Strength Evaluation of Existing Concrete Buildings

Reported by ACI Committee 437

Antonio Nanni* Chair

Jeffrey S. West* Secretary

Tarek Alkhrdaji*	Azer Kehnemui	Stephen Pessiki
Joseph A. Amon	Andrew T. Krauklis	Predrag L. Popovic
Nicholas J. Carino*	Michael W. Lee*	Guillermo Ramirez*
Mary H. Darr	Daniel J. McCarthy	Andrew Scanlon
Mark William Fantozzi	Patrick R. McCormick	K. Nam Shiu
Paul E. Gaudette	Matthew A. Mettemeyer	Avanti C. Shroff
Zareh B. Gregorian	Thomas E. Nehil	Jay Thomas
Pawan R. Gupta	Renato Parretti*	Habib M. Zein Al-Abideen
Ashok M. Kakade	Brian J. Pashina	Paul H. Ziehl*
Dov Kaminetzky		

*Members of the committee who prepared this report.

The strength of existing concrete buildings and structures can be evaluated analytically or in conjunction with a load test. 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 tests investigation are described in detail. Analytical investigations should follow the principles of strength design outlined in ACI 318. Working stress analysis can supplement the analytical investigations by relating the actual state of stress in structural components to the observed conditions. Procedures for conducting static load tests and criteria indicated for deflection under load and recovery are recommended.

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

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CHAPTER 1—INTRODUCTION

1.1—Scope

This report provides recommendations to establish the loads that can be sustained safely by the structural elements of an existing concrete building. The procedures can be applied generally to other 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, and post-tensioned cast-in-place (concrete).

1.2—Applications

The procedures recommended in this report apply where strength evaluation of an existing concrete building is required in the following circumstances:

- Structures that show damage from excess or improper loading, explosions, vibrations, fire, or other causes;
- 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;
- Structures suspected to be substandard in design, detail, material, or construction;
- Structures where there is doubt as to the structural adequacy and the original design criteria are not known;
- Structures undergoing expansion or a change in use or occupancy and where the new design criteria exceed the original design criteria;
- Structures that require performance testing following remediation (repair or strengthening); and
- Structures that require testing by order of the building official before issuing a Certificate of Occupancy.

1.3—Exceptions

This report does not address the following conditions:

- Performance testing of structures with unusual design concepts;
- Product development testing where load tests are carried out for quality control or approval of mass-produced elements;
- Evaluation of foundations or soil conditions; and
- Structural engineering research.

1.4—Categories of evaluation

There are a number of different characteristics or levels of performance of an existing concrete structure that can be evaluated. These include:

- Stability of the entire structure;
- Stability of individual components of the structure;
- Strength and safety of individual structural elements;
- Stiffness of the entire structure;
- Durability of the structure;
- Stiffness of individual structural elements;
- Susceptibility of individual structural elements to excess long-term deformation;
- Dynamic response of individual structural elements;
- Fire resistance of the structure; and
- Serviceability of the structure.

This report deals with the evaluation of an existing concrete building for stability, strength, and safety. Although not intended to be an in-depth review of durability, this report addresses durability-related aspects so that the engineer is alerted to significant features that could compromise the structural performance of an existing concrete building or its components, either at the time of the investigation or over time.

1.5—Procedure for a structural evaluation

Most structural evaluations have a number of 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:

- Defining the existing condition of the building, including:
    1. Reviewing available information;
    2. Conducting a condition survey;
    3. Determining the cause and rate of progression of existing distress;
    4. Performing preliminary structural analysis; and
    5. Determining the degree of repair to precede the evaluation.
- Selecting the structural elements that require detailed evaluation;
- Assessing past, present, and future loading conditions to which the structure has and will be exposed under anticipated use;
- Conducting the evaluation;
- Evaluating the results; and
- Preparing a comprehensive report including description of procedure and findings of all previous steps.

1.6—Commentary

Engineering judgment is critical in the strength evaluation of reinforced concrete buildings. Judgment of qualified structural engineers may take precedence over compliance with code provisions or formulas for analyses that may not be applicable to the case studied. There is no such thing as an absolute measurement of structural safety in an existing concrete building, particularly in buildings that are deteriorated due to prolonged exposure to the environment or that have been damaged in a physical event, such as a fire. Similarly, there are no generally recognized criteria for evaluating serviceability of an existing concrete building. Engineering judgment and close consultation with the owner regarding the intended use of the building and expected level of performance are required in this type of evaluation.

The following conclusions regarding the integrity of a structure are possible as a result of a strength evaluation:

- The structure is adequate for intended use over its expected life if maintained properly;
- The structure, although adequate for intended use and existing conditions, may not remain so in the future due to deterioration of concrete or reinforcing materials, or changes are likely to occur that will invalidate the evaluation findings;
- The structure is inadequate for its intended use but may be adequate for alternative use;
- The structure is inadequate and needs remedial work;
- The structure is inadequate and beyond repair; and
- The information or data are not sufficient to reach a definitive conclusion.

1.7—Organization of the report

The remainder of this report is structured into the following five chapters and two appendixes:

Chapter 2 discusses what information should be gathered to perform a strength evaluation and how that information can be gathered. Two primary topics are covered. The first is a review of existing records on the building. The second is the condition survey of the building, including guidelines on the proper recognition of abnormalities in a concrete structure and survey methods available for evaluation of structural concrete.

Chapter 3 outlines procedures that should be used to assess the quality and mechanical properties of the concrete and reinforcing materials in the structure. Discussion is included on sampling techniques, petrographic, and chemical analyses of concrete, and test methods available to assess the mechanical properties of concrete and its reinforcement.

Chapter 4 provides procedures to assess the past, present, and future loading conditions of the structure or structural component in question. The second part of the chapter discusses how to select the proper method for evaluating the strength of an existing structure.

Chapter 5 provides commentary on the conduct of a strength evaluation for an existing concrete structure. Analytical techniques are discussed, and the use of load tests to supplement the analytical evaluation is considered.

Chapter 6 lists available references on the strength evaluation of existing concrete structures.

Appendix A describes an in-place load test method under development.

Appendix B briefly describes relevant documents for strength evaluation of existing structures.

CHAPTER 2—PRELIMINARY INVESTIGATION

This chapter describes the initial work that should be performed during a strength evaluation of an existing concrete building. The object of the preliminary investigation is to establish the structure's existing condition to obtain a reliable assessment of the available structural capacity. This requires estimating the concrete's condition and strength and the reinforcing steel's condition, location, yield strength, and area. Sources of information that should be reviewed before carrying out the condition survey are discussed. Available techniques for conducting a condition survey are described.

2.1—Review of existing information

To learn as much as possible about the structure, all sources of existing information concerning the design, construction, and service life of the building should be researched. 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 evaluated in a strength evaluation. The investigator needs to exercise judgment in determining which sources need to be consulted for the specific strength evaluation being conducted.

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

- Architectural, structural, mechanical, electrical, and plumbing contract drawings and specifications;
- Structural design calculations;
- Change orders to the original contract drawings and specifications;
- 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;
- Records of the local building department;
- Geotechnical investigation reports including anticipated structure settlements; and
- The structural design code referenced by the local code at the time of design.

2.1.2 *Construction materials*—Project documents should be checked to understand the type of materials that were specified and used for the building, including:

- Reports on the proportions and properties of the concrete mixtures, including information on the admixtures used, such as water-reducers and air-entraining agents with or without chlorides, and corrosion inhibitors;
- Reinforcing steel mill test reports;
- 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; and
- 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.

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

- Correspondence records of the design team, owner, general contractor, specialty subcontractors, and material suppliers and fabricators;
- Field inspection reports;

- Contractor and subcontractor daily records;
- Job progress photographs, films, and videos;
- Concrete cylinder compressive strength test reports;
- Field slump and air-content test reports;
- Delivery tickets from concrete trucks;
- As-built drawings;
- Survey notes and records;
- Reports filed by local building inspectors;
- Drawings and specifications kept in the trailers or offices of the contractor and the subcontractors during the construction period; and
- Records of accounting departments that may indicate materials used in construction.

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

2.1.5 *Service history of the building*—This includes all documents that define the history of the building such as:

- Records of current and former owners/occupants, their legal representatives, and their insurers;
- Maintenance records;
- Documents and records concerning previous repair and remodeling, including summaries of condition evaluations and reports associated with the changes made;
- Records maintained by owners of adjacent structures;
- Weather records;
- Logs of seismic activity and activity or records of other extreme weather events, such as hurricanes (where applicable); and
- Cadastral aerial photography.

2.2—Condition survey of the building

All areas of deterioration and distress in the structural elements of the building 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 of the steps outlined below may not be required in a particular strength evaluation. The engineer performing the evaluation decides what information will be needed to determine the existing condition of structural elements of the particular building that is being evaluated.

2.2.1 *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 building is essential for a successful strength evaluation. Additional information on the causes and evaluation of concrete structural distress is found in ACI 201.1R, ACI 207.3R, ACI 222R, ACI 222.2R, ACI 224R, ACI 224.1R, ACI 309.2R, ACI 362R, ACI 364.1R, and ACI 423.4R, as well as documents of other organizations such as the International Concrete Repair Institute (ICRI).

2.2.2 *Visual examination*—All 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 of the building. Liberal use of photographs, notes, and sketches to document this examination is recommended. Abnormalities should be recorded as to type, magnitude, location, and severity.

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

To employ the analytical method of strength evaluation, it is necessary to obtain accurate information on the member properties, dimensions, and positioning of the structural components in the building. 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.

2.2.3 *In-place tests for assessing the compressive strength of concrete*—A number of 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 (1991), and in Bungey and Millard (1996).

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.

