In-Place Methods to Estimate Concrete Strength

Reported by ACI Committee 228

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Guidance is provided on the use of methods to estimate the in-place strength of concrete in new and existing construction. The methods include: rebound number, penetration resistance, pullout, break-off, ultrasonic pulse velocity, maturity, and cast-in-place cylinders. The principle, inherent limitations, and repeatability of each method are reviewed. Procedures are presented for developing the relationship needed to estimate compressive strength from in-place results. Factors to consider in planning in-place tests are discussed, and statistical techniques to interpret test results are presented. The use of in-place tests for acceptance of concrete is introduced. The appendix provides information on the number of strength levels that should be used to develop the strength relationship and explains a regression analysis procedure that accounts for error in both dependent and independent variables.

Keywords: coefficient of variation; **compressive strength**; construction; in-place tests; **nondestructive tests**; safety; sampling; statistical analysis.

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CHAPTER 1—INTRODUCTION

1.1—Scope

In-place tests are performed typically on concrete within a structure, in contrast to tests performed on molded specimens made from the concrete to be used in the structure. Historically, they have been called nondestructive tests because some of the early tests did not damage the concrete. Over the years, however, new methods have developed that result in superficial local damage. Therefore, the terminology in-place tests is used as a general category that includes those that do not alter the concrete and those that result in minor surface damage. In this Report, the principal application of in-place tests is to estimate the compressive strength of the concrete. The significant characteristic of most of these tests is that they do not directly measure the compressive strength of the concrete in a structure. Instead, they measure some other property that can be correlated to compressive strength (Popovics 1998). The strength is then estimated from a previously established relationship between the measured property and concrete strength. The uncertainty of the estimated compressive strength depends on the variability of the in-place test results and the uncertainty of the relationship between these two parameters. These sources of uncertainty are discussed in this Report.

In-place tests can be used to estimate concrete strength during construction so that operations that require a specific strength can be performed safely or curing procedures can be terminated. They can also be used to estimate concrete strength during the evaluation of existing structures. These two applications require slightly different approaches, so parts of this Report are separated into sections dealing with new and existing construction.

A variety of techniques are available for estimating the in-place strength of concrete (Malhotra 1976; Bungey 1989; Malhotra and Carino 1991). No attempt is made to review all of these methods in this report; only those methods that have been standardized by ASTM are discussed. Teodoru (1989) prepared a compilation of national standards on in-place test methods.

1.2—Need for in-place tests during construction

In North American practice, the most widely used test for concrete is the compressive strength test of the standard cylinder (ASTM C 31/C 31M). This test procedure is relatively easy to perform in terms of sampling, specimen preparation, and strength measurement. When properly performed, this test has low within-test variation and low interlaboratory variation and, therefore, readily lends itself to use as a standard test method. The compressive strength so obtained is used to calculate the nominal strengths of structural members. Therefore, this strength value is an essential parameter in design codes.

When carried out according to standard procedures, however, the results of the cylinder compression test represent the potential strength of the concrete as delivered to a site. The test is used mainly as a basis for quality control of the concrete to ensure that contract requirements are met. It is not intended for determining the in-place strength of the concrete because it makes no allowance for the effects of placing, compaction, or curing. It is unusual for the concrete in a structure to have the same properties as a standard-cured cylinder at the same test age. Also, standard-cured cylinders are usually tested for acceptance purposes at an age of 28 days; therefore, the results of these tests cannot be used to determine whether adequate strength exists at earlier ages for safe removal of formwork or the application of post-tensioning. The concrete in some parts of a structure, such as columns, may develop strength equal to the standard 28-day cylinder strength by the time it is subjected to design loads. Concrete in most flexural members (especially pretensioned flexural members) does not develop its 28-day strength before the members are required to support large percentages of their design loads. For these reasons, in-place tests are used to estimate the concrete strength at critical locations in a structure and at times when crucial construction operations are scheduled.

Traditionally, some measure of the strength of the concrete in the structure has been obtained by using field-cured cylinders prepared and cured in accordance with ASTM C 31/C 31M. These cylinders are cured on or in the structure under, as nearly as possible, the same conditions as the concrete in the structure. Measured strengths of field-cured cylinders may be significantly different from in-place strengths because it is difficult, and often impossible, to have identical bleeding, consolidation, and curing conditions for concrete in cylinders and concrete in structures (Soutsos et al. 2000). Field-cured specimens need to be handled with care and stored properly to avoid misleading test results.

Construction schedules often require that operations such as form removal, post-tensioning, termination of curing, and removal of reshores be carried out as early as possible. To enable these operations to proceed safely at the earliest possible time requires the use of reliable in-place tests to estimate the in-place strength. The need for such strength information is emphasized by several construction failures that possibly could have been prevented had in-place testing been used (Lew 1980; Carino et al. 1983). In-place testing not only increases safety but can result in substantial cost savings by permitting accelerated construction schedules (Bickley 1982a).

1.3—Influence of ACI 318

Before 1983, ACI 318 required testing of field-cured cylinders to demonstrate the adequacy of concrete strength before removal of formwork or reshoring. Section 6.2.2.1 of ACI 318-83 allowed the use of alternative procedures to test field-cured cylinders. The building official, however, must approve the alternative procedure before its use. Since 1983, ACI 318 has permitted the use of in-place testing as an alternative to testing field-cured cylinders. The commentary to ACI 318-02 (Section R6.2) lists four procedures, which are covered in this Report, that may be used, provided there are sufficient correlation data (ACI 318R).

Most design provisions in ACI 318 are based on the compressive strength of standard cylinders. Thus, to evaluate structural capacity under construction loading, it is necessary to have an estimate of the equivalent cylinder strength of the concrete as it exists in the structure. If in-place tests are used, a valid relationship between the results of in-place tests and the compressive strength of cylinders must be established. At present, there are no standard practices for developing the required relationship. There are also no generally accepted guidelines for interpretation of in-place test results. These deficiencies have been impediments to widespread adoption of in-place tests. One of the objectives of this Report is to eliminate some of these deficiencies.

1.4—Recommendations in other ACI documents

After the 1995 version of this Report was published, other ACI documents incorporated in-place tests as alternative procedures for estimating in-place strength. One of these documents is ACI 301. In the 1999 version of ACI 301, Paragraph 1.6.5.2 on in-place testing of hardened concrete includes the following:

"Use of the rebound hammer in accordance with ASTM C 805, pulse-velocity method in accordance with ASTM C 597, or other nondestructive tests may be permitted by the Architect/Engineer in evaluating the uniformity and relative concrete strength in-place, or for selecting areas to be cored."

ACI 301-99 states in Paragraph 1.6.6.1 that the results of in-place tests "will be valid only if the tests have been conducted using properly calibrated equipment in accordance with recognized standard procedures and acceptable correlation between test results and concrete compressive strength has been established and is submitted." Paragraph 1.6.7.2 of ACI 301-99, however, restricts the use of these tests in acceptance of concrete by stating that: "Nondestructive tests shall not be used as the sole basis for accepting or rejecting concrete," but they may be used to "evaluate" concrete when the standard-cured cylinder strengths fail to meet the specified strength criteria.

ACI 301-99 also mentions in-place tests in Article 2.3.4 dealing with required strength for removal of formwork. Specifically, it is stated that the following methods may be used when permitted or specified, provided sufficient correlation data are submitted:

- ASTM C 873 (cast-in-place cylinders);
- ASTM C 803/C 803M (penetration resistance);
- ASTM C 900 (pullout);
- ASTM C 1074 (maturity method); and
- ASTM C 1150 (break-off).

These same methods are also recommended as alternatives to testing field-cured cylinders for estimating in-place strength for the purpose of terminating curing procedures.

ACI 308.1 also mentions in-place tests as acceptable methods for estimating in-place strength for the purpose of terminating curing procedures (see Paragraph 1.6.4 of ACI 308.1-98). Thus, project specifications can reference standard specifications that allow in-place testing as an alternative to testing field-cured cylinders. In all cases, however, sufficient correlation data are required and permission has to be granted before using an in-place test method. This Report explains how the required correlation data can be acquired and it provides guidance on how to implement an in-place testing program.

1.5—Existing construction

Reliable estimates of the in-place concrete strength are required for structural evaluation of existing structures (ACI 437R). Historically, in-place strength has been estimated by testing cores drilled from the structure. In-place tests can supplement coring and can permit more economical evaluation of the concrete in the structure. The critical step in such applications is to establish the relationship between in-place test results and concrete strength. The present approach is to correlate results of in-place tests performed at selected locations with strength of corresponding cores. In-place testing does not eliminate the need for coring, but it can reduce the total amount of coring needed to evaluate a large volume of concrete. A sound sampling plan is needed to acquire the correlation data, and appropriate statistical methods should be used for reliable interpretation of test results.

