett, F. M., and MacGregor, J. G., 1994b, "Effect of Core Length-to-Diameter Ratio on Concrete Core Strengths," *ACI Materials Journal*, V. 91, No. 4, July-Aug., pp. 339-348. doi: 10.14359/4160

Bartlett, F. M., and MacGregor, J. G., 1994c, "Effect of Moisture Condition on Concrete Core Strengths," *ACI Materials Journal*, V. 91, No. 3, May-June, pp. 227-236. doi: 10.14359/4328

Bartlett, F. M., and MacGregor, J. G., 1995, "Equivalent Specified Concrete Strength from Core Test Data," *Concrete International*, V. 17, No. 5, May, pp. 52-58.

Bartlett, F. M., and MacGregor, J. G., 1996, "Statistical Analysis of the Compressive Strength of Concrete in Structures," *ACI Materials Journal*, V. 93, No. 2, Mar.-Apr., pp. 158-168. doi: 10.14359/1353

Bartlett, F. M., and MacGregor, J. G., 1997, "Precision of In-Place Concrete Strengths Predicted Using Core Strength Correction Factors Obtained by Weighted Regression Analysis," *Structural Safety*, V. 19, No. 4, pp. 397-410. doi: 10.1016/S0167-4730(97)00020-9

Bartlett, F. M., and MacGregor, J. G., 1999, "Variation of In-Place Concrete Strength in Structures," *ACI Materials Journal*, V. 96, No. 2, Mar.-Apr., pp. 261-269. doi: 10.14359/454

Bloem, D. L., 1965, "Concrete Strength Measurement—Cores vs. Cylinders," *Proceedings of ASTM*, V. 65, pp. 668-696.

Bloem, D. L., 1968, "Concrete Strength in Structures," *ACI Journal Proceedings*, V. 65, No. 3, Mar., pp. 176-187.

Bungey, J. H., 1979, "Determining Concrete Strength by Using Small-Diameter Cores," *Magazine of Concrete Research*, V. 31, No. 107, June, pp. 91-98. doi: 10.1680/macr.1979.31.107.91

Bungey, J. H., 1989, *The Testing of Concrete in Structures,* second edition, Chapman and Hall, New York, 228 pp.

Bungey, J. H., and Madandoust, R., 1992, "Factors Influencing Pull-off Tests on Concrete," *Magazine of Concrete Research*, V. 44, No. 158, Mar., pp. 21-30. doi: 10.1680/macr.1992.44.158.21

Bungey, J. H.; Millard, S. G.; and Grantham, M., 2006, *Testing of Concrete in Structures*, fourth edition, Taylor & Francis, 339 pp.

Bungey, J. H.; Millard, S. G.; and Shaw, M. R., 1994, "The Influence of Reinforcing Steel on Radar Surveys of Concrete Structures," *Construction & Building Materials*, V. 8, No. 2, pp. 119-126. doi: 10.1016/S0950-0618(09)90021-6

Carino, N. J., 2004a, "Pullout Test," *Handbook on Nondestructive Testing of Concrete*, second edition, V. M. Malhotra, and N. J. Carino, eds., CRC Press Inc., Boca Raton, FL.

Carino, N. J., 2004b, "Stress Wave Propagation Methods," *Handbook on Nondestructive Testing of Concrete*, V. M. Malhotra and N. J. Carino, eds., CRC Press, Boca Raton, FL.

Carino, N. J.; Sansalone, M.; and Hsu, N. N., 1986, "Flaw Detection in Concrete by Frequency Spectrum Analysis of Impact-Echo Waveforms," *International Advances in Nondestructive Testing*, W. J. McGonnagle, ed., Gordon & Breach Science Publishers, New York, pp. 117-146.

Chabowski, A. J., and Bryden-Smith, D., 1979, "A Simple Pullout Test to Assess the In-Situ Strength of Concrete," *Concrete International*, V. 1, No. 12, Dec., pp. 35-40.

Concrete Innovation Appraisal Service, 2000, "Guidelines for the Rapid Load Testing of Concrete Structural Members," *CIAS Report* 00-1, Farmington Hills, MI, 97 pp.