Where in-place tests are used for estimating in-place compressive strength, a strength relationship that correlates compressive strength and the test measurement should be developed by testing core samples 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 representative samples of 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. Therefore, in-place testing can reduce the number of cores taken but cannot eliminate the need for drilling cores from the building.

When in-place tests are used only to compare relative concrete strength in different parts of the structure, however, 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 2.2.3.1 through 2.2.3.4 summarize a number of currently available in-place tests 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.

2.2.3.1 *Rebound number*—Procedures for conducting this test are given in ASTM C 805. 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 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.

The rebound distance depends on how much of the initial hammer energy is absorbed by the interaction of the plunger with the concrete. The greater the amount of absorbed energy, the lower the rebound number. 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 C 805 requires averaging 10 rebound readings for a test. Procedures for discarding abnormally high or low 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. A rebound number increases as the moisture content of concrete decreases, and tests on a dry surface will not correlate with interior concrete that is moist. The direction of the instrument (sideward, upward, downward) affects the rebound distance, so this should be considered when comparing readings and using correlation relationships. Manufacturers provide correction factors to account for varying hammer positions.

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 C 805 and ACI 228.1R should be considered when evaluating rebound number results.

2.2.3.2 *Probe penetration*—The procedures for this test method are given in ASTM C 803/C 803M.* 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 energy absorption 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 conic 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 into 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 larger probe 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 a 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 estimate compressive strength on the basis of probe penetration results.

2.2.3.3 *Pulse velocity*—The procedures for this method are given in ASTM C 597. The test equipment includes a transmitter, receiver, and electronic instrumentation. The test consists of measuring the time required for a pulse of ultrasonic 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 an acoustic couplant, such as petroleum jelly or vacuum grease. The pulse travels through the member and is detected by the receiving transducer, which is coupled to the opposite surface. Instrumentation measures and displays the pulse transit time. The distance between the transducers is divided

*The commercial test system for performing the test is known as the Windsor probe.

by the transit time to obtain the pulse velocity through the concrete under test.

The pulse velocity is proportional to the square root of the elastic modulus and inversely proportional to the mass density of the concrete. 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 produce 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 of the most critical of these 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 operator should take great care to understand these factors and ensure proper coupling to the concrete when using the pulse velocity to estimate concrete strength.

Under laboratory conditions, excellent correlations have been reported between velocity and compressive strength development for a given concrete. These findings, however, should not be interpreted to mean that highly reliable in-place strength predictions can be routinely made. Reasonable strength predictions are possible only if correlation relationships include those characteristics of the in-place concrete that have a bearing on pulse velocity. 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 (see Section 2.2.5.2).

2.2.3.4 *Pullout test*—The pullout test consists of measuring the load required to pull an embedded metal insert out of a concrete member (see 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). 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 least affected by details of the concrete proportions. The strength relationship, however, depends on aggregate density and maximum aggregate size.

ASTM C 900 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. 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.

2.2.4 *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. A variety of electromagnetic devices, known as covermeters, are used for this purpose. 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 is also capable of locating embedded metallic objects, but commercial systems cannot be used to estimate bar size. The following sections summarize these available tools. Additional information can be found in ACI 228.2R, Malhotra and Carino (1991), and Bungey and Millard (1996).

2.2.4.1 *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 flow resistance of magnetic flux in a material. 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 a magnetic field and a magnetic flux flowing between the ends of the yoke. The other coil senses the magnitude of the flux. When a steel bar is located within the path of the flux, the reluctance decreases and the magnetic flux increases. 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. Refer to ACI 228.2R for additional discussion of these types of meters. With proper calibration, 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. Dixon (1987) and Snell, Wallace, and Rutledge (1988) report additional details. 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 covermeter is based on the principle of eddy currents. This type of covermeter employs a probe that includes a coil excited by a high-frequency electrical current. The alternating current sets up an alternating magnetic field. When 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 instruments include probes for estimating bar size in addition to probes for estimating cover depth.

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. Covermeters are limited to detecting reinforcement located within about 6 in. (150 mm) of the exposed concrete surface. They are usually not effective in heavily reinforced sections, particularly sections with two or more adjacent bars 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 to verify that the device is operating correctly.

The accuracy of covermeters 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 when 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 covermeter surveys should be verified by drilling or chipping a selected area or areas as deemed necessary to confirm or calibrate the measured concrete cover and bar size (see Section 2.2.4.4).

2.2.4.2 *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 member's material. The quantity of radiation passing through the member is recorded on film similar to that used in medical applications. The length of exposure is determined by the film speed, strength of radiation, source to film distance, 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.

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 when 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 about 20 in. (0.5 m) thickness. Additional penetration depth up to about 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 can pose health hazards and should 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.

Results from radiographic tests should be verified by drilling or chipping selected areas as deemed necessary to confirm location of reinforcing steel.

2.2.4.3 *Ground-penetrating radar*—Pulsed radar systems (see Section 2.2.5.5) can be used to locate embedded reinforcement. This method offers advantages over magnetic methods as a result of its greater penetration. Access to one side of a member is all that is generally needed to perform an investigation. 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.

2.2.4.4 *Removal of concrete cover*—This method removes the concrete cover to locate and determine the size of embedded reinforcing steel, either by chipping or power drilling, to determine the depth of cover. These methods are used primarily for verification and calibration of the results of the nondestructive methods outlined above. Removal of concrete cover is the only reliable technique available to determine the condition of embedded reinforcing steel in deteriorated structures.

2.2.5 *Nondestructive tests for identifying internal abnormalities*—A strength evaluation may also determine if internal abnormalities exist that can reduce structural capacity, such as internal voids, cracks, or regions of inferior concrete quality. Compared with methods of strength determination, some techniques for locating internal defects require more complex instrumentation and specialized expertise to perform the tests and interpret the results. Refer to ACI 228.2R, Malhotra and Carino (1991), and Bungey and Millard (1996) for additional information.

2.2.5.1 *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 results when the surface over a hollow, delaminated, or thin area is struck, compared with a higher-frequency, ringing sound over undamaged and relatively thick concrete. For slabs, such areas can be detected by a heavy steel chain dragged over the concrete surface, unless the slab has a smooth, hard finish, in which case inadequate vibration is set up by the chains. 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 are found in ASTM D 4580.

2.2.5.2 *Pulse velocity*—The principle of pulse velocity is described in Section 2.2.3.3. 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 void between the transducers, the pulse travels through the concrete around the void. 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 nature of the abnormality. Cores are often taken to determine the nature of the indicated abnormality.

2.2.5.3 *Impact-echo method*—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 when 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 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.

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, Sansalone, and Hsu 1986). The frequency of that peak can be related to the depth of the reflecting interface by a simple relationship (Sansalone and Streett 1997). An ASTM test method has been developed for using the impact-echo method to measure the thickness of plate-like structures (ASTM C 1383).

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, Sansalone, and Poston 1996; Wouters et al. 1999; Lin and Sansalone 1996). The test provides information on the condition of the concrete 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 particular 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). No standardized test methods (ASTM) have been developed for internal defect detection using the impact-echo method.

2.2.5.4 *Impulse-response method*—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 structural vibration response of the portion of the structure surrounding the impact point (Davis, Evans, and Hertlein 1997). Measured response and the force history are used to calculate the impulse response spectrum of the structure (Sansalone and Carino 1991). 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. The impulse-response method reports on a larger volume of a structure than the impact-echo method but cannot define the exact location or depth of a hidden defect. As a result, it is often used in conjunction with impact-echo testing. An experienced engineer can extract several measures of structural response that can be used to compare responses at different test points (Davis and Dunn 1974; ACI 228.2R; Davis and Hertlein 1995).

2.2.5.5 *Ground-penetrating radar*—This method is similar in principle to the other echo techniques, except that electromagnetic energy is introduced into the material. An antenna placed on the concrete surface sends out an extremely short-duration radio frequency pulse. A portion of the pulse is reflected back to the antenna, which also acts as receiver, and the remainder penetrates into the concrete. If the concrete member contains boundaries between materials with different electrical properties, some of the pulse sent into the concrete is reflected back to the antenna. Knowing the velocity of the pulse in the concrete, the depth of the interface can be determined (ACI 228.2R). A digital recording system displays 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 individual sections. 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 when access is limited to one side. The penetrating ability of the electromagnetic pulse depends on the electrical conductivity of the material and the frequency of the radiation. As electrical conductivity increases, pulse penetration decreases. In testing concrete, a higher moisture content reduces pulse penetration.

There are two ASTM standards on the use of ground-penetrating radar, both of which have been developed for highway applications. ASTM D 4748 measures the thickness of bound pavement layers, and ASTM D 6087 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) has published rules (July 2002) that regulate the purchase and use of ground-penetrating radar equipment.

2.2.5.6 *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 detector to locate hot or cold spots on the surface. The locations of these hot and cold spots serve as indications of the locations of internal defects in the concrete. The technique has been successfully used to locate regions of delamination in concrete pavements and bridge decks (ASTM D 4788).

There must be heat flow through the member 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 must 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 that 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 apparent temperature of the surface. It is often useful to take a photographic or video record of the areas of the concrete surface being investigated by infrared photography. By comparing the two, surface defects can be eliminated from consideration as internal defects in the concrete.

2.2.5.7 *Radiography*—As discussed in Section 2.2.4.2, 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 the exposed film. Cracks and voids, on the other hand, absorb less radiation and show up as dark areas on the film. Crack planes parallel to the radiation direction are detected more readily than cracks perpendicular to the radiation direction.

CHAPTER 3—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 2. Sampling techniques, petrographic and chemical analyses of concrete, and test methods are discussed.

3.1—Concrete

The compressive strength of concrete is the most significant concrete property with regard 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, 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 concrete's condition and causes of deterioration may be obtained directly from petrographic and chemical analysis of the concrete.