1.6—Objective of report

This Report reviews ASTM test methods for estimating the in-place strength of concrete in new construction and in existing structures. The overall objective is to provide the potential user with a guide to assist in planning, conducting, and interpreting the results of in-place tests.

Chapter 2 discusses the underlying principles and inherent limitations of in-place tests. Chapter 3 reviews the statistical characteristics of in-place tests. Chapter 4 outlines procedures to develop the relationship needed to estimate in-place compressive strength. Chapter 5 discusses factors to be considered in planning the in-place testing program. Chapter 6 presents statistical techniques to interpret in-place test results. Chapter 7 discusses in-place testing for acceptance of concrete. Chapter 8 lists the cited references. The appendix provides details on the statistical principles discussed in the report and includes an illustrative example.

CHAPTER 2—REVIEW OF METHODS

2.1—Introduction

Often, the objective of in-place testing is to estimate the compressive strength of concrete in the structure. To make a strength estimate, it is necessary to have a known relationship between the result of the in-place test and the strength of the concrete. For new construction, this relationship is usually established empirically in the laboratory. For existing construction, the relationship is usually established by performing in-place tests at selected locations in the structure and determining the strength of cores drilled from adjacent locations. Figure 2.1 is a schematic of a strength relationship in which the cylinder compressive strength is plotted as a function of an in-place test result. This relationship would be used to estimate the strength of concrete in a structure based on the value of the in-place test result obtained from testing the structure. The accuracy of the strength estimate depends on the degree of correlation between the strength of concrete and the quantity measured by the in-place test. The user of in-place tests should have an understanding of what property is measured by the test and how this property is related to the strength of concrete.

The purpose of this chapter is to explain the underlying principles of the widely used in-place test methods, and to identify the factors, other than concrete strength, that can influence the test results. Additional background information on these methods is available in the references by Malhotra (1976), Bungey (1989), and Malhotra and Carino (1991).

The following methods are discussed:

- Rebound number;
- Penetration resistance;
- Pullout;
- Break-off;
- Ultrasonic pulse velocity;
- Maturity; and
- Cast-in-place cylinder.

2.2—Rebound number (ASTM C 805)

The operation of the rebound hammer (also called the Schmidt Hammer or Swiss Hammer) is illustrated in Fig. 2.2.

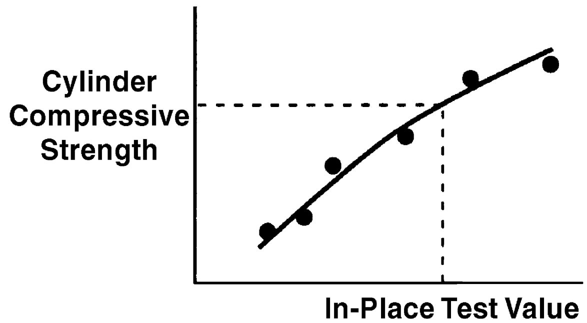

Fig. 2.1—Schematic of relationship between cylinder compressive strength and in-place test value.

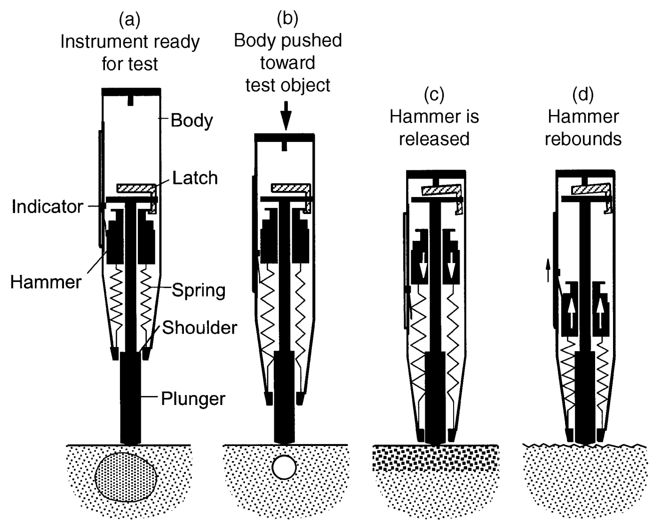

Fig. 2.2—Schematic to illustrate operation of the rebound hammer.

The device consists of the following main components: 1) outer body; 2) plunger; 3) hammer; and 4) spring. To perform the test, the plunger is extended from the body of the instrument and brought into contact with the concrete surface. When the plunger is extended, a latching mechanism locks the hammer to the upper end of the plunger. The body of the instrument is then pushed toward the concrete member. This action causes an extension of the spring connecting the hammer to the body (Fig. 2.2(b)). When the body is pushed to its limit of travel, the latch is released, and the spring pulls the hammer toward the concrete member (Fig. 2.2(c)). The hammer impacts the shoulder area of the plunger and rebounds (Fig. 2.2(d)). The rebounding hammer moves the slide indicator, which records the rebound distance. The rebound distance is measured on a scale numbered from 10 to 100 and is recorded as the rebound number.

The key to understanding the inherent limitations of this test for estimating strength is recognizing the factors influencing the rebound distance. From a fundamental point of view, the test is a complex problem of impact loading and stress-wave propagation. The rebound distance depends on the kinetic energy in the hammer before impact with the shoulder of the plunger and the amount of that energy absorbed during the impact. Part of the energy is absorbed as mechanical friction in the instrument, and part of the energy is absorbed in the interaction of the plunger with the concrete. It is the latter factor that makes the rebound number an indicator of the concrete properties. The energy absorbed by the concrete depends on the stress-strain relationship of the concrete. Therefore, absorbed energy is related to the strength and the stiffness of the concrete. A low-strength, low-stiffness concrete will absorb more energy than a high-strength, high-stiffness concrete. Thus, the low-strength concrete will result in a lower rebound number. Because it is possible for two concrete mixtures to have the same strength but different stiffnesses, there could be different rebound numbers even if the strengths are equal. Conversely, it is possible for two concretes with different strengths to have the same rebound numbers if the stiffness of the low-strength concrete is greater than the stiffness of the high-strength concrete. Because aggregate type affects the stiffness of concrete, it is necessary to develop the strength relationship on concrete made with the same materials that will be used for the concrete in the structure.

In rebound-hammer testing, the concrete near the point where the plunger impacts influences the rebound value. Therefore, the test is sensitive to the conditions at the location where the test is performed. If the plunger is located over a hard aggregate particle (Fig. 2.2(a)), an unusually high rebound number will result. On the other hand, if the plunger is located over a large air void (Fig. 2.2(b)) or over a soft aggregate particle, a lower rebound number will occur. Reinforcing bars with shallow concrete cover may also affect rebound numbers if tests are done directly over the bars. To account for these possibilities, ASTM C 805 requires that 10 rebound numbers be taken for a test. If a reading differs by more than six units from the average, that reading should be discarded and a new average should be computed based on the remaining readings. If more than two readings differ from the average by six units, the entire set of readings is discarded.

Because the rebound number is affected mainly by the near-surface layer of concrete, the rebound number may not represent the interior concrete. The presence of surface carbonation (Fig. 2.2(c)) can result in higher rebound numbers that are not indicative of the interior concrete. Similarly, a dry surface will result in higher rebound numbers than for the moist, interior concrete. Absorptive oiled plywood can absorb moisture from the concrete and produce a harder surface layer than concrete cast against steel forms. Similarly, curing conditions affect the strength and stiffness of the near-surface concrete more than the interior concrete. The surface texture may also influence the rebound number. When the test is performed on rough concrete (Fig. 2.2(d)), local crushing occurs under the plunger and the indicated concrete strength will be lower than the true value. Rough surfaces should be ground before testing. If the formed surfaces are smooth, grinding is unnecessary. A hard, smooth surface, such as a surface produced by trowel finishing, can result in higher rebound numbers. Finally, the rebound distance is affected by the orientation of the instrument, and the strength relationship must be developed for the same instrument orientation as will be used for in-place testing.

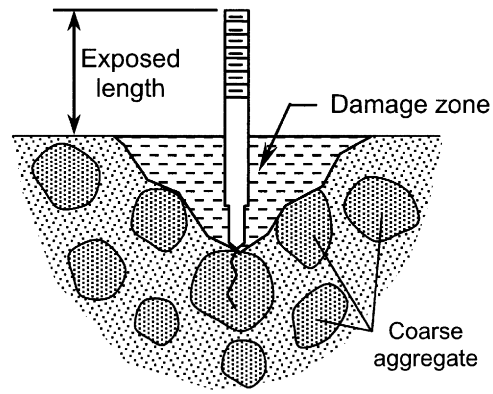

Fig. 2.3—Approximate shape of failure zone in concrete during probe penetration test.