Concrete Reinforcing Steel Institute, 2014, *Vintage Steel Reinforcement in Concrete Structures*, Schaumburg, IL, 352 pp.

Davis, A., and Dunn, C., 1974, "From Theory to Field Experience with the Nondestructive Vibration Testing of Piles," *Proceedings – Institution of Civil Engineers*, V. 57, No. 4, Dec., pp. 571-593. doi: 10.1680/iicep.1974.3895

Davis, A. G.; Evans, J. G.; and Hertlein, B. H., 1997, "Nondestructive Evaluation of Concrete Radioactive Waste Tanks," *Journal of Performance of Constructed Facilities*, V. 11, No. 4, pp. 161-167. doi: 10.1061/(ASCE)0887-3828(1997)11:4(161)

Davis, A. G., and Hertlein, B. H., 1995, "Nondestructive Testing of Concrete Chimneys and Other Structures," *Proceedings of Conference on Nondestructive Evaluation of Aging Structures and Dams*, SPIE, V. 2456, pp. 129-136.

Dixon, D., 1987, "Analysis of Cover Meter Data," *Computer Use for Statistical Analysis of Concrete Test Data*, SP-101, P. Balaguru and V. Ramakrishnan, eds., American Concrete Institute, Farmington Hills, MI, pp. 47-64.

Domone, P. L., and Castro, P. F., 1987, "An Expanding Sleeve Test for In-Situ Concrete and Mortar Strength Evaluation," *Proceedings*, Structural Faults and Repairs 87, Engineering Technics Press, Edinburgh, UK.

El Batanouny, M. K.; Ziehl, P. H.; Larosche, A.; Mangual, J.; Matta, F.; and Nanni, A., 2014, "Acoustic Emission Monitoring for Assessment of Prestressed Concrete Beams," *Construction & Building Materials*, V. 58, pp. 46-53. doi: 10.1016/j.conbuildmat.2014.01.100

Elstner, R. C.; Gustafson, D. P.; Hanson, J. M.; and Rice, P. F., 1987, "Application of ACI 318 Load Test Requirements," *CRSI Professional Members' Bulletin* No. 16, Concrete Reinforcing Steel Institute, Schaumburg, IL, 11 pp.

Erlin, B., 1994, "Petrographic Examination," *Significance of Tests and Properties of Concrete and Concrete-Making Materials, Part III, Tests and Properties of Concrete Aggregates,* ASTM STP 169B, P. Klieger and J. F. Lamond, eds., ASTM International, West Conshohocken, PA, pp. 210-218.

Federal Communications Commission (FCC), 2002, "DA 02-1658 Revision of Part 15 of the Commission's Rules Regarding Ultra-Wideband Transmission Systems," Citation – 17 FCC Rcd 13522 (19), July.

FitzSimons, N., and Longinow, A., 1975, "Guidance for Load Tests of Buildings," *Journal of the Structural Division*, V. 101, July, pp. 1367-1380.

Fling, R. S.; McCrate, T. E.; and Doncaster, C. W., 1989, "Load Test Compared to Earlier Structure Failure," *Concrete International*, V. 18, No. 11, Nov., pp. 22-27.

Galati, N.; Nanni, A.; Tumialan, J. G.; and Ziehl, P. H., 2008, "In Situ Evaluation of Two Concrete Slab Systems I: Load Determination and Loading Procedure," *Journal of Performance of Constructed Facilities*, V. 22, No. 4, Aug., pp. 207-216. doi: 10.1061/(ASCE)0887-3828(2008)22:4(207)

Gaynor, R. D., 1965, "Effect of Horizontal Reinforcing Steel on the Strength of Molded Cylinders," *ACI Journal Proceedings*, V. 62, No. 7, July, pp. 837-840. doi: 10.14359/51685622

Guedelhoefer, O. C., and Janney, J. R., 1980, "Evaluation of Performance by Full Scale Testing," *Full-Scale Load Testing of Structures*, ASTM STP-702, W. R. Schriever, ed., ASTM International, West Conshohocken, PA, pp. 5-24.