3.1.1 *Guidelines on sampling concrete*—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 C 823. 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 2, 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 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.

For tests intended to measure the average value of a concrete property, such as strength, elastic modulus, or air content, the number of samples should be determined in accordance with ASTM E 122. The required number of samples generally depends on:

- The maximum allowable difference (or error) between the sample average and the true average;
- The variability of the test results; and

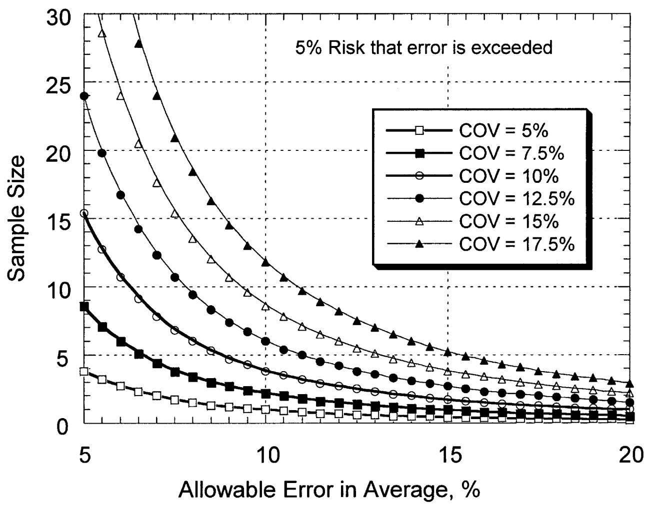

Fig. 3.1—Sample size based on ASTM E 122; risk = 5%.

- The acceptable risk that the maximum allowable difference is exceeded.

Figure 3.1 illustrates how ASTM E 122 can be used to determine the sample size. The vertical axis gives the number of samples needed as a function of the maximum allowable difference (as a percentage of the true average) and as a function of the coefficient of variation of the test results. In Fig. 3.1, the risk that the maximum allowable error will be exceeded is 5%, but other levels can be used. Because the variability of test results is usually not known in advance, an estimate should 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. Particular 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 decreases as its elevation within a placement increases (Bartlett and MacGregor 1999).

3.1.1.1 *Core sampling*—The procedures for removing concrete samples by core drilling are given in ASTM C 42/C 42M. The following guidelines are of particular importance in core sampling:

- *Equipment*—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.
- 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.
- *Core diameter*—Cores to be tested for a strength property should have a minimum diameter of at least twice, but preferably three times, the maximum nominal size of the coarse aggregate, or 3.75 in. (95 mm), whichever is greater. The use of small diameter cores results in lower and more erratic strengths (Bungey 1979; Bartlett and MacGregor 1994a).
- *Core length*—Where possible, cores to be tested for a strength property should have a length of twice their diameter.
- Embedded reinforcing steel should be avoided in a core to be tested for compressive strength.
- Avoid cutting electrical conduits or prestressing steel. Use covermeters (see Section 2.2.4) to locate embedded metal items before drilling.
- Where possible, core drilling should completely penetrate the concrete section to avoid having to break off the core to facilitate removal. If thorough-drilling is not feasible, the core should be drilled about 2 in. (50 mm) longer than required to allow for possible damage at the base of the core.
- Where 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 Section 3.1.1. 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.

3.1.1.2 *Random sampling of broken concrete*—Sampling of broken concrete generally should not be used where 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.

3.1.2 *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.

3.1.2.1 *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. 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 can provide some of the following information about the concrete:

- Density of the cement paste and color of the cement;
- Type of cement used;

- Proportion of unhydrated cement;
- Presence of pozzolans or slag cement;
- Volumetric proportions of aggregates, cement paste, and air voids;
- Homogeneity of the concrete;
- Presence and type of fibers (fiber reinforced concrete);
- Presence of foreign materials, including debris or organic materials;
- Aggregate shape, size distribution, and composition;
- Nature of interface between aggregates and cement paste;
- Extent to which aggregate particles are coated and the nature of the coating substance;
- Potential for deleterious reactions between the aggregate and cement alkalis, sulfates, and sulfides;
- Presence of unsound aggregates (fractured or porous);
- Air content and various dimensional characteristics of the air-void system, including entrained and entrapped air;
- Characteristics and distribution of voids;
- Occurrence of settlement and bleeding in fresh concrete;
- Degree of consolidation; and
- Presence of surface treatments.

Petrography can also provide information on the following items to aid in the determination of causes of concrete deterioration:

- Occurrence and distribution of fractures;
- Presence of contaminating substances;
- Surface-finish-related problems;
- Curing-related problems;
- Presence of deterioration caused by exposure to freezing and thawing;
- Presence of reaction products in cracks or around aggregates, indicating deleterious alkali-aggregate reactions;
- Presence of ettringite within cement paste (other than in pore system or voids) and in cracks indicating sulfate attack;
- Presence of corrosion products;
- Presence of deterioration due to abrasion or fire exposure; and
- Weathering patterns from surface-to-bottom.

The standard procedures for the petrographic examination of samples of hardened concrete are addressed by ASTM C 856. Procedures for a microscopical 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 C 457. ASTM C 295 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 Section 3.1.1 and following ASTM C 823. 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 (see Section 3.1.1).

The petrographer should be provided with information regarding the preconstruction, construction, and post-construction history and performance of the structure. Particular items of interest include:

- Original concrete mixture proportions, including information on chemical admixtures and slag cement;
- Concrete surface treatments or coatings;
- Curing conditions;
- Placement conditions, including concrete temperature, air temperature, ambient humidity, and wind conditions;
- Placement and finishing techniques;
- Location and orientation of core or sample in structure;
- Exposure conditions during service; and
- Description of distressed or deteriorated locations in structure, including photographs.

3.1.2.2 *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 gage 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 (ASR), depth of carbonation, and chloride content. To assess the risk of reinforcement corrosion, one of the more common uses of chemical testing is to measure the depth of carbonation and chloride concentration (corrosion mechanisms and factors for corrosion are discussed in detail in ACI 222R and ACI 222.2R).

Carbonation contributes to the risk of reinforcing steel corrosion by disrupting the passivity of the steel. More specifically, concrete carbonation occurs when its pH is reduced to approximately nine or less (ACI 222R). Chemical testing to determine the depth of carbonation can be accomplished by splitting a core lengthwise and applying a mixture of phenolphthalein indicator dye to the freshly fractured core surface. The indicator changes from colorless to a magenta color above a pH of nine. 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. Figure 3.2 shows the carbonation front on a concrete core as evidenced by the color variation. Any steel within this depth, denoted by the light color at the right end of the core, 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. When the chloride concentration reaches a threshold level at the reinforcement surface, corrosion of the

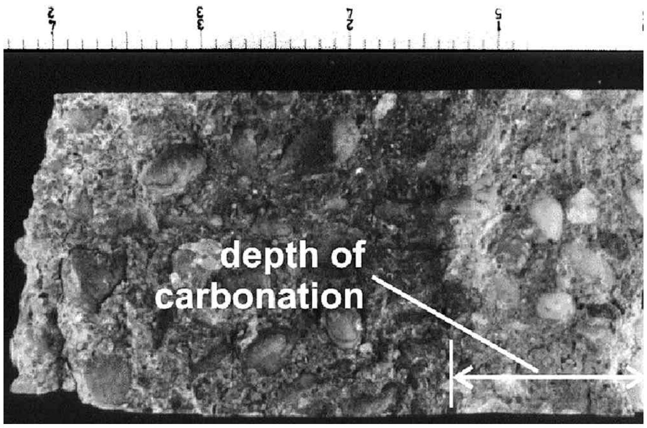

Fig. 3.2—Depth of carbonation as indicated by color change in phenolphthalein indicator.

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. When using cores, 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 C 1152/C 1152M, C 1218/C 1218M, and AASHTO T 260. 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 C 1218/C 1218M and C 1152/C 1152M, respectively. The chloride concentration is determined by potentiometric titration of the prepared sample with silver nitrate, as described in ASTM C 114. 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 inhibitors can be important when assessing the likely impact of chloride contamination on the anticipated performance of the structure.

3.1.3 *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 when used in conjunction with core testing.

3.1.3.1 *Testing cores*—Compressive strength of concrete cores taken from an existing structure should be determined in accordance with ASTM C 39/C 39M and ASTM C 42/C 42M. Key points in this procedure are:

- For core length-diameter ratios less than 1.75, apply the appropriate strength correction factors given in ASTM C 42/C 42M. These correction factors are approximate and engineering judgment should be exercised (Bartlett and MacGregor 1994b).
- Unless specified otherwise, cores should be tested in a moisture condition that is representative of the in-place concrete. Excessive moisture gradients in the cores will reduce the measured compressive strength (Bartlett and MacGregor 1994c). Care should be taken to avoid large variations in moisture resulting from drilling water, wetting during sawing or grinding of ends, and drying during storage. ASTM C 42/C 42M and ACI 318 provide guidance on moisture conditioning. Additional discussion is provided by Neville (2001). For core testing related to the strength evaluation of an existing concrete structure, careful consideration should be given to whether procedures for the moisture conditioning of cores should differ from those specified by ACI 318 and ASTM C 42/C 42M.
- Depending on age and strength level, compressive strength values obtained from core tests can either be lower or higher than those obtained from tests of standard 6 x 12 in. (150 x 300 mm) cylinders molded from samples of concrete taken during construction. For mature concrete, the core strength varies from 100% of the cylinder strength for 3000 psi (20 MPa) concrete to 70% for 9000 psi (60 MPa) concrete (Mindess and Young 1981). These are only generalizations, and rational procedures have been proposed for making more reliable estimates of the equivalent specified strength for use in structural capacity calculations on the basis of core strengths (Bartlett and MacGregor 1995).
- Care should be exercised in end preparation of cores before testing for compressive strength. When capping compound is used, its thickness is limited by ASTM C 617. This is especially critical for high-strength concrete.
- 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).
- The interpretation of core strengths is not a simple matter. Involved parties should agree on the evaluation criteria before sampling begins (Neville 2001).