In summary, while the rebound number test is simple to perform, there are many factors other than concrete strength that influence the test results. As a result, estimated strengths are not as reliable as those from other in-place test methods to be discussed.

2.3—Penetration resistance (ASTM C 803/C 803M)

In the penetration-resistance technique, one measures the depth of penetration of a rod (probe) or a pin forced into the hardened concrete by a driver unit.

The probe-penetration technique involves the use of a specially designed gun to drive a hardened steel probe into the concrete. (The commercial test system is known as the Windsor Probe.) The depth of penetration of the probe is an indicator of the concrete strength. This method is similar to the rebound number test, except that the probe impacts the concrete with much higher energy than the plunger of the rebound hammer. The probe penetrates into the concrete while the plunger of the rebound hammer produces only a minor surface indentation. A theoretical analysis of this test is even more complicated than the rebound test, but again the essence of the test involves the initial kinetic energy of the probe and energy absorption by the concrete. The probe penetrates into the concrete until its initial kinetic energy is absorbed. The initial kinetic energy is governed by the charge of smokeless powder used to propel the probe, the location of the probe in the gun barrel before firing, and frictional losses as the probe travels through the barrel. An essential requirement of this test is that the probes have a consistent value of initial kinetic energy. ASTM C 803/C 803M requires that the probe exit velocities do not have a coefficient of variation greater than 3% based on 10 tests by approved ballistic methods.

As the probe penetrates into the concrete, some energy is absorbed by friction between the probe and the concrete, and some is absorbed by crushing and fracturing of the concrete. There are no rigorous studies of the factors affecting the geometry of the fracture zone, but its general shape is probably as illustrated in Fig. 2.3. There is usually a cone-shaped region in which the concrete is heavily fractured, and most of the probe energy is absorbed in this zone.

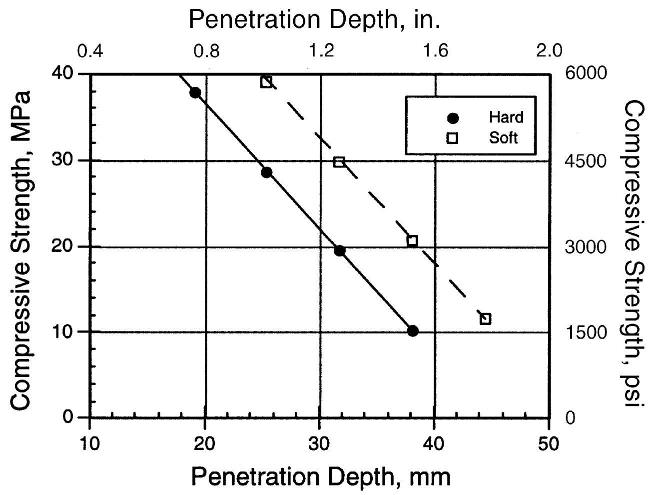

Fig. 2.4—Effect of aggregate type on relationship between concrete strength and depth of probe penetration.

The probe tip can travel through mortar and aggregate; in general, cracks in the fracture zone will be through the mortar matrix and the coarse-aggregate particles. Hence, the strength properties of both the mortar and coarse aggregate influence the penetration distance. This contrasts with the behavior of normal-strength concrete in a compression test, where mortar strength has the predominant influence on measured compressive strength. Thus, an important characteristic of the probe penetration test is that the type of coarse aggregate greatly affects the relationship between concrete strength and depth of probe penetration. For example, Fig. 2.4 compares empirical relationships between compressive strength and probe penetration for concrete made with a soft aggregate (such as limestone) and concrete made with a hard aggregate (such as chert). For equal compressive strengths, the concrete with the soft aggregate allows greater probe penetration than the concrete with the hard aggregate. More detailed information on the influence of aggregate type on strength relationships can be found in Malhotra (1976), Bungey (1989), and Malhotra and Carino (1991).

Because the probe penetrates into the concrete, test results are not usually affected by local surface conditions such as texture and moisture content. A harder surface layer, however, as would occur with trowel finishing, can result in low penetration values and excessive scatter of data. In addition, the direction in which the test is performed is unimportant if the probe is driven perpendicular to the surface. The penetration will be affected by the presence of reinforcing bars within the zone of influence of the penetrating probe. Thus, the location of the reinforcing steel should be determined before selecting test sites. Covermeters can be used for this purpose (ACI 228.2R).

In practice, it is customary to measure the exposed length of the probes. The fundamental relationship, however, is between concrete strength and depth of penetration. Therefore, when assessing the variability of test results (refer to Chapter 3), it is preferable to express the coefficient of variation in terms of penetration depth rather than exposed length.

Before 1999, the hardened steel probes were limited to use in concrete with compressive strength less than about 40 MPa (6000 psi). There was a tendency for the probes to fracture within the threaded region when testing stronger concrete. Al-Manaseer and Aquino (1999) reported that a newer probe made with stress-relieved alloy steel was successfully used to test concrete with a compressive strength of 117 MPa (17,000 psi).

A pin penetration test device, requiring less energy than the Windsor Probe system, was developed by Nasser (Nasser and Al-Manaseer 1987a,b), and the procedure for its use was subsequently incorporated into ASTM C 803/C 803M. A spring-loaded device is used to drive a pointed 3.56 mm (0.140 in.) diameter hardened steel pin into the concrete. The penetration by the pin creates a small indentation (or hole) in the surface of the concrete. The pin is removed from the hole, the hole is cleaned with an air jet, and the hole depth is measured with a suitable depth gage. The penetration depth is used to estimate compressive strength from a previously established strength relationship.

The kinetic energy delivered by the pin penetration device is estimated to be about 1.3% of the energy delivered by the Windsor Probe system (Carino and Tank 1989). Because of the low energy level, the penetration of the pin is reduced greatly if the pin encounters a coarse-aggregate particle. Thus, the test is intended as a penetration test of the mortar fraction of the concrete. Results of tests that penetrate coarse-aggregate particles are not considered in determining the average pin penetration resistance (ASTM C 803/C 803M). A pin may become blunted during penetration. Because the degree of blunting affects the penetration depth, ASTM C 803/C 803M requires that a new pin be used for each penetration test.

The sensitivity of the pin penetration to changes in compressive strength decreases for concrete strength above 28 MPa (4000 psi) (Carino and Tank 1989). Therefore, the pin penetration test system is not recommended for testing concrete having a compressive strength above 28 MPa (4000 psi).

In summary, concrete strength can be estimated by measuring the penetration depth of a probe or pin driven into the concrete at constant energy. Penetration tests are less affected by surface conditions than the rebound number method. The coarse aggregate, however, has a significant effect on the resulting penetration. For the gun-driven probe system, the type of coarse aggregate affects the strength relationship; for the spring-driven pin system, tests that impact coarse aggregate particles are disregarded.

2.4—Pullout test (ASTM C 900)

The pullout test measures the maximum force required to pull an embedded metal insert with an enlarged head from a concrete specimen or structure. The pullout force is applied by a loading system that reacts against the concrete surface through a reaction ring concentric with the insert (Fig. 2.5). As the insert is pulled out, a roughly cone-shaped fragment of the concrete is extracted. The large diameter of the conic fragment, d_2, is determined by the inner diameter of the reaction ring, and the small diameter d_1 is determined by the insert-head diameter. Requirements for the testing configuration are

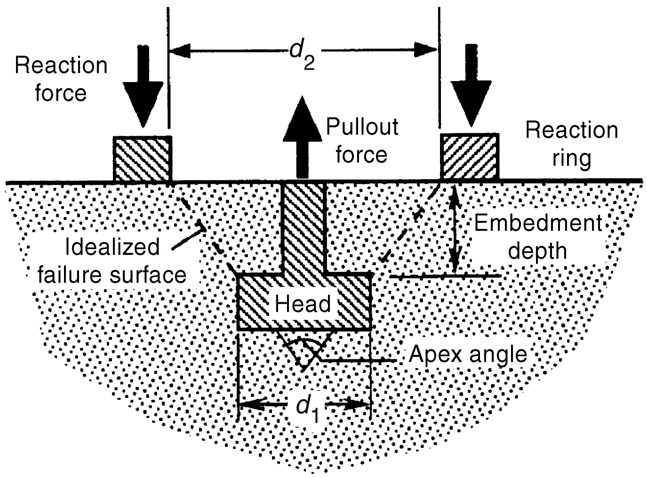

Fig. 2.5—Schematic of pullout test.

given in ASTM C 900. The embedment depth and head diameter must be equal, but there is no requirement on the magnitude of these dimensions. The inner diameter of the reaction ring can be between 2.0 and 2.4 times the insert-head diameter. This means that the apex angle of the conic frustum defined by the insert-head diameter and the inside diameter of the reaction ring can vary between 54 and 70 degrees. The same test geometry must be used for developing the strength relationship and for the in-place testing.