Hall, W. B., and Tsai, M., 1989, "Load Testing, Structural Reliability and Test Evaluation," *Structural Safety*, V. 6, No. 2-4, pp. 285-302. doi: 10.1016/0167-4730(89)90028-3

Hanson, J. M., 2007, "Survey of Practice to Determine Strength of In Situ Concrete from Core Tests," *Journal of Performance of Constructed Facilities*, V. 21, No. 1, Feb., pp. 22-25. doi: 10.1061/(ASCE)0887-3828(2007)21:1(22)

Ivanyi, M., 1976, "Load Tests of Building Structures," *Journal of the Structural Division*, V. 102, June, pp. 1260-1261.

Jaeger, B. J.; Sansalone, M. J.; and Poston, R. W., 1996, "Detecting Voids in Grouted Tendon Ducts of Post-Tensioned Concrete Structures Using the Impact-Echo Method," *ACI Structural Journal*, V. 93, No. 4, July-Aug., pp. 462-472. doi: 10.14359/9705

Kaminetzky, D., 1991, *Design and Construction Failures Lessons from Forensic Investigations*, McGraw-Hill, 600 pp.

Kesner, K.; Poston, R. W.; Salmassian, K.; and Fulton, G. R., 1999, "Repair of Marina del Ray Seawall," *Concrete International*, V. 21, No. 12, Dec., pp. 43-50.

Lin, J. M., and Sansalone, M. J., 1996, "Impact-Echo Studies of Interfacial Bond Quality in Concrete: Part I—Effects of Unbonded Fraction of Area," *ACI Materials Journal*, V. 93, No. 3, May-June, pp. 223-232.

Mailhot, G.; Bisaillon, A.; Carette, G. G.; and Malhotra, V. M., 1979, "In-Place Concrete Strength: New Pullout Methods," *ACI Journal Proceedings*, V. 76, No. 12, Dec., pp. 1267-1282.

Mailvaganam, N. P., 1992, *Repair and Protection of Concrete Structures*, CRC Press, Boca Raton, FL, 473 pp.

Malhotra, V. M., 1976, "Testing Hardened Concrete: Non-Destructive Methods," *ACI Monograph* No. 9, American Concrete Institute, Farmington Hills, MI, 204 pp.

Malhotra, V. M., and Carino, N. J., eds., 2004, *Handbook on Nondestructive Testing of Concrete*, second edition, CRC Press Inc., Boca Raton, FL.

Mariscotti, M. A. J.; Jalinoos, F.; Frigerio, T.; Ruffolo, M.; and Thieberger, P., 2009, "Gamma-Ray Imaging for Void and Corrosion Assessment," *Concrete International*, V. 31, No. 11, Nov., pp. 48-53.

Mettemeyer, M.; Serra, P.; Wuerthele, M.; Schuster, G.; and Nanni, A., 1999, "Shear Load Testing of CFRP Strengthened Double-Tee Beams in Precast Parking Garage," *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, Fourth International Symposium, SP-188, C. W. Dolan, S. H. Rizkalla, and A. Nanni, eds., American Concrete Institute, Farmington Hills, MI, pp. 1063-1072.

Mielenz, R. C., 1994, "Petrographic Evaluation of Concrete Aggregates," *Significance of Tests and Properties of Concrete and Concrete-Making Materials, Part IV, Tests and Properties of Concrete Aggregates*, ASTM STP 169B, P. Klieger and J. F. Lamond, eds., ASTM International, West Conshohocken, PA, pp. 341-364.

Mindess, S., and Young, F., 1981, *Concrete,* Prentice Hall, Inc., 671 pp.

Nanni, A., and Gold, W. J., 1998a, "Evaluating CFRP Strengthening Systems In-Situ," *Concrete Repair Bulletin*, V. 11, No. 1, Jan.-Feb., pp. 12-14.

Nanni, A., and Gold, W. J., 1998b, "Strength Assessment of External FRP Reinforcement," *Concrete International*, V. 20, No. 6, June, pp. 39-42.