3.1.3.2 *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 or nondestructive tests are commonly used in conjunction with tests of drilled cores to reduce the amount of coring required to estimate compressive strengths throughout the structure. Considerable care is required to establish valid estimates of

Table 3.1—Reinforcing bar specifications and properties: 1911 to present (CRSI 2001)

ASTM specification	Years		Steel type	Grade 33 (structural)		Grade 40 (intermediate)		Grade 50 (hard)		Grade 60		Grade 75	
	Start	End		Minimum yield, psi*	Maximum yield, psi	Minimum yield, psi	Maximum yield, psi	Minimum yield, psi	Maximum yield, psi	Minimum yield, psi	Maximum yield, psi	Minimum yield, psi	Maximum yield, psi
A 15	1911	1966	Billet	33,000	55,000	40,000	70,000	50,000	80,000	—	—	—	—
A 408	1957	1966	Billet	33,000	55,000	40,000	70,000	50,000	80,000	—	—	—	—
A 432	1959	1966	Billet	—	—	—	—	—	—	60,000	90,000	—	—
A 431	1959	1966	Billet	—	—	—	—	—	—	—	—	75,000	100,000
A 615	1968	1972	Billet	—	—	40,000	70,000	—	—	60,000	90,000	75,000	100,000
A 615	1974	1986	Billet	—	—	40,000	70,000	—	—	60,000	90,000	—	—
A 615	1987	Present	Billet	—	—	40,000	70,000	—	—	60,000	90,000	75,000	100,000
A 16	1913	1966	Rail	—	—	—	—	50,000	80,000	—	—	—	—
A 61	1963	1966	Rail	—	—	—	—	—	—	60,000	90,000	—	—
A 616	1968	1999	Rail	—	—	—	—	50,000	80,000	60,000	90,000	—	—
A 160	1936	1964	Axle	33,000	55,000	40,000	70,000	50,000	80,000	—	—	—	—
A 160	1965	1966	Axle	33,000	55,000	40,000	70,000	50,000	80,000	60,000	90,000	—	—
A 617	1968	1999	Axle	—	—	40,000	70,000	—	—	60,000	90,000	—	—
A 996	2000	Present	Rail, axle	—	—	40,000	70,000	50,000	80,000	60,000	90,000	—	—
A 706	1974	Present	Low-alloy	—	—	—	—	—	—	60,000	80,000	—	—
A 955M	1996	Present	Stainless	—	—	40,000	70,000	—	—	60,000	90,000	75,000	100,000

*1000 psi = 6.895 MPa.

compressive strength based on these indirect tests. See ACI 228.1R and Section 2.2.3 for further information.

3.2—Reinforcing steel

3.2.1 *Determination of yield strength*—The yield strength of the reinforcing steel can be established by two methods. Information from mill test reports furnished by the manufacturer of the reinforcing steel can be used if the engineer and the building official are in agreement. Yield strengths from mill test reports, however, tend to be greater than those obtained from tests of field samples. When mill test reports are not available nor desirable, sampling and destructive testing of specimens taken from the structure will be required. Guidelines for this method are given in Section 3.2.3.

The Concrete Reinforcing Steel Institute (CRSI) provides information on reinforcing systems in older structures (CRSI 1981). Information on reinforcing bar specifications, yield strengths, sizes, and allowable stresses is also provided by CRSI (2001). Table 3.1, adapted from the CRSI document, summarizes ASTM specifications and corresponding ranges of yield strength for bars manufactured from 1911 to present.

3.2.2 *Sampling techniques*—When the yield strength of embedded reinforcing steel is determined by testing, the recommendations listed below should be followed:

- Tension test specimens shall be the full section of the bar (ASTM A 370—Annex 9). Requirements for specimen length, preparation, testing, and determination of the yield strength are provided by ASTM A 370.
- In the event that bar samples meeting the length requirements of ASTM A 370 (Annex 9) cannot be obtained, samples may be prepared (machined) according to the general requirements of ASTM A 370 for testing and determination of mechanical properties.
- Samples should be removed at locations of minimum stress in the reinforcement.
- To avoid excessive reduction in member strength, no two samples should be removed from the same cross section (location) of a structural member.
- 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.
- 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).
- For longer spans or larger loaded areas, more samples should be taken from locations well distributed through the portion being investigated 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. Some discussion of extraction of unbonded single-strand tendons for testing can be found in ACI 423.4R.

3.2.3 *Additional considerations*—The strength evaluation of concrete structures can require consideration of several reinforcement-related factors in addition to the yield strength of the reinforcement, 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 and, as a result, the bond and development of these bars could be significantly different from those of modern reinforcement CRSI (2001). 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, CRSI (2001) conservatively recommends assuming that the required development length is twice that based on current code provisions. Concrete deterioration will also increase the development length of reinforcement.

Corrosion of reinforcement can lead to reduction in member capacity and ductility as a result of reinforcement section loss or disruption of bond. No guidelines are available for the assessment of reduced capacity due to corrosion damage. Because reinforcement corrosion normally results in disruption and cracking of the concrete surrounding the bar, bond to the concrete will also be negatively affected. 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 potentials (ASTM C 876) and polarization resistance. Refer to ACI 222R and ACI 228.2R for additional information on these types of tests.

CHAPTER 4—ASSESSMENT OF LOADING CONDITIONS AND SELECTION OF EVALUATION METHOD

4.1—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.

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

4.1.1.1 *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.

4.1.1.2 *Superimposed dead loads*—Superimposed dead loads include the weight of all construction materials incorporated into the building, exclusive of the self-weight of the structure. Examples include the weight of architectural floor and ceiling finishes, partitions, mechanical systems, and exterior cladding. The magnitude of superimposed dead loads can be estimated by performing a field survey of the building 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 building.

4.1.2 *Live loads*—The magnitude, location, and orientation of live loads on a structural component depend on the intended use of the building. Past, present, and future usage conditions should be established accurately so that appropriate assumptions can be made for the selection of live loads. Design live loads prescribed in the local building code should be used as the minimum live load in the evaluation. In the absence of specific requirements in the local building code, the live loads specified in ASCE 7 should be used.

When evaluating a structure for serviceability in addition to strength, estimate the live loads that will be present during normal conditions of occupancy of the building. Estimates of live loads can be obtained by performing detailed field surveys and measurements of loads in other buildings 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).

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

4.1.4 *Rain loads*—In evaluating roofs, consider 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.

4.1.5 *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, reference ASCE 7 and information available from NOAA.

4.1.6 *Seismic loads*—Seismic loading conditions are presented in local building codes. In addition, detailed seismic load information is presented in ASCE 7, and various documents published by the Building Seismic Safety Council (BSSC) and the Federal Emergency Management Agency (FEMA) under the National Earthquake Hazards Reduction Program (NEHRP). If ability of the structure to resist seismic loads is of concern, the evaluation of the structure should also follow criteria contained in appropriate BSSC and FEMA documents.

4.1.7 *Thermal effects*—Where restraint exists, expansion and contraction of a concrete building 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 1999).

Large concrete sections do not respond as quickly to sudden changes in ambient temperature as smaller sections. Therefore, effects of rate of heat gain and loss 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 any 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.

4.1.8 *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, can be significant. Examples of this effect are a reinforced concrete column under sustained loading where stresses in the embedded reinforcing steel can increase over time due to creep of the concrete or pretensioned structures. If creep or shrinkage effects are significant, appropriate measures should be taken in the evaluation process. The complex mechanisms associated with creep and shrinkage often make it difficult to quantify the effects (in terms of stress or load) with precision. Guidance for estimating the effects of creep and shrinkage can be found in ACI 209R, ACI 224R, and the *PCI Design Handbook*.

4.1.9 *Soil and hydrostatic pressure*—Significant loads can be imposed on a building 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 at the maximum design 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 should be determined by a geotechnical engineer.

4.1.10 *Fire*—If the building being evaluated has been exposed to fire, consider the effects of localized damage caused by the heat of the fire or by the fire-fighting efforts. Attention should be paid to the effect the heat of the fire had on the strength of the structure. Volume changes of concrete elements during a fire can cause significant damage. Potential damage to reinforcing steel or prestressing tendon should also be considered in the evaluation process. Additional information on damage due to fire is found in ACI 216R. Petrographic analysis and in-place tests can be used to assess the extent of fire damage.

4.1.11 *Loading combinations*—For purposes of strength evaluation, load factors and load combinations should conform to the provisions of ACI 318 and the local building code. If load factors and the corresponding strength reduction factors other than those of ACI 318 are used, the reserve strength of the structure resulting from the evaluation will be different than the reserve strength implied by ACI 318. Where serviceability is to be evaluated, load factors equal to 1.0 for all load cases are normally appropriate. Multiple load combinations are normally required to assess fully the performance of the structure.

Structural design philosophies, load factors, and load combinations have changed considerably over time. In many cases, the evaluation is being performed on a building that was designed to conform with a local building code or an ACI 318 code that has been superseded. Therefore, it may not be clear which edition of the local building code or ACI code is appropriate for the evaluation. As a general rule, if the objective of the evaluation is solely to determine the structural adequacy of a building for its intended use, the evaluation should be performed following the current code. If the objective of the evaluation is to determine whether a building was properly designed, then the evaluation should follow the code edition in effect at the time of the original design. Building codes often recognize that older buildings may not comply with the requirements of the current code. Most building codes include specific provisions to deal with older buildings. Engineers should consult with the local building official while planning the evaluation.