Unlike the rebound hammer and probe-penetration tests, the pullout test subjects the concrete to a static loading that lends itself to stress analysis. The finite-element method has been used to calculate the stresses induced in the concrete before cracking (Stone and Carino 1984) and where the concrete has cracked (Ottosen 1981). In these analyses, the concrete was assumed to be a homogeneous solid and the influence of discrete coarse-aggregate particles was not modeled. There is agreement (in cited literature) that the test subjects the concrete to a nonuniform, three-dimensional state of stress. Figure 2.6 shows the approximate directions (trajectories) of the principal stresses acting in radial planes (those passing through the center of the insert) before cracking for apex angles of 54 and 70 degrees. Because of symmetry, only 1/2 of the specimen is shown. These trajectories would be expected to change after cracking develops. Before cracking there are tensile stresses that are approximately perpendicular to the eventual failure surface measured by Stone and Carino (1984). Compressive stresses are directed from the insert head toward the ring. The principal stresses are nonuniform and are greatest near the top edge of the insert head.

A series of analytical and experimental studies, some of which are critically reviewed by Yener and Chen (1984), has been carried out to determine the failure mechanism of the pullout test. While the conclusions have been different, it is generally agreed that circumferential cracking (producing the failure cone) begins in the highly stressed region next to the insert head at a pullout load that is a fraction of the ultimate value. With increasing load, the circumferential cracking propagates from the insert head toward the reaction ring.

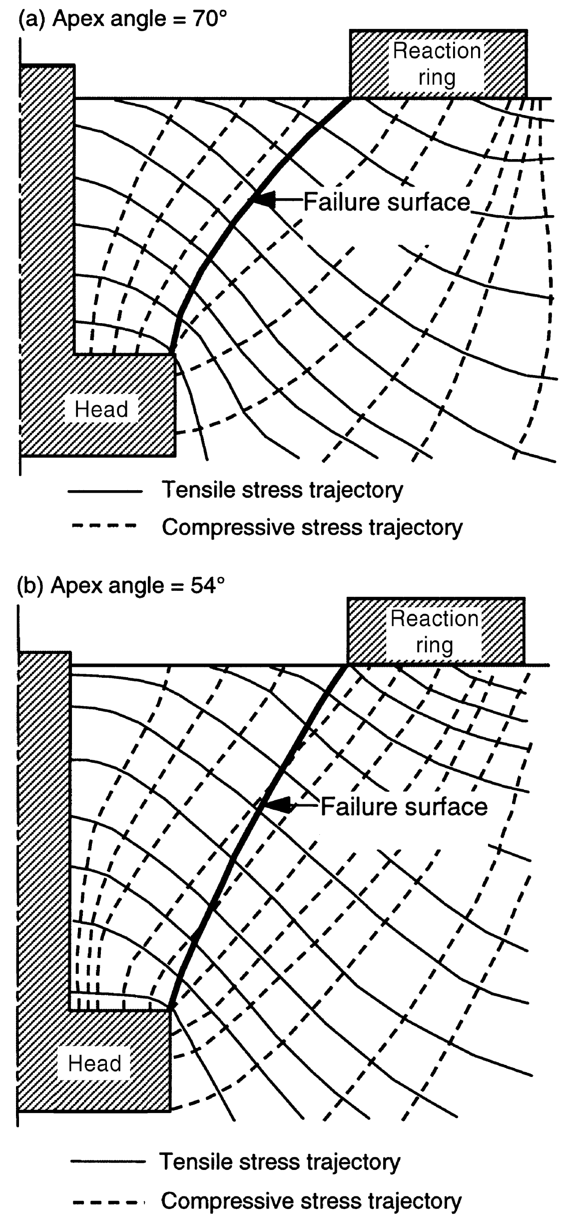

Fig. 2.6—Principal stress trajectories before cracking for pullout test in a homogeneous material and measured fracture surfaces in physical tests (Stone and Carino 1984).

There is no agreement, however, on the nature of the final failure mechanism governing the magnitude of the ultimate pullout load.

Ottosen (1981) concluded that failure is due to "crushing" of concrete in a narrow band between the insert head and the reaction ring. Thus, the pullout load is related directly to the compressive strength of the concrete. In another analytical study, Yener (1994) concluded that failure occurred by outward crushing of concrete around the perimeter of the failure cone near the reaction ring. Using linear-elastic fracture mechanics and a two-dimensional model, Ballarini, Shah, and Keer (1986) concluded that ultimate load is governed by the fracture toughness of the matrix. In an experimental study, Stone and Carino (1983) concluded that before ultimate load, circumferential cracking extends from the insert head to the

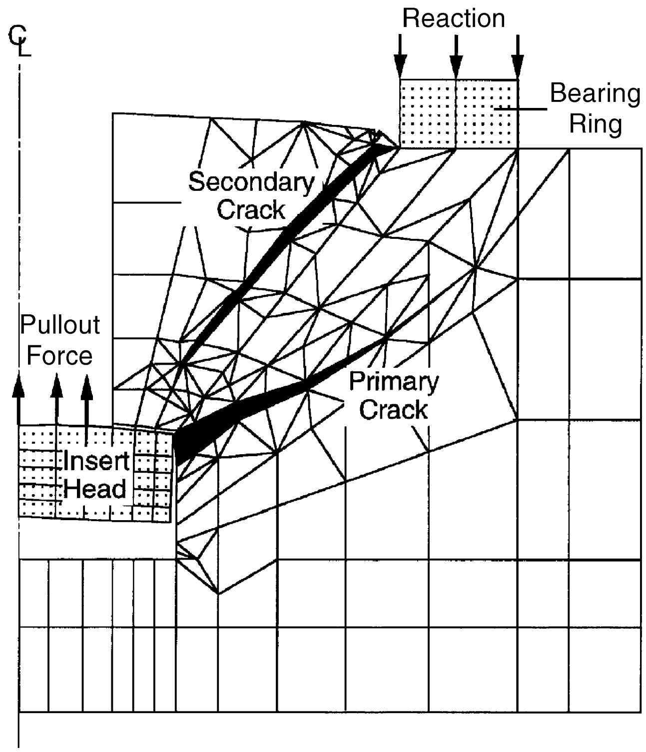

Fig. 2.7—Circumferential cracks predicted by nonlinear fracture mechanics analysis of pullout test by Hellier et al. (1987).

reaction ring and that additional load is resisted by aggregate interlock across the circumferential crack. In this case, failure occurs when sufficient aggregate particles have been pulled out of the mortar matrix. According to the aggregate interlock theory, maximum pullout force is not directly related to the compressive strength. There is good correlation, however, between ultimate pullout load and compressive strength of concrete because both values are influenced by the mortar strength (Stone and Carino 1984). In another study, using nonlinear fracture mechanics and a discrete cracking model, Hellier at al. (1987) showed excellent agreement between the predicted and observed internal cracking in the pullout test. Figure 2.7 shows the displaced shape of the finite-element model used. The analysis showed that a *primary* circumferential crack developed at the corner of the insert head and propagated outward at a shallow angle. This crack ceased to grow when it penetrated a tensile-free region. A *secondary* crack developed subsequently and propagated as shown in the figure. The secondary crack appeared to coincide with the final fracture surface observed when the conical fragment was extracted from the concrete mass during pullout testing. This study also concluded that the ultimate pullout load is not governed by uniaxial compressive failure in the concrete.

A positive feature of the pullout test is that it produces a well-defined fracture surface in the concrete and measures a static strength property of the concrete. Because there is no consensus on which strength property is measured, it is necessary to develop an empirical relationship between the pullout strength and the compressive strength of the concrete. The relationship that is developed is applicable to only the particular test configuration and concrete materials used in the correlation testing.

The pullout strength is primarily governed by the concrete located next to the conic frustum defined by the insert head and reaction ring. Commercial inserts have embedment depths of about 25 to 30 mm (1 to 1.2 in.). Thus, only a small volume of concrete is tested, and because of the inherent heterogeneity of concrete, the average within-batch coefficient of variation of these pullout tests has been found to be between 7 and 10%, which is about two to three times that of standard cylinder-compression tests.