Nanni, A.; Gold, W. J.; Thomas, J.; and Vossoughi, H., 1998, "FRP Strengthening and On-Site Evaluation of a PC Slab," *Proceedings of the Second International Conference on Composites in Infrastructure (ICCI-98)*, V. 1, H. Saadatmanesh and M. R. Ehsani, eds., Tucson, AZ, pp. 202-212.

Nanni, A., and Mettemeyer, M., 2001, "Diagnostic Load Testing of a Two-Way Post-Tensioned Concrete Slab," *Practice Periodical on Structural Design and Construction*, V. 6, No. 2, May, pp. 73-82. doi: 10.1061/(ASCE)1084-0680(2001)6:2(73)

Nehil, T. E.; Nanni, A.; and Masetti, F., 2007, "Strength Evaluation by Load Testing," *Concrete International*, V. 29, No. 3, Mar., pp. 53-58.

Neville, A., 2001, "Core Tests: Easy to Perform, Not Easy to Interpret," *Concrete International*, V. 23, No. 11, Nov., pp. 59-68.

Petersen, C. G., 1997, "LOK-Test and CAPO-Test Pullout Testing, Twenty Years' Experience," *Proceedings of Conference on Non-Destructive Testing in Civil Engineering,* J. H. Bungey, ed., British Institute of Non-Destructive Testing, pp. 77-96.

Popovic, P.; Stork, P.; and Arnold, R., 1991, "Use of Hydraulics for Load Testing of Prestressed Concrete Inverted Tee Girder," *PCI Journal*, V. 36, No. 6, Sept.-Oct., pp. 72-78. doi: 10.15554/pcij.09011991.72.78

Powers, L. J., 2002, "Petrography as a Concrete Repair Tool," *Concrete Repair Bulletin*, V. 15, No. 1, pp. 22-25.

Prestressed/Precast Concrete Institute, 2004, *PCI Design Handbook,* sixth edition, Chicago, IL, 728 pp.

Raths, C. H., and Guedelhoefer, O. C., 1980, "Correlation of Load Testing with Design," *Full-Scale Load Testing of Structures,* ASTM STP 702, W. R. Schriever, ed., ASTM International, West Conshohocken, PA, pp. 91-113.

Sansalone, M., and Carino, N. J., 1986, "Impact Echo, A Method for Flaw Detection in Concrete Using Transient Stress Waves," *Report* No. NBSIR 86-3452, National Bureau of Standards, NTIS, Springfield, VA, 234 pp.

Sansalone, M., and Carino, N. J., 1988, "Impact-Echo Method: Detecting Honeycombing, the Depth of Surface-Opening Cracks, and Ungrouted Ducts," *Concrete International*, V. 10, No. 4, Apr., pp. 38-46.

Sansalone, M., and Carino, N. J., 1989, "Detecting Delaminations in Concrete Slabs with and without Overlays Using the Impact Echo Method," *ACI Materials Journal*, V. 86, No. 2, Mar.-Apr., pp. 175-184. doi: 10.14359/2350

Sansalone, M., and Streett, W. B., 1997, *Impact-Echo: Nondestructive Testing of Concrete and Masonr*y, Bullbrier Press, Jersey Shore, PA.

Snell, L. M.; Wallace, N.; and Rutledge, R. B., 1988, "Locating Reinforcement in Concrete," *Concrete International*, V. 10, No. 4, Apr., pp. 19-23.

Stone, W. C., and Carino, N. J., 1983, "Deformation and Failure in Large-Scale Pullout Tests," *ACI Journal Proceedings*, V. 80, No. 6, Nov.-Dec., pp. 501-513. doi: 10.14359/10871

Tam, C. T.; Lai, L. H.; and Lam, P. W., 1977, "Orthogonal Detection Technique for Determination of Size and Cover of Embedded Reinforcement," *Journal of Institution of Engineers (Malaysia)*, V. 22, June, pp. 6-16.

Tumialan, J. G.; Galati, N.; and Nanni, A., 2012, *Inspection, Testing, and Monitoring of Buildings and Bridges*, The National Council of Structural Engineers Associations (NCSEA), Chicago, IL.