4.2—Selecting the proper method of evaluation

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

- Evaluation solely by analysis;
- Evaluation by analysis and in-place load testing; and
- Evaluation by analysis and small-scale model tests.

4.2.1 *Evaluation solely by analysis*—Evaluation solely by analysis is recommended where:

- Sufficient information is available, or obtainable by field investigation, about the physical characteristics, material properties, and anticipated loadings and structural behavior;
- Load testing is impractical or unsafe because of the needed load magnitude, complexity of the loads and testing arrangements required, or both; and
- Members are suspected of being susceptible to sudden failure. A load test in such a case would endanger the safety of the structure and those persons conducting the test. Failure by compression (such as columns or arches), shear, or anchorage is usually sudden.

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

- There exists an accepted methodology for analyzing the type of structural system under consideration. Information on analysis methods for reinforced concrete buildings may be found in ACI 318 and textbooks on structural analysis and reinforced concrete;
- Characteristics of the structural elements can be determined and modeled within acceptable limits of error;
- 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; and
- 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.

4.2.2 *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 318 (Chapter 20); Anderson and Popovic (1988); Barboni, Benedetti, and Nanni (1997); Bares and FitzSimons (1975); Bungey (1989); CIAS (2000); Elstner et al. (1987); FitzSimons and Longinow (1975); Fling, McCrate, and Doncaster (1989); Guedelhoefer and Janney (1980); Hall and Tsai (1989); Ivanyi (1976); Kaminetzky (1991); Mettemeyer et al. (1999); Nanni and Gold (1998a); Nanni and Gold (1998b); Nanni and Mettemeyer (2001); Nanni et al. (1998); Popovic, Stork, and Arnold (1991); and Raths and Guedelhoefer (1980); for further information.

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

- 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;
- The loading and material characteristics of the structural element(s) cannot be readily determined;
- The existing distress introduces significant uncertainties into the parameters necessary to perform an analytical evaluation;
- The degree of suspected deficiencies in design, material, or construction cannot be readily determined; and
- Where there is doubt concerning adequacy of structural elements for new loading that exceeds the allowable stresses calculated using the original design.

4.2.3 *Evaluation by analysis and small-scale models*—In the past, the construction and testing of small-scale models provided a feasible alternative to conducting a full-scale load test (ACI 444R; Harris 1980; and Sabnis et al. 1983). Modern computational techniques have essentially replaced load testing of small-scale models. Presently, load tests on small-scale models are rarely performed by practicing engineers and are used primarily in research environments, which are outside the scope of this report.

CHAPTER 5—EVALUATION

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 building 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 building may need to be evaluated if multiple defects are suspected or if large areas of building are being evaluated. Consider the following items in determining the extent of evaluation:

- Variations in the condition of the building and material properties;
- Variation in type of structural framing systems;
- Differences in loading intensity required by intended use; and
- 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 of the building before undertaking the evaluation.

5.1—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.

5.1.1 *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. Most engineers will conduct this part of the analysis assuming that individual members have linear-elastic behavior, even though this is not strictly true for reinforced concrete. The alternative, non-linear analysis, is not routinely feasible and requires special capabilities not found in most engineering offices. An analysis done by elastic methods, however, provides a reasonable estimate for the values of important load effects.

In the second form of 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 fact that concrete has a relatively low tensile strength, and cracked section properties are often used. Understanding the working stress properties of a structure undergoing a load test can be valuable when assessing conditions between incremental stages of loading. A working stress analysis, in accordance with Appendix A (Alternate Design Method) of ACI 318-99, can be beneficial when relating observed condition (such as cracking, deflection, or camber) to the actual state of stress in the structural components.

Where 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. The average concrete compressive strength obtained by testing concrete cores may be divided by 0.85 to arrive at the in-place concrete strength value to be used in strength calculations (Bloem 1968).

5.1.2 *Levels of analysis*

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

- Loading conditions for the building are known with a high degree of certainty after examining existing data;
- 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:
1. Dimensions of the structure and its members can be determined by field measurements and are used to establish dead loads;
2. The location, size, and depth of concrete cover of embedded reinforcing steel can be determined by field investigation;
3. Material characteristics essential to the analysis can be determined, or estimated reasonably, by the use of invasive or nondestructive tests; and
4. Estimates of the strength of the foundations can be obtained by conducting appropriate geotechnical explorations and soil tests; and

- 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.

5.1.2.2 *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.

5.1.2.3 *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:

- All assumptions necessary for performing the structural analyses should be clearly documented. Care should be taken to describe those assumptions made in the strength evaluation by accounting for existing distress, deterioration, or damage; and
- 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.

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

- The effects of non-prismatic members on the relative stiffness of components in the structure;
- Torsional characteristics of structural members;
- Two-way load response in slab systems;
- Column support and structural fixities in terms of moment-rotation characteristics; and
- Column base characteristics as influenced by soil conditions.

Modifications may be made to the results of the theoretical structural analyses to account for the anticipated future condition of the structure. These modifications should include any anticipated repairs and maintenance and any anticipated deterioration of the structure.

5.1.4 *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 318.

Uncertainty about the structure is clearly reduced where fieldwork has established the actual material strengths of steel and concrete; the size, location, and configuration of reinforcement; and identified member and structural dimensions. This supporting work can serve as justification for using a different strength-reduction factor ϕ for evaluation, as opposed to design. Suggested values of ϕ for evaluating structures for which uncertainty has been clearly reduced are reported in Section 20.2.5 of ACI 318-02. Experience and engineering judgment are important in this case.

When the analytical evaluation indicates that the structure does not satisfy the intent of ACI 318, the building official may approve a lower load rating for the structure based on the results of such evaluation.

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

- Analyses show that the building 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 ϕ) exceeds that required for factored loads;
- 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 building 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. In cases where the structure is only 5% or less under strength, engineering judgment of the particular circumstances may indicate that the structure can be used without further restriction; and
- 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.

5.2—Supplementing the analytical evaluation with load tests

5.2.1 *Conditions for use*—In-place load testing is recommended only if all of the following conditions are met:

- The test results will permit rational interpretation of the structural strength of the element to be tested;
- The influence of adjacent structural members, components, or whole structures can be accounted for during the load test and when evaluating the results of the tests. This influence includes full accounting of alternate load paths that are available in the building;
- 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
- 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. Theoretical calculations for predicting 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 435.7R; ACI 435.8R).

ACI 423.4R describes the limitations of full-scale load testing when evaluating structures with unbonded post-tensioning tendons damaged by corrosion. The following caution statement is provided:

> "Load testing of slabs and beams in accordance with ACI 318 under "Strength Evaluation of Existing Structures" provides no detailed information about the condition of the individual tendons. A significant number of tendons could have failed without being detected by a load test. The period of time subsequent to the testing for which the test results are valid is subject to a great deal of uncertainty, and ACI 318 provides no guidance for estimating the remaining service life. Also, load testing is expensive and disruptive. Therefore, although a load test can confirm that the tested part of the structure has adequate strength at the time of the test, this method has limitations when applied to structures with suggested or known corrosion damage to unbonded tendons."

5.2.2 *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:
- Static tests; and
- Dynamic tests, using special test procedures developed specifically for the characteristics of the structure to be tested. Such procedures are beyond the scope of this report.

5.2.3 *General requirements*—The following general requirements are applicable when conducting a load test:
- A qualified engineer (called "supervising engineer" in this document), acceptable to the building official, should design and directly supervise the tests;
- The portion of the structure to be tested should be at least 56 days old. Earlier testing may be permitted if mutually acceptable to all parties involved. In such cases, it is important to consider carefully the age of the concrete as it relates to the strength of concrete;
- The structure should be loaded to adequately test the suspected source of weakness;
- 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;
- Load tests on exposed concrete structures should preferably be conducted at temperatures above 32 °F (0 °C); and
- On environmentally exposed structures, the environmental conditions, especially the ambient temperatures and wind, should be recorded at frequent intervals during the load test.

5.2.4 *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:
- When the test load is applied by using separate elements, such as iron bars, bricks, sandbags, or concrete block, 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);
- Separate pieces should be of uniform shape, and the weight of each piece should not differ by more than 5% from the average weight. The average weight should be determined by weighing at least 20 pieces taken at random;
- If nonuniform loading elements are used, each element should be measured to determine surface contact area, weighed, and marked appropriately;
- The loading elements should be easily weighed;
- The load devices should be easy to apply and readily removable;
- Materials that readily absorb moisture should not be used as loading elements;
- Securely anchor test load devices applied to sloping surfaces to prevent shifting. Load components in all directions should be accounted for to prevent movements;
- 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; and
- When using hydraulic or pneumatic load-application systems, provide adequate supports to transfer the reactions, except where these reactions are part of the loading scheme. Ensure that these loading devices continue to function in a uniform fashion, even under significant deformation of the structure.

The total accumulated test load should be within 5% of the intended value. Arrangement of the test load should consider the following:
- Extreme 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. For example, care should be taken not to excessively unstress a prestressed beam by reacting against it in a manner that creates tension where there is no mild reinforcement;
- The test load should be arranged as close as possible to the arrangement of the load for which the structure was designed;
- If the test load cannot be arranged as described above, it should be arranged to produce load effects in the structure similar to those that would be produced by the design load;

- If uniform design loads are approximated with converging (concentrated) loads, stress concentrations at the points of load application should not be significant; and
- Design the test load to produce the maximum load effect in the area being tested. This includes use of checkerboard or similar type pattern loads, if required by the applicable building code.

5.2.5 *Instrumentation*—The following guidelines are applicable to installation of instrumentation systems for monitoring a load test.