In new construction, the most desirable approach for pullout testing is to attach the inserts to formwork before concrete placement. It is also possible, however, to place inserts into unformed surfaces, such as tops of slabs, by placing the inserts into fresh concrete that is sufficiently workable. The hardware includes a metal plate attached to the insert to provide a smooth bearing surface and a plastic cup to allow embedment of the plate slightly below the surface. The plastic cup also ensures that the insert will "float" in the fresh concrete and not settle before the concrete sets. When inserts are placed manually, care is required to maintain representative concrete properties at placement locations and to reduce the amount of air that becomes entrapped on the underside of the plates. In an early study, Vogt, Beizai, and Dilly (1984) reported higher than expected within-test variability when using manually placed inserts. Later work by Dilly and Vogt (1988), however, resulted in variability similar to that expected with inserts fastened to formwork. The recommended approach is to push the insert into fresh concrete and then float it horizontally over a distance of 50 to 100 mm (2 to 4 in.) to allow aggregate to flow into the pullout failure zone. After insertion, the insert should be tilted about 20 to 30 degrees from the vertical to allow entrapped air to escape from beneath the steel plate. Care should be exercised to ensure that the plate is completely below the concrete surface. To prevent movement of the insert before the concrete sets, fresh concrete can be placed in the cup.

In existing construction, it is possible to perform pullout tests using post-installed inserts. The procedure for performing post-installed pullout tests was included in the 1999 revision of ASTM C 900 and is summarized in Fig. 2.8. The procedure involves the following basic steps:

- Grinding the test area so that it is flat;
- Drilling a hole perpendicular to the surface of the concrete;
- Undercutting a slot to engage an expandable insert;
- Expanding an insert into the milled slot; and
- Pulling the insert out of the concrete.

The test geometry is the same as for the cast-in-place insert. In a commercial test system, known as CAPO (for Cut And PullOut), the insert is a coiled, split ring that is expanded with specially designed hardware. The CAPO system performs similarly to the cast-in-place system of the same geometry (Petersen 1984, 1997). Care is required during preparation to ensure that the hole is drilled perpendicular to the test surface. The surface must be flat so that the bearing ring of the loading system is supported uniformly when the insert is extracted. Nonuniform bearing of the reaction ring may result in an incomplete circle for the top surface of the extracted frustum. If this occurs, the test result must be rejected (ASTM C 900). Cooling water used for drilling and undercutting should be removed from the hole as soon as the undercutting is completed, and the hole should be protected from ingress of water until the test is completed. This is to prevent penetration of water into the fracture zone, which might affect the measured pullout load.

Other types of pullout test configurations are available for existing construction (Mailhot et al. 1979; Chabowski and Bryden-Smith 1980; Domone and Castro 1987). These typically involve drilling a hole and inserting an expanding 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 mechanisms as the standard pullout test. These techniques have not been standardized as ASTM tests methods; however, the internal fracture test by Chabowski and Bryden-Smith (1980) has been incorporated into a British standard (BS 1881-Part 207).

In summary, the pullout test can be used to estimate the strength of concrete by measuring the force required to extract an insert embedded in fresh concrete or installed in hardened concrete. The test results in a complex, three-dimensional state of stress in the concrete. While the exact failure mechanism is still a matter of controversy, there is a strong relationship between the compressive strength of concrete and pullout strength.

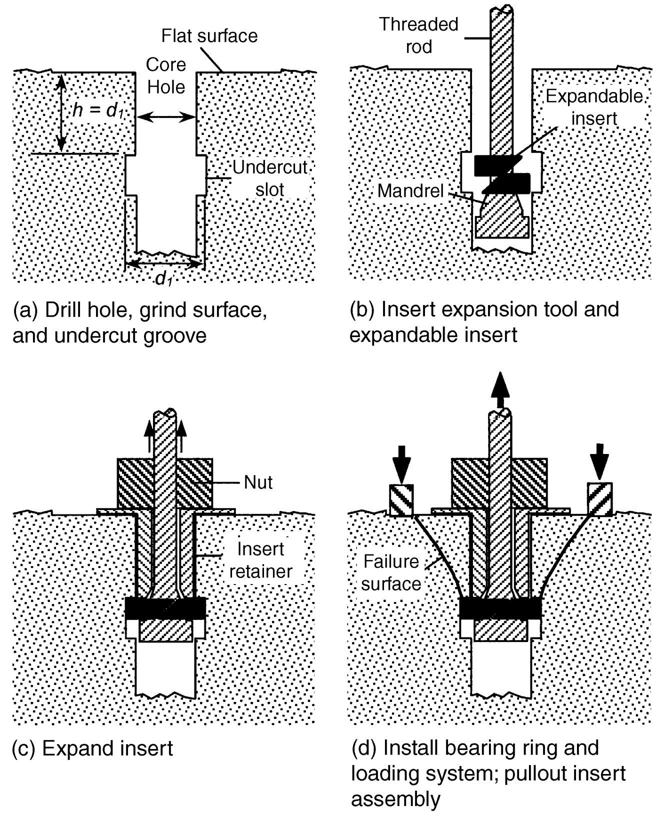

Fig. 2.8—Technique for post-installed pullout test (ASTM C 900).

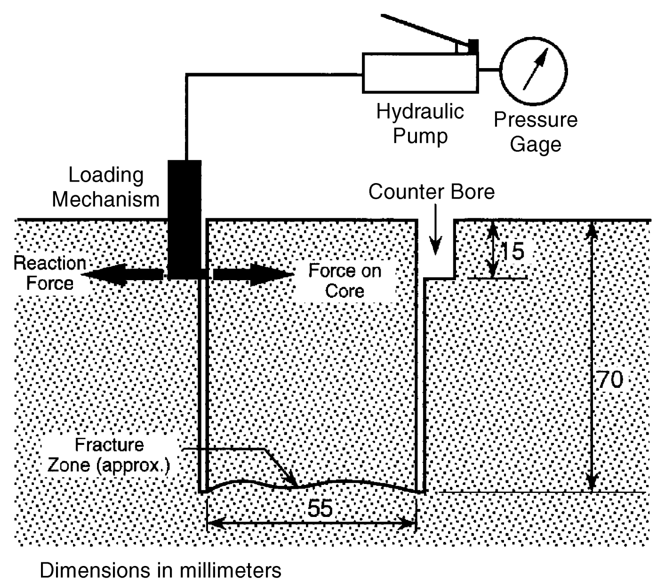

Fig. 2.9—Schematic of break-off test.

2.5—Break-off number (ASTM C 1150)

The break-off test measures the force required to break off a cylindrical core from a larger concrete mass (Johansen 1979). The measured force and a pre-established strength relationship are used to estimate the in-place compressive strength. Standard procedures for using this method are given in ASTM C 1150.

A schematic of the break-off test is shown in Fig. 2.9. For new construction, the core is formed by inserting a cylindrical plastic sleeve into the surface of the fresh concrete. The sleeve includes a ring to form the counter bore for the loading system. The sleeves can also be attached to the sides of formwork and filled during concrete placement (refer to Chapter 5 for attachment method). Alternatively, test specimens can be prepared in hardened concrete by using a special core bit to cut the core and the counter bore. Thus, the break-off test can be used to evaluate concrete in both new and existing construction.

When the in-place compressive strength is to be estimated, the sleeve is removed, and a special loading device is placed into the counter bore. A pump supplies hydraulic fluid to the loading device that applies a horizontal force to the top of the core as shown in Fig. 2.9. The reaction to the horizontal force is provided by a ring that bears against the counter bore. The force on the core is gradually increased until the core ruptures at its base. The hydraulic fluid pressure is monitored with a pressure gage having an indicator to register the maximum pressure achieved during the test. The maximum pressure gage reading in units of bars (1 bar = 0.1 MPa [14.5 psi]) is called the *break-off number* of the concrete.

For new construction, the concrete should be workable to insert sleeves easily into the concrete surface. To reduce interference between the sleeve and coarse aggregate particles, the maximum aggregate size in the concrete is limited to about 1/2 of the sleeve diameter. According to ASTM C 1150, the break-off test is not recommended for concrete having a maximum nominal aggregate size greater than 25 mm (1 in.). There is evidence that variability of the break-off number increases for larger aggregate sizes (refer to Chapter 3). Sleeve

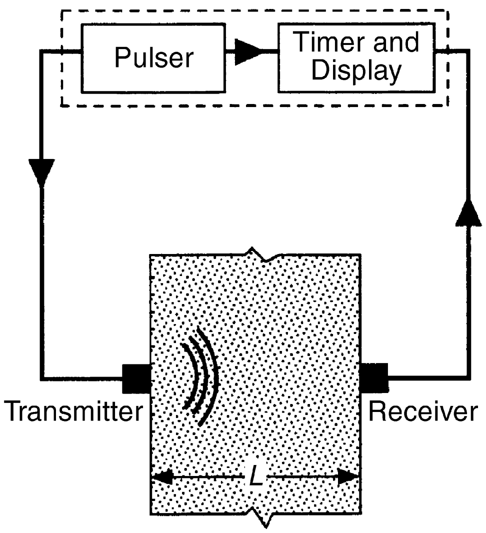

Fig. 2.10—Schematic of apparatus to measure ultrasonic pulse velocity.

insertion should be performed carefully to ensure good consolidation around the sleeve and to minimize disturbance at the base of the formed core. Problems with sleeves floating out of fluid concrete mixtures have been reported (Naik, Salameh, and Hassaballah 1987).