Vecchio, F. J., and Collins, M. P., 1990, "Investigating the Collapse of a Warehouse," *Concrete International*, V. 12, No. 3, Mar., pp. 72-78.

Wen, Y. K., and Yeo, G. L., 2001, "Design Live Loads for Passenger Cars Parking Garages," *Journal of Structural Engineering*, V. 127, No. 3, pp. 280-289. doi: 10.1061/(ASCE)0733-9445(2001)127:3(280)

Williams, T. J.; Nozick, L. K.; Sansalone, M. J.; and Poston, R. W., 2006a, "Sampling Techniques for Evaluating Large Concrete Structures: Part I," *ACI Structural Journal*, V. 103, No. 3, May-June, pp. 399-408.

Williams, T. J.; Nozick, L. K.; Sansalone, M. J.; Poston, R. W.; and Kesner, K., 2006b, "Sampling Techniques for Evaluating Large Concrete Structures: Part II," *ACI Structural Journal*, V. 103, No. 4, July-Aug., pp. 505-512.

Wouters, J. P.; Peterson, J. E.; Kesner, K.; and Poston, R. W., 1999, "Impact-Echo Locates Subgrade Voids," *Concrete International*, V. 21, No. 5, May, pp. 23-27.

Zhan, R., and Xie, H., 2009, "GPR Measurement of the Diameter of Steel Bars in Concrete Specimens Based on the Stationary Wavelet Transform," *Insight—Non-Destructive Testing and Condition Monitoring*, V. 51, No. 3, pp. 151-155.

Ziehl, P. H.; Galati, N.; Nanni, A.; and Tumialan, J. G., 2008, "In-Situ Evaluation of Two Concrete Slab Systems II: Evaluation Criteria and Outcomes," *Journal of Performance of Constructed Facilities*, V. 22, No. 4, pp. 217-227. doi: 10.1061/(ASCE)0887-3828(2008)22:4(217)

American Concrete Institute
*Always advancing*

As ACI begins its second century of advancing concrete knowledge, its original chartered purpose remains “to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading knowledge.” In keeping with this purpose, ACI supports the following activities:

- Technical committees that produce consensus reports, guides, specifications, and codes.
- Spring and fall conventions to facilitate the work of its committees.
- Educational seminars that disseminate reliable information on concrete.
- Certification programs for personnel employed within the concrete industry.
- Student programs such as scholarships, internships, and competitions.
- Sponsoring and co-sponsoring international conferences and symposia.
- Formal coordination with several international concrete related societies.
- Periodicals: the ACI Structural Journal, Materials Journal, and Concrete International.

Benefits of membership include a subscription to Concrete International and to an ACI Journal. ACI members receive discounts of up to 40% on all ACI products and services, including documents, seminars and convention registration fees.

As a member of ACI, you join thousands of practitioners and professionals worldwide who share a commitment to maintain the highest industry standards for concrete technology, construction, and practices. In addition, ACI chapters provide opportunities for interaction of professionals and practitioners at a local level to discuss and share concrete knowledge and fellowship.

**American Concrete Institute**
**38800 Country Club Drive**
**Farmington Hills, MI 48331**
**Phone: +1.248.848.3700**
**Fax: +1.248.848.3701**

**www.concrete.org**

American Concrete Institute
*Always advancing*

38800 Country Club Drive
Farmington Hills, MI 48331 USA
+1.248.848.3700
www.concrete.org

The American Concrete Institute (ACI) is a leading authority and resource worldwide for the development and distribution of consensus-based standards and technical resources, educational programs, and certifications for individuals and organizations involved in concrete design, construction, and materials, who share a commitment to pursuing the best use of concrete.

Individuals interested in the activities of ACI are encouraged to explore the ACI website for membership opportunities, committee activities, and a wide variety of concrete resources. As a volunteer member-driven organization, ACI invites partnerships and welcomes all concrete professionals who wish to be part of a respected, connected, social group that provides an opportunity for professional growth, networking and enjoyment.

9 781641 950596