- Instrumentation should monitor deflections, lateral deformations, support rotations, and support settlement or shifting during application of the test load;
- Measurement devices should be mounted to determine relative changes in the shape of the structure or structural element during the test;
- During the load test, instrumentation should be protected from environmental influences such as direct sunlight, significant temperature variations, and wind;
- 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 themselves. If necessary, compensation factors can be developed for application to the data obtained from the load test;
- On flexural members, strain measurements should be made at critical locations;
- Deflection and strain measurement devices should be duplicated in critical areas;
- The acceptable error in instruments used for measuring displacements should not exceed 5% of the calculated theoretical deformation but not more than 0.005 in. (0.13 mm);
- 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);
- Displacement transducers and resistance strain gages are available and can allow rapid electronic collection of data from a large number of points when connected to a data acquisition system. Their installation, however, can be time consuming and costly, particularly on sites exposed to the weather;
- Inclinometers are 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 when displacement transducers are difficult to mount. Inclinometers can be mounted on a variety of vertical and horizontal surfaces;
- Mechanical devices, such as dial gages, are typically sturdy and simple to operate, but collection of data can be slow and often requires that someone enter into the structure during performance of the test, which can be dangerous. These devices are valuable for measuring small deflections in stiff structures;
- Large deflections can be easily measured by suspending graduated scales from critical points and reading them with a surveyor's level from a remote location;
- Deflection measurement devices could be placed at the point(s) of maximum expected deflection. Devices could also be placed at the supports to detect column shortening, if deemed appropriate by the engineer;
- 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;
- Crack movement (opening or closing) can be measured with dial gages and displacement transducers. Crack movement can also be measured accurately by using gage points and an extensometer;
- In deteriorated structures, cracks are present either on the top or bottom surfaces of the slabs and beams or the sides of columns. Some of these cracks may have meaning with respect to the structural behavior, while others are simply 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 when the structure contains numerous existing cracks or exhibits deterioration;
- 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;
- 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; and
- Data acquisition systems can be used for simultaneously collecting readings from several devices as the load is being applied. Such devices include pressure transformers, load cells, LVDTs, inclinometers, extensometers, and strain gages. This allows for real-time monitoring of the measured structural response.

Some of the instrumentation used to monitor the behavior of a test member is summarized in Table 5.1. This table includes the common names for the devices, some of their suggested uses, recommended minimum measurable values, and measuring ranges. Additional information on these devices as well as many others is available in literature (Russell 1980; Bungey 1989; Carr 1993; and Fraden 1993).

5.2.6 *Shoring*—Shoring should be provided before a load test, whether the whole 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

Table 5.1—Summary of typical instrumentation used in in-place load testing

Parameter	Devices	Suggested minimum measurable value	Measuring range
Deflection	LVDT	0.0001 in.	±2 in.
	Dial gage	0.001 in.	±3 in.
Rotation	Inclinometer	0.001 degree	±3 degrees
Strain	Strain gage	5 με	±3000 με
	Extensometer	50 με	±10,000 με
	LVDT	50 με	±10,000 με
Crack width	Extensometer	0.0001 in.	±0.2 in.
Load	Load cell	10 lb	0 to 200,000 lb
	Pressure transducer	100 lb	0 to 200,000 lb

Note: 1 in. = 25.4 mm; 1 lb = 4.45 N.

portion of the building for which collapse is possible. The effects of impact loading on the shoring, which is likely if a structure or member fails during the test, should be considered in the selection of shoring elements. This may be accomplished by designing the shoring to support at least twice the total test load plus the existing dead load.

In multistory structures, consider shoring more than one level to prevent progressive collapse in the event of failure. 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. In any case, 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.

5.2.7 *Static load tests of flexural members*

5.2.7.1 *Guidelines*—The following guidelines are presented for conducting static load tests of flexural members:

- Install shoring and instruments before any test load is applied. Take a series of base elevation readings immediately before the application of the test load to serve as a datum for making deflection readings on the various elements of the structure during the load test;
- No portion of the test 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;
- After total dead load deflections have stabilized, existing cracks and other defects should be observed, marked, and recorded;
- The total test load TL should be defined as the load that produces a load effect (bending moment, shear force, or axial force, as appropriate) not less than $0.85(1.4D + 1.7L)$ at the section being studied, where D and L are dead and live loads, respectively. Elstner et al. (1987) discuss the determination of test loads in reinforced concrete construction. The determination of the test load should include live load reductions as permitted by the applicable building code;
- The test load should be applied in the predetermined pattern in at least four approximately equal increments. If serviceability is a criterion in the evaluation of the structure, an intermediate load increment equivalent to $1.0D + 1.0L$ should be included so that the service behavior of the structure can be evaluated. The test loads should be applied without impact and without causing vibration of the structure. After applying each increment of the test load, deflection and crack measurements should be made at equal time intervals until the deflections attain effectively constant values. For this purpose, the structure may be considered stabilized when the change between successive deflection readings does not exceed 10% of the initial total deflection recorded for the current load increment. If at any time during the test the measured deflections reach or exceed precalculated values, the test should be stopped and only be continued with the written permission of the supervising engineer;
- The supervising engineer 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 supervising engineer should analyze the significance of any distress and determine whether it is safe to continue with the test;
- Load-deflection curves should be developed for all critical points of deflection measurements during the load test. Various electronic data-gathering and plotting equipment are 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 total test load is approached; and
- After the total test load has been in position for 24 h, deflection readings should be taken. The load should then be removed in decrements no greater than twice the increments used to apply the test load. Deflection readings should be taken before and after each load decrement has been removed. Final deflection readings should be taken 24 h after removal of the entire superimposed test load.

5.2.7.2 *Criteria for evaluation of the 24 h static load test*—The procedures and criteria for interpreting the data should be completely established before a load test is conducted. The general procedure required by ACI 318 for 24 h load testing involves gradually applying the test load until the total test load is reached and maintained for 24 h. Measurements should be taken before any test load is applied, at the point of full loading, after 24 h of constant loading, and 24 h subsequent to the removal of the test load. If structural safety is the only criterion for the evaluation of the structure, and if the structure under the test load does not show visible evidence of failure, it passed the test if it meets one the following criteria given in ACI 318:

1) If the measured maximum deflection Δ_{max} of a beam, floor, roof, or slab is less than $l_t^2/(20,000h)$, where l_t = span of the member (in.) under load test, and h = the total depth of the member (in.). The span of a member is the distance between centers of supports or clear distance between the supports plus the depth of the member, whichever is smaller. In determining limiting deflection for a cantilever, l_t should be taken as twice the distance from the support to the end, and the deflection should be adjusted for movement of the support; or

2) If the measured residual deflection Δ_{rmax} of a beam, floor, roof, or slab is less than $\Delta_{max}/4$.

Note: "Visible evidence of failure" includes cracking, spalling, crushing, deflections, or rotations of such a magnitude and extent that it is deemed excessive in the opinion of a qualified engineer and not compatible with the safety requirements for the structure.

If the measured maximum and residual deflections do not meet requirements indicated in 1) or 2) above, the load test may be repeated. The repeat test should be conducted no earlier than 72 h after removal of the first test load. The portion of the structure tested in the repeated test shall be acceptable if deflection recovery satisfies the condition

$$\Delta_{r\ max} \leq \frac{\Delta_{f\ max}}{5} \tag{5-1}$$

where $\Delta_{f max}$ is the maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test.

If serviceability is the criterion, the deflections caused by the test load corresponding to $1.0D + 1.0L$ should not exceed that stipulated before the test. The significance of any cracks should be considered by a qualified engineer.

If the structure fails during the load test on the basis of the deflection criterion but shows no evidence of structural or material failure, either all necessary repairs or changes should be made to make the structure adequate for the rated capacity, or a lower rating should be established. No retesting of a structure, or any portion thereof that has previously failed a load test, should be permitted, unless appropriate structural repairs and strengthening are employed to upgrade the structure.

5.2.8 *Static load tests of elements in shear*—Load test to evaluate the shear capacity of structural elements is not addressed in ACI 318, and it should not be recommended except under unusual circumstances. This recommendation is due to the uncertainty associated with the brittle and sudden characteristics of shear failures. A great deal of reliance is placed on the judgment of a qualified engineer conducting a load test for shear capacity. Each test is unique in terms of the characteristics of the structural elements being evaluated. Therefore, specific guidelines for conducting such tests cannot be simply listed as for load tests of flexural members. The following guidelines are presented for consideration by a qualified engineer who determines that a load test for evaluation of shear capacity needs to be conducted:

- 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, as these parameters greatly impact the shear capacity of a structural element;
- The test load should not be less than $0.85(1.4D + 1.7L)$;
- The load test should be preceded by a structural analysis to predict with more accuracy the performance of the structure;
- Shoring of the structure is imperative. Provide shoring similar to that discussed for testing flexural members;
- Instrumentation of the structure should concentrate on shear crack-width monitoring in addition to deflections. Installation of instrumentation to monitor crack widths is potentially dangerous to the workers and should be avoided while load is applied to a member;
- The critical components of the structure should be monitored continuously during the test;
- When 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; and
- When minimum requirements are not specified by the code, acceptance criteria for the load test should be developed based on the judgment of a qualified engineer with concurrence of the building official. Such acceptance criteria may be based on crack formation and movements at and along existing crack planes.

5.2.9 *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 building or elements within the building. 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:

- The concrete structure has been designed conservatively. There are a number of 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, providing larger areas of reinforcement than required by calculation when selecting bars, use of conservative design theories, overestimation of dead loads, and inaccurate modeling of boundary and support conditions;
- Actual concrete compressive strengths may exceed the specified design strengths;
- The structural analyses do not accurately model the load-sharing characteristics of the structure; and
- Membrane forces can often play a significant role in increasing the load capacity of reinforced and prestressed concrete slabs (Vecchio and Collins 1990).