Like the pullout test, the break-off test subjects the concrete to a slowly applied force and measures a static strength property of the concrete. The core is loaded as a cantilever, and the concrete at the base of the core is subject to a combination of bending and shear. In early work (Johansen 1979), the results of the break-off test were reported as the break-off strength, computed as the flexural stress at the base of the core corresponding to the ultimate force applied to the core. This approach required a calibration curve to convert the pressure gage reading to a force, and it assumed that the stress distribution could be calculated by a simple bending formula. In ASTM C 1150, the flexural strength is not computed, and the break-off number (pressure gage reading) is related directly to the compressive strength. This approach simplifies data analysis, but it is still essential to calibrate the testing instrument that will be used on the structure to ensure that the gage readings correspond to the forces applied to the cores.

The computed flexural strength based on the break-off test is about 30% greater than the modulus of rupture obtained by standard beam tests (Johansen 1979; Yener and Chen 1985).

The relationships between break-off strength and compressive strength have been found to be nonlinear (Johansen 1979; Barker and Ramirez 1988), which is in accordance with the usual practice of relating the modulus of rupture of concrete to a power of compressive strength. The relationship between break-off strength and modulus of rupture may be more uncertain than that between break-off strength and compressive strength (Barker and Ramirez 1987).

The break-off test has been used successfully on a variety of construction projects in the Scandinavian countries, including major offshore oil platforms (Carlsson, Eeg, and Jahren 1984). In addition to its use for estimating in-place compressive strength, the method has also been used to evaluate the bond strength between concrete and overlay materials (Dahl-Jorgenson and Johansen 1984).

In summary, the break-off test is based on measuring the force to break off a small core from the concrete mass. It can be used on new and existing construction, depending on the method used to form the core. The concrete is subjected to a well-defined loading condition, and the failure is due to the combination of bending and shearing stresses acting at the base of the core. At the time of this writing (2000), the method had not found widespread use, and ASTM is considering withdrawal of the test method.

2.6—Ultrasonic pulse velocity (ASTM C 597)

The ultrasonic pulse velocity test, as prescribed in ASTM C 597, determines the propagation velocity of a pulse of vibrational energy through a concrete member (Jones 1949; Leslie and Cheesman 1949). The operational principle of modern testing equipment is illustrated in Fig. 2.10. A pulser sends a short-duration, high-voltage signal to a transducer, causing the transducer to vibrate at its resonant frequency. At the start of the electrical pulse, an electronic timer is switched on. The transducer vibrations are transferred to the concrete through a viscous coupling fluid. The vibrational pulse travels through the member and is detected by a receiving transducer coupled to the opposite concrete surface. When the pulse is received, the electronic timer is turned off and the elapsed travel time is displayed. The direct path length between the transducers is divided by the travel time to obtain the pulse velocity through the concrete.

It is also possible, in theory, to measure the attenuation of the ultrasonic pulse as it travels from the transmitter to the receiver (Teodoru 1988). Pulse attenuation is a measure of the intrinsic damping of a material and is related empirically to strength. Pulse attenuation measurements require an oscilloscope to display the signal from the receiving transducer, and care should be used to obtain identical coupling and contact pressure on the transducers at each test point. In addition, the travel path length should be the same.

From the principles of elastic wave propagation, the pulse velocity is proportional to the square root of the elastic modulus (ACI 228.2R). Because the elastic modulus and strength of a given concrete increase with maturity, it follows that pulse velocity may provide a means of estimating strength of concrete, even though there is no direct physical relationship between these two properties. As concrete matures, however, the elastic modulus and compressive strength increase at different rates. At early maturities, the elastic modulus increases at a higher rate than strength, and at later maturities, the elastic modulus increases at a lower rate. As a result, over a wide range of maturity, the relationship between compressive strength and pulse velocity is highly nonlinear. Figure 2.11 shows a typical relationship between compressive strength and pulse velocity. Note that this is only an illustrative example and the actual relationship depends on the specific concrete mixture. At early maturities, a given increase in compressive strength results in a relatively large increase in pulse velocity, while at later maturities the velocity increase is smaller for the same strength increase. For example, a strength increase from 3 to 8 MPa (400 to 1200 psi roughly) may result in a velocity increase from about 2400 to 3040 m/s (7900 to 10,000 ft/s roughly). On the other hand, a strength increase from 25 to 30 MPa (3600 to 4400 psi roughly) may result in a velocity increase of only 3800 to 3920 m/s (12,500 to 12,900 ft/s roughly). Thus, the sensitivity of the pulse velocity as an indicator of change in concrete strength decreases with increasing maturity and strength.

Factors other than concrete strength can affect pulse velocity, and changes in pulse velocity due to these factors may overshadow changes due to strength (Sturrup, Vecchio, and Caratin 1984). For example, the pulse velocity depends strongly on the type and amount of aggregate in the concrete, but the strength of normal-strength concrete (less than about 40 MPa or 6000 psi) is less sensitive to these factors. As the volumetric aggregate content of concrete increases, pulse velocity increases, but the compressive strength may not be affected appreciably (Jones 1962). Another important factor is moisture content. As the moisture content of concrete increases from the air-dry to saturated condition, it is reported that pulse velocity may increase up to 5% (Bungey 1989). If the effects of moisture are not considered, erroneous conclusions may be drawn about in-place strength, especially in mature concrete. The curing process also affects the relationship between pulse velocity and strength, especially when accelerated methods are used (Teodoru 1986).

The amount and orientation of the steel reinforcement will also influence the pulse velocities. Because the pulse velocity through steel is about 40% greater than through concrete, the pulse velocity through a heavily reinforced concrete member may be greater than through one with little reinforcement. This is especially troublesome when reinforcing bars are oriented parallel to the pulse-propagation direction. The pulse may be refracted into the bars and transmitted to the receiver at the pulse velocity in steel. The resulting apparent velocity through the member will be greater than the actual velocity through the concrete. Failure to account for the presence and orientation of reinforcement may lead to incorrect conclusions about concrete strength. Correction factors, such as those discussed in Malhotra (1976) and Bungey (1989), have been proposed, but their accuracy has not been established conclusively.

The measured pulse velocity may also be affected by the presence of cracks or voids along the propagation path from transmitter to receiver. The pulse may be diffracted around the discontinuities, thereby increasing the travel path and travel time. Without additional knowledge about the interior condition of the concrete member, the apparent decrease in pulse velocity could be incorrectly interpreted as a low compressive strength.

In this test method, all of the concrete between the transmitting and receiving transducers affects the travel time. Test results are, therefore, relatively insensitive to the normal heterogeneity of concrete. Consequently, the test method has been found to have an extremely low within-batch coefficient of variation. This does not mean, however, that the strength estimates are necessarily highly reliable.

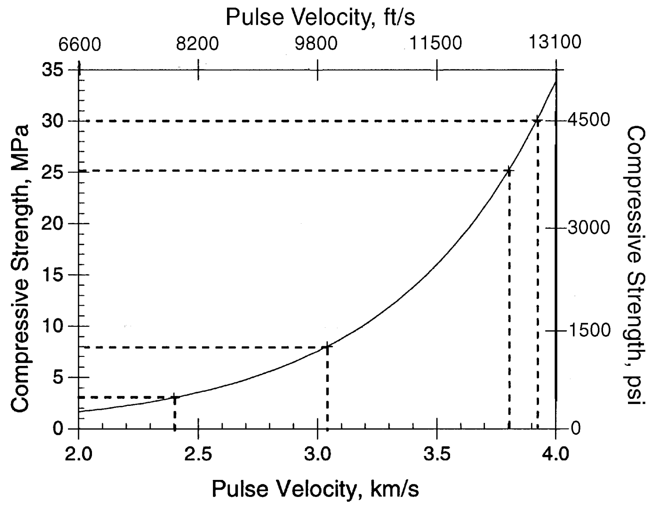

Fig. 2.11—Schematic of typical relationship between pulse velocity and compressive strength of a given concrete mixture.

In summary, pulse velocity can be used to estimate strength in new and existing construction. For a given concrete, a change in pulse velocity is fundamentally related to a change in elastic modulus. Because elastic modulus and strength are not linearly related, pulse velocity is inherently a less-sensitive indicator of concrete strength as strength increases. The amount and type of aggregate has a strong influence on the pulse velocity versus strength relationship, and the in-place pulse velocity is affected by moisture content and the presence of steel reinforcement. Refer to ACI 228.2R for additional discussion of the pulse velocity method.

2.7—Maturity method (ASTM C 1074)

Freshly placed concrete gains strength because of the exothermic chemical reactions between the water and cementitious materials in the mixture. Provided sufficient moisture is present, the rates of the hydration reactions are influenced by the concrete temperature; an increase in temperature causes an increase in the reaction rates. The extent of hydration and, therefore, strength at any age depends on the thermal history of the concrete.

The maturity method is a technique to estimate in-place strength by accounting for the effects of temperature and time on strength development. The thermal history of the concrete and a maturity function are used to calculate a maturity index that quantifies the combined effects of time and temperature. The strength of a particular concrete mixture is expressed as a function of its maturity index by means of a strength-maturity relationship. If samples of the same concrete are subjected to different temperature conditions, the strength-maturity relationship for that concrete and the temperature histories of the samples can be used to estimate their strengths.

The maturity function is a mathematical expression that converts the temperature history of the concrete to a maturity index. Several such functions have been proposed and are reviewed in Malhotra (1971), RILEM (1981), and Malhotra and Carino (1991). The key feature of a maturity function is

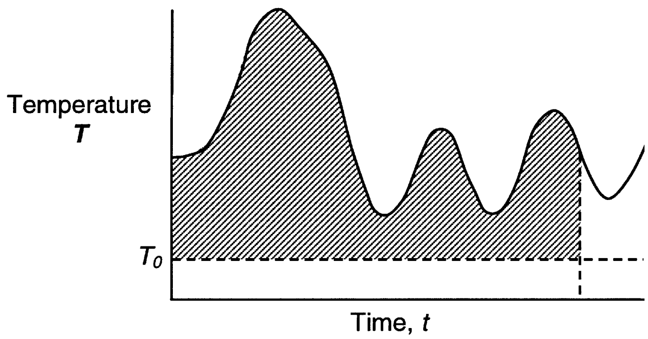

Fig. 2.12—Maturity function based on assumption that the initial rate of strength gain varies linearly with temperature; shaded area is the temperature-time factor (Eq. (2-1)).

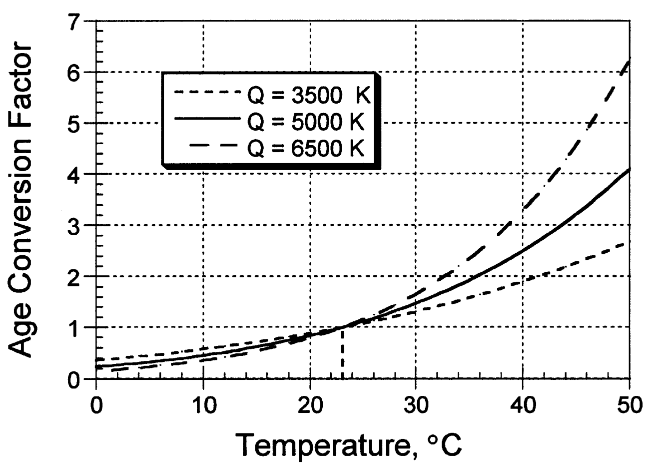

Fig. 2.13—Age conversion factor for different Q*-values and specified temperature of 23 °C based on Eq. (2-2).*

the expression used to represent the influence of temperature on the initial rate of strength development. Two expressions are commonly used. In one approach, it is assumed that the initial rate of strength development is a linear function of temperature, and this leads to the simple maturity function shown in Fig. 2.12. In this case, the maturity index at any age is the area between a datum temperature T_0 and the temperature curve of the concrete. The term temperature-time factor is used for this area and is calculated as follows

$$M(t) = \Sigma(T_a - T_0)\Delta t \tag{2-1}$$

where

$M(t)$ = temperature-time factor at age t, deg-days or deg-h;
Δt = a time interval, days or h;
T_a = average concrete temperature during time interval Δt; and
T_0 = datum temperature.

Traditionally, the datum temperature used in Eq. (2-1) has been taken as the temperature below which strength gain ceases, which has been assumed to be about –10 °C (14 °F). It has been suggested, however, that a single value for the datum temperature is not the most appropriate approach and that the datum temperature should be evaluated for the specific materials in the concrete mixture (Carino 1984). ASTM C 1074 recommends a datum temperature of 0 °C (32 °F) for concrete made with ASTM Type I cement when the concrete temperature is expected to be between 0 and 40 °C (32 and 104 °F). ASTM C 1074 also provides a procedure to determine experimentally the datum temperature for other types of cement and for different ranges of curing temperature.

In the second approach, the maturity function assumes that the initial rate of strength gain varies exponentially with concrete temperature. This exponential function is used to compute an equivalent age of the concrete at some specified temperature as follows

$$t_e = \Sigma e^{-Q\left(\frac{1}{T_a} - \frac{1}{T_s}\right)}\Delta t \tag{2-2}$$

where

t_e = equivalent age at a specified temperature T_s, days or h;
Q = activation energy divided by the gas constant, K (Kelvin);
T_a = average temperature of concrete during time interval Δt, K;
T_s = specified temperature, K; and
Δt = time interval, days or h.

In Eq. (2-2), the exponential function converts a time interval Δt at the actual concrete temperature to an equivalent interval (in terms of strength gain) at the specified temperature. In North America, the specified temperature is typically taken to be 23 °C (296 K), whereas in Europe, 20 °C (293 K) is typically used. The exponential function in Eq. (2-2) can be considered an age conversion factor. To calculate the equivalent age of a concrete mixture, one needs the value of a characteristic known as the activation energy, which depends on the type of cementitious materials (Carino and Tank 1992). The water-cementitious material ratio (*w/cm*) may also influence the activation energy. The Q-value in Eq. (2-2) is the activation energy divided by the gas constant (8.31 joules/[mole·K]). ASTM C 1074 recommends a Q-value of 5000 K for concrete made with ASTM Type I cement and provides procedures for determining the Q-value for other cementitious systems. Figure 2.13 shows how the age conversion factor varies with concrete temperature for different Q-values and a specified temperature of 23 °C. As the Q-value increases, the relationship between age conversion factor and temperature becomes more nonlinear.

To use the maturity method requires establishing the strength-maturity relationship for the concrete that will be used in the structure. The temperature history of the in-place concrete is monitored continuously and the in-place maturity index (temperature-time factor or equivalent age) is computed from this data. The in-place strength can be estimated from the maturity index and strength-maturity relationship. There are instruments that automatically compute the maturity index, but care should be exercised in their use because the value of T_0 or Q used by the instrument may not be applicable to the concrete in the structure. ASTM C 1074 gives the procedure for using the maturity method and provides examples to illustrate calculation of the temperature-time factor or equivalent age from the recorded temperature history of the concrete. ACI 306R illustrates the use of the maturity method to estimate in-place strength during cold-weather concreting operations.

The maturity method is intended for estimating strength development of newly placed concrete. Strength estimates are based on two important assumptions:

1. There is sufficient water for continued hydration; and
2. The concrete in the structure is the same as that used to develop the strength-maturity relationship.

Proper curing procedures (as provided in ACI 308R) will ensure that the first condition is satisfied. The second condition requires additional confirmation that the concrete in the structure has the correct strength potential. This can be achieved by performing accelerated strength tests on concrete sampled from the structure or by performing other in-place tests that give positive indications of the strength level. Such verification is essential when estimates of in-place strength are used for timing critical operations such as formwork removal or application of post-tensioning.

In summary, the maturity method is used to estimate strength development in construction. Because the method relies only on measurement of the in-place temperature, other information is required to ensure that the in-place concrete has the intended mixture proportions. The correct datum temperature or *Q*-value is required to improve the accuracy of the strength estimation at early ages.

2.8—Cast-in-place cylinders (ASTM C 873)

This is a technique for obtaining cylindrical concrete specimens from newly cast slabs without drilling cores. The method is described in ASTM C 873 and involves using a mold, as illustrated in Fig. 2.14. The outer sleeve is nailed to the formwork and is used to support a cylindrical mold. The sleeve can be adjusted for different slab thicknesses. The mold is filled when the slab is cast, and the concrete in the mold is allowed to cure with the slab. The objective of the technique is to obtain a test specimen that has been subjected to the same thermal history as the concrete in the structure. To determine the in-place strength, the mold is removed from the sleeve and stripped from the concrete cylinder. The cylinder is capped and tested in compression. For cases in which the length-diameter ratio of the cylinders is less than two, the measured compressive strengths should be corrected by the factors in ASTM C 42/C 42M.

In summary, because the cast-in-place cylinder technique involves a compressive strength test of a cylindrical specimen, a strength relationship is not required. To obtain an accurate estimate of the in-place strength, care is required to ensure that the concrete in the mold is properly consolidated in accordance with ASTM C 873. There will always be some uncertainty in the actual in-place strength because the length-diameter ratio correction factors are inherently approximate.