5.3—Research needs

The engineering community is constantly seeking new methods to effectively load test and monitor structures. Among the load test methods, cyclic loading offers significant advantages in terms of reliability and economy, as compared with the traditional 24 h static loading. The principles of load cycling allow for the determination of linearity of response as well as repeatability and permanency of deformation. More details on this procedure and a suggested protocol are offered in Appendix A.

With respect to monitoring equipment, there is great interest in utilizing non-contact type devices that have been primarily developed for surveying application. New optical technologies allow for accurate and remote sensors with the possibility of continuous monitoring.

CHAPTER 6—REFERENCES

6.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Association of State Highway and Transportation Officials (AASHTO)

T 260 Sampling and Testing for Total Chloride Ion Content in Concrete and Concrete Raw Materials

ACI International

201.1R Guide for Making a Condition Survey of Concrete in Service

207.3R Guide for Evaluation of Concrete in Existing Massive Structures for Service Conditions

209R Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

216R Guide for Determining the Fire Endurance of Concrete Elements

222R Corrosion of Metals in Concrete

222.2R Corrosion of Prestressing Steels

224R Control of Cracking in Concrete Structures

224.1R Causes, Evaluation, and Repair of Cracks in Concrete Structures

228.1R In-Place Methods to Estimate Concrete Strength

228.2R Nondestructive Test Methods for Evaluation of Concrete in Structures

309.2R Identification and Control of Visible Effects of Consolidation on Formed Concrete Surfaces

318 Building Code Requirements for Structural Concrete

362R State-of-the-Art Report on Parking Structures

364.1R Guide to Evaluation of Concrete Structures Prior to Rehabilitation

365.1R Service-Life Prediction—State-of-the-Art Report

423.4R Corrosion and Repair of Unbonded Single-Strand Tendons

435R Control of Deflections in Concrete Structures

435.7R State-of-the-Art Report on Temperature-Induced Deflections of Reinforced Concrete Members

435.8R Observed Deflections of Reinforced Concrete Slab Systems, and Causes of Large Deflections

444R Models of Concrete Structures—State of the Art

American Society of Civil Engineers (ASCE)

ASCE 7 Minimum Design Loads for Buildings and Other Structures

SEI/ASCE 11 Guideline for Structural Condition Assessment of Existing Buildings

ASTM International

A 370 Test Methods and Definitions for Mechanical Testing of Steel Products

C 39 Test Method for Compressive Strength of Cylindrical Concrete Specimens

C 42/C 42 M Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

C 114 Test Methods for Chemical Analysis of Hydraulic Cement

C 295 Guide for Petrographic Examination of Aggregates for Concrete

C 457 Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

C 597 Test Method for Pulse Velocity Through Concrete

C 617 Practice for Capping Cylindrical Concrete Specimens

C 803/C 803M Test Method for Penetration Resistance of Hardened Concrete

C 805 Test Method for Rebound Number of Hardened Concrete

C 823 Practice for Examination and Sampling of Hardened Concrete in Constructions

C 856 Practice for Petrographic Examination of Hardened Concrete

C 876 Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel

C 900 Test Method for Pullout Strength of Hardened Concrete

C 1152/C 1152M Test Method for Acid-Soluble Chloride in Mortar and Concrete

C 1218/C 1218M Test Method for Water-Soluble Chloride in Mortar and Concrete

C 1383/C 1383M Test Method for Measuring P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method

D 4580 Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding

D 4748 Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar

D 4788 Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography

D 6087 Test Method for Evaluating Asphalt-Covered Concrete Bridge Decks Using Ground Penetrating Radar

E 122 Practice for Choice of Sample Size to Estimate a Measure of Quality for a Lot or Process

International Organization for Standardization (ISO)

ISO 2394 General Principles on Reliability for Structures

ISO 13822 Bases for Design of Structures—Assessment of Existing Structures

RILEM—International Association for Building Materials and Structures

RILEM TBS-2 General Recommendation for Statistical Loading Test of Load-Bearing Concrete Structures *in situ*

These publications may be obtained from these organizations:

American Association of State Highway and Transportation Officials (AASHTO)
444 North Capitol St. N.W., Ste 249
Washington, DC 20001

ACI International (ACI)
PO Box 9094
Farmington Hills, MI 48333-9094

American Society of Civil Engineers (ASCE)
1801 Alexander Bell Dr.
Reston, VA 20191-4400

ASTM International
100 Barr Harbor Dr.
PO Box C700
West Conshohocken, PA 19428

International Code Council (ICC)
5203 Leesborg Pike, Ste 708
Falls Church, VA 22041

International Concrete Repair Institute (ICRI)
3166 S. River Road, Suite 132
Des Plaines, IL 60018

ISO Central Secretariat
International Organization for Standardization (ISO)
1, rue de Varembé, Case postale 56
CH-1211 Geneva 20, Switzerland

RILEM Secretariat General
E N S—Bâtiment Cournot, 61 avenue du Président Wilson
F-94235 Cachan Cedex, France

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APPENDIX A—CYCLIC LOAD TEST METHOD

A.1 *Cyclic load test*—Cyclic load tests have been performed on building structures as a diagnostic method to evaluate the performance of structural members in a short duration of time as compared with a standard load test (CIAS 2000). The cyclic load test is not meant to replace the standard 24 h load test. It has been used, however, as a way to validate the behavior of structural members, particularly in cases of upgrading using emerging materials not included in the building code and having a limited available track record.

The loads and measurements are designed to reveal key performance characteristics of the structural member under consideration. Depending upon load magnitude and geometry of the structure, a suitable load application method is selected. The primary difference among the methods used in

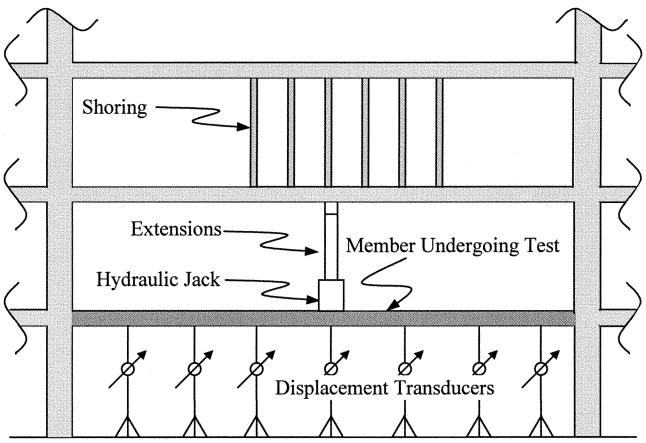

Fig. A.1—Push-down test configuration.

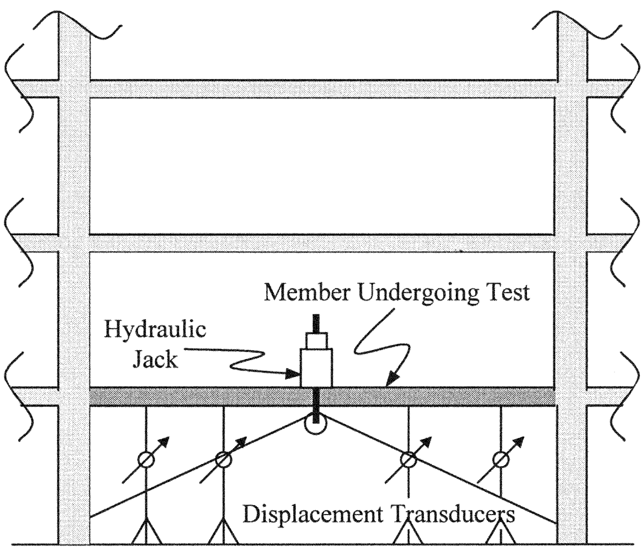

Fig. A.2—Pull-down test configuration with a fixed reaction.

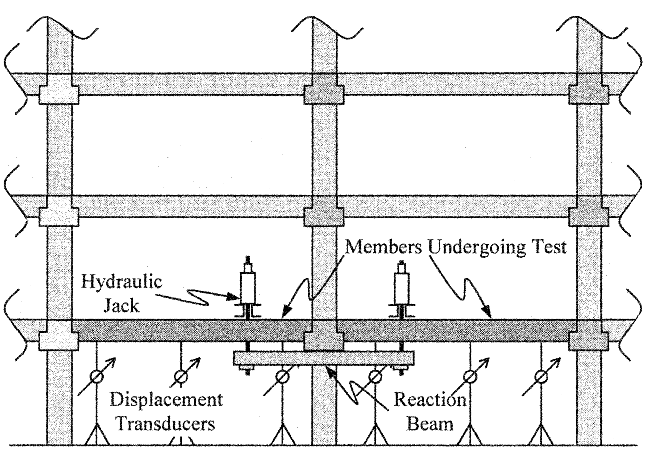

Fig. A.3—Closed loop test configuration.

the field is based on how the reaction is provided to the hydraulic jacks that apply the cyclic test load. Some of the methods commonly used for cyclic load testing are:

- *Push-down method*—One or more hydraulic jacks provide downward concentrated forces on the test member, as shown in Fig. A-1;
- *Pull-down method (fixed and mobile reaction)*—The reaction to the pull-down force is provided by a fixed structure, such as a pile or a column, below the tested member or by a mobile device such as a forklift or truck (Fig. A-2); and
- *Closed loop method*—The test setup is arranged in such a way that no external reaction is required for load application, as shown in Fig. A.3.

A cyclic load test consists of the application of concentrated loads in a quasistatic manner to the structural member, in at least six loading/unloading cycles, using one of the methods mentioned previously. The minimum number of cycles and the number of steps should be as described as follows:

Benchmark—The initial reading of the instrumentation is taken no more than 30 min before any test load is applied. It is shown in Fig. A.4 as the constant line beginning at time zero and indicating no load.

Cycle A—The first load cycle consists of five load steps, each increased by no more than 10% of the total test load expected in the cyclic load test and defined as $0.85(1.4D + 1.7L)$. The load is increased in steps, not exceeding the service level of the member or 50% of the total test load, as shown in Fig. A.4. At the end of each load step, the load should be maintained until measured parameters that define the response of the structure (such as deflections, rotations, and strains) have stabilized, but not less than 2 min. The maximum load level for the cycle should also be maintained until the structural response parameters have stabilized, but not less than 2 min. While unloading, the load should be held constant at the same load levels as the loading steps for at least 2 min, as shown in Fig. A.4. During each unloading phase, a minimum load P_{min}—at least 10% of the total test load, should be maintained to leave the test devices engaged.

Cycle B—The second load cycle is a repeat of Cycle A, which checks the repeatability of the structural response parameters obtained in the first cycle (see Section A.2).

Cycles C and D—Load Cycles C and D are identical and achieve a maximum load level that is approximately halfway between the maximum load level achieved in Cycles A and B and 100% of the total test load. The loading procedure is similar to that of Load Cycles A and B.

Cycles E and F—The fifth and sixth load cycles, E and F respectively, should be identical, and they should reach the total test load, as shown in Fig. A.4. Engineering judgment should be used in determining the duration of a load step, particularly when a suspected source of weakness involves a mechanism that may be sensitive to the duration of loading.

Final cycle—At the conclusion of Cycle F, the test load should be decreased to zero, as shown in Fig. A.4. A final reading should be taken no sooner than 2 min after the total test load, not including the equipment used to apply the load, has been removed.

Figure A.5 illustrates a schematic load versus deflection curve derived from the six cycles.

A.2 *Criteria for evaluation of cyclic load test*—The tested structural member is satisfactory if:

- The structural response parameters are stable under a

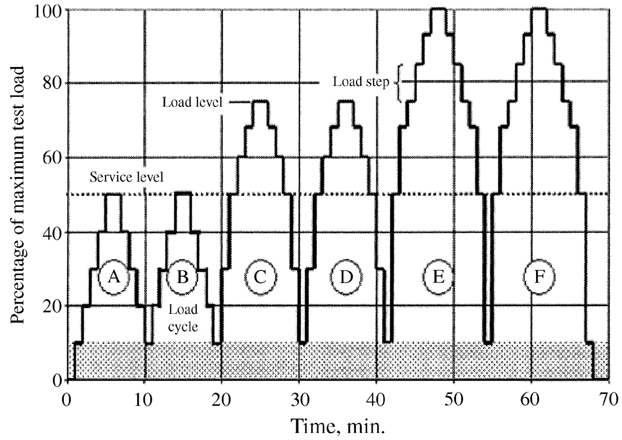

Fig. A.4—Load steps and cycles for a cyclic load test.

constant load, which shows the ability of the member in maintaining the load safely;

- The repeatability of the member deflection, which is a ratio of the difference between maximum and residual deflections recorded during the second of two identical load cycles to that of the first, is greater than 95%. An upper-bound value for the repeatability coefficient would be a logical limitation, but it has not yet been determined. Repeatability is given by Eq. (A-1) as follows (Fig. A.6)

$$\text{Repeatability} = \frac{\Delta^B_{max} - \Delta^B_r}{\Delta^A_{max} - \Delta^A_r} \times 100\% \qquad \text{(A-1)}$$

where

Δ^B_{max} = maximum deflection in Cycle B under a load of P_{max};

Δ^B_r = residual deflection after Cycle B under a load of P_{min};

Δ^A_{max} = maximum deflection in Cycle A under a load of P_{max}; and

Δ^A_r = residual deflection after Cycle A under a load of P_{min}.

Equation (A-1) should also be verified for Cycles C and D, and E and F.

- Deviation from linearity, which is a measure of the nonlinear behavior of the member being tested, should have a value less than 25%. The linearity of any point i on the load-deflection envelope is the percent ratio of the slope of that point's secant line, expressed by $\tan(\alpha_i)$, to the slope of the reference secant line, expressed by $\tan(\alpha_{ref})$, as shown in Eq. (A-2) (Fig. A.5)

$$\text{Linearity}_i = \frac{\tan(\alpha_i)}{\tan(\alpha_{ref})} \times 100\% \qquad \text{(A-2)}$$

The deviation from linearity of any point on the load-deflection envelope is the compliment of the linearity of that point, as given by Eq. (A-3)

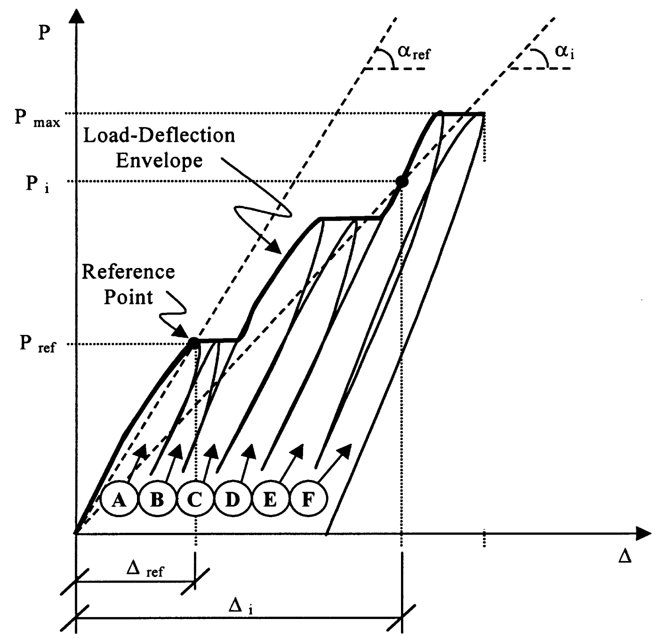

Fig. A.5—Schematic load-versus-deflection curve for six cycles.

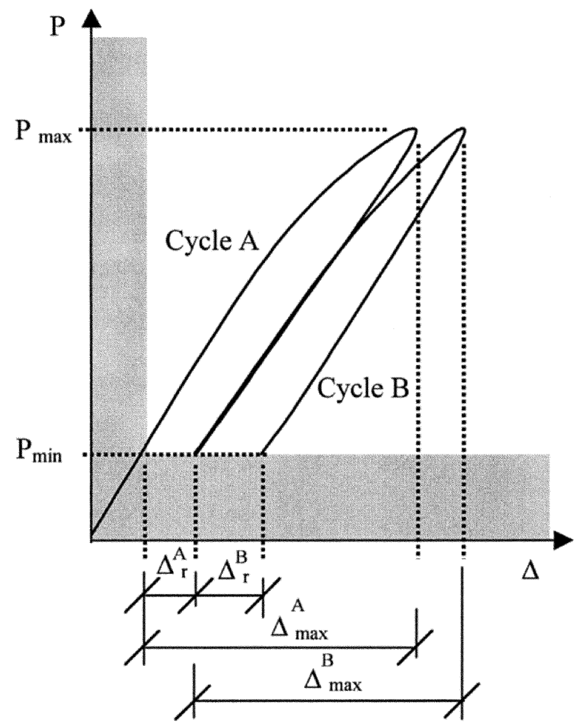

Fig. A.6—Example of load-versus-deflection curve for two cycles.

$$\text{Deviation from Linearity}_i = 100\% - \text{Linearity}_i \qquad \text{(A-3)}$$

- Permanency, which is the amount of permanent change displayed by any structural response parameter during the second of two identical load cycles, should be less than 10%. Permanency (for example, during Cycle B) is given by Eq. (A-4), as follows (Fig. A.6)

$$\text{Permanency} = \frac{\Delta_r^B}{\Delta_{max}^B} \times 100\% \qquad \text{(A-4)}$$

Besides the previous criteria, the structure should not show any visible evidence of failure.

APPENDIX B—REPORTS FROM OTHER ORGANIZATIONS

In addition to ACI, other national and international organizations have developed or are developing guidelines for the assessment of structures (RILEM TBS-2 1983; SEI/ASCE 11-99 1999; ISO 13822; RILEM JCSS 2001). These documents may or may not be specific to concrete or buildings but offer relevant and complementary information to this report. A brief synopsis of each is offered in this section:

General Recommendation for Statical Loading Test of Load-Bearing Concrete Structures in Situ (RILEM TBS-2)—The Technical Committee 20-TBS was established in 1971, and its final report was accepted by RILEM in 1980. This document includes recommendations regarding short-term and long-term deflections and a method of evaluating the serviceability limit state of structures.

ASCE Standard: Guideline for Structural Condition Assessment of Existing Building (SEI/ASCE 11-99)—The standard provides guidelines and methodologies for assessing the structural condition of existing buildings constructed of combinations of materials including concrete, metals, masonry, and wood. This standard establishes the assessment procedure including investigation, testing methods, and format for the report of the condition assessment. Because any evaluation will involve engineering judgment and contains factors that cannot be readily defined and standardized, a section providing guidance for evaluations is also included.

Basis for Design of Structures—Assessment of Existing Structures (ISO 13822)—This international standard provides general requirements and procedures for the assessment of existing structures, such as buildings, bridges, and industrial structures, based on the principles of structural reliability and risk analysis. The basis for the reliability assessment is contained in the performance requirements for safety and serviceability of ISO 2394.

Probabilistic Assessment of Existing Structures (RILEM JCSS)—The RILEM JCSS report contains practical and operational recommendations and rules for the assessment and analysis of existing structures, provides guidance for the acceptance criteria, and includes illustrated examples and case studies. The report includes four parts including assessment of existing structures, analysis, acceptance criteria, and examples of case studies.