2.9—Strength limitations

Most test procedures have some limitations regarding the applicable strength range. In some cases, the test apparatus has not been designed for testing low-strength or high-strength concrete, and in other cases there is limited experience in using the methods to test high-strength concrete. The useful strength ranges for the various methods are summarized in Table 2.1. These ranges are approximate and may be extended if the user can show a reliable strength relationship at higher strengths.

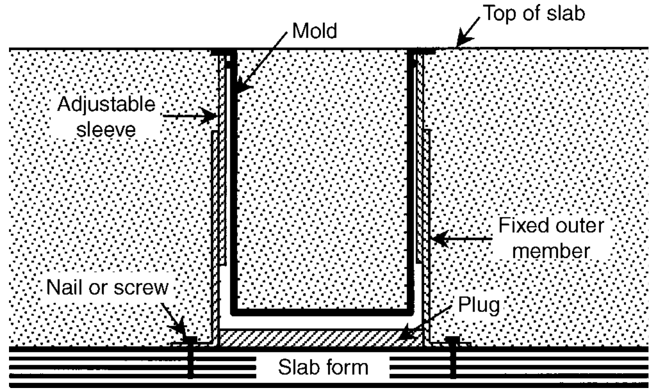

Fig. 2.14—Special mold and support hardware to obtain cast-in-place concrete specimen.

Table 2.1—Useful compressive strength ranges for in-place test methods

Test method	Range of compressive strength*	
	MPa	psi
Rebound number	10 to 40	1500 to 6000
Probe penetration	10 to 120	1500 to 17,000†
Pin penetration	3 to 30	500 to 4000
Pullout	2 to 130‡	300 to 19,000‡
Ultrasonic pulse velocity	1 to 70	100 to 10,000
Break-off	3 to 50	500 to 7000
Maturity	No limit	
Cast-in-place cylinder	No limit	

*Higher strengths may be tested if satisfactory data are presented for the test method and equipment to be used.

†For strengths above 40 MPa (6000 psi), special probes are required.

‡For strengths above 55 MPa (8000 psi), special high-strength bolts are required to extract pullout inserts.

2.10—Combined methods

The term combined method refers to the use of two or more in-place test methods to estimate concrete strength. By combining results from more than one in-place test, a multi-variable correlation can be established to estimate strength. Combined methods are reported to increase the reliability of the estimated strength. The underlying concept is that if the two methods are influenced in different ways by the same factor, their combined use results in a canceling effect that improves the accuracy of the estimated strength. For example, an increase in moisture content increases pulse velocity but decreases the rebound number.

Combined methods were developed and have been used in Eastern Europe to evaluate concrete strength in existing construction or in precast elements (Făcaoăru 1970, 1984; Teodoru 1986, 1988). Combinations, such as pulse velocity

Table 2.2—Relative performance of in-place tests

Test method	ASTM Standard	Accuracy*		Ease of use*
		New construction	Existing construction	
Rebound number	C 805	+	+	++
Penetration resistance	C 803/C 803M	+	+	++
Pullout	C 900	++	++	+
Break-off	C 1150	++	++	+
Pulse velocity	C 597	++	+	+
Maturity	C 1074	++†	N/A	+
Cast-in-place cylinder	C 873	++	N/A	+

*A test method with a ++ results in a more accurate strength estimate or is easier to use than a method with a +. N/A indicates that the method is not applicable to existing construction.
†Requires verification by other tests.

and rebound number (or pulse velocity, rebound number, and pulse attenuation), have resulted in strength relationships with higher correlation coefficients than when these methods are used individually. The improvements, however, have usually only been marginal (Tanigawa, Baba, and Mori 1984; Samarin and Dhir 1984; Samarin and Meynink 1981; Teodoru 1988).

Another approach is to use the maturity method in combination with another in-place test that measures an actual strength property of the concrete, such as a pullout test or break-off test. The maturity method is used to determine when the in-place concrete should have reached the required strength, then the other test method is carried out to verify that the strength has been achieved. This approach is especially beneficial when in-place tests involve embedded hardware. The use of the maturity method to determine when the other test should be performed may avoid premature testing. In addition, maturity readings can be used to assess the significance of lower or higher than expected in-place test results (Soutsos et al. 2000).

It is emphasized that combining methods is not an end in itself. A combined method should be used in those cases where it is the most economical way to obtain a reliable estimate of concrete strength (Leshchinsky 1991). In North America, the use of combined methods has aroused little interest among researchers and practitioners. There have been no efforts to develop ASTM standards for their use.

2.11—Summary

Methods that can be used to estimate the in-place strength of concrete have been reviewed. While other procedures have been proposed (Malhotra 1976; Bungey 1989; Malhotra and Carino 1991), the discussion has been limited to those techniques that have been standardized as ASTM test methods.

Table 2.2 summarizes the relative performance of the in-place tests discussed in this report in terms of accuracy of estimated strength and ease of use. The table also indicates which methods are applicable to new construction and which are applicable to existing construction. Generally, those methods requiring embedment of hardware are limited to use in new construction. In general, those techniques that involve preplanning of test locations and embedment of hardware require more effort to use. Those methods, however, also tend to give more reliable strength estimates. The user should consider the relative importance of accuracy and ease of use when selecting the most appropriate in-place testing system for a particular application.

In-place tests provide alternatives to core tests for estimating the strength of concrete in a structure or can supplement the data obtained from a limited number of cores. These methods are based on measuring a concrete property that has some relationship to strength. The accuracy of these methods is, in part, determined by the degree of correlation between strength and the physical quantity measured by the in-place test. For proper evaluation of test results, the user should be aware of those factors other than concrete strength that can affect the test results. Additional fundamental research is needed to improve the understanding of how these methods are related to concrete strength and how the test results are affected by factors other than strength.

An essential step for using these methods to estimate the in-place strength is the development of a relationship between strength and the quantity measured by the in-place test. The data acquired for developing the strength relationship provide valuable information on the reliability of the estimates. Subsequent chapters of this report discuss the statistical characteristics of the tests, methods for developing strength relationships, planning of in-place tests, and interpretation of the results. The final chapter deals with the use of in-place tests for acceptance of concrete.

CHAPTER 3—STATISTICAL CHARACTERISTICS OF TEST RESULTS

3.1—Need for statistical analysis

In designing a structure to safely resist the expected loads, the engineer uses the specified compressive strength f_c' of the concrete. The strength of the concrete in a structure is variable and, as indicated in ACI 214, the specified compressive strength is approximately the strength that is expected to be exceeded with about 90% probability (10% of tests are expected to fall below the specified strength.). To ensure that this condition is satisfied, the concrete supplied for the structure must have an average standard-cured cylinder strength more than f_c' as specified in Chapter 5 of ACI 318-02. When the strength of concrete in a structure is in question because of low standard-cured cylinder strengths or suspected curing deficiencies, ACI 318 states that the concrete is structurally adequate if the in-place strength, as represented by the average strength of three cores, is not less than $0.85f_c'$ (refer also to Chapter 7).

In assessing the ability of a partially completed structure to resist construction loads, the committee believes it is reasonable that the tenth-percentile in-place compressive strength (strength exceeded with 90% probability) should be equal to at least 0.85 of the required compressive strength at the time of application of the construction loads. The required strength means the compressive strength used in computing the nominal load resistance of structural elements. In-place tests can be used to estimate the tenth-percentile strength with a high degree of confidence only if test data are subjected to statistical analysis.

The use of the tenth-percentile strength as the in-place strength that can be relied upon to resist construction loads is considered reasonable by users of in-place tests. The critical nature of construction operations in partially completed structures, the sensitivity of early-age strength on the previous thermal history of the concrete, and the general lack of careful consideration of construction loading during the design of a structure, dictate the use of a conservative procedure for evaluating in-place test results. For situations where the consequences of a failure may not be serious, the estimated mean strength may be an acceptable measure to assess the adequacy of the in-place strength for proceeding with construction operations. Examples of such situations would include slabs-on-ground, pavements, and some repairs. Inadequate strength at the time of a proposed construction operation can usually be remedied by simply providing for additional curing before proceeding with the operation.

In-place tests may also be used to evaluate the strength of an existing structure. They are often used to answer questions that arise because of low strengths of standard-cured cylinders. Failure to meet specified acceptance criteria can result in severe penalties for the builder. In such cases, the use of the tenth-percentile strength as the reliable strength level to resist design loads is not the appropriate technique for analyzing in-place test data. The existing ACI 318 criteria for the acceptance of concrete strength in an existing structure are based on testing cores. Based on ACI 318, if the average compressive strength of three cores exceeds 85% of the specified compressive strength and no single core strength is less than 75% of the specified strength, the concrete strength is deemed to be acceptable. There are, however, no analogous acceptance criteria for the estimated in-place compressive strength based on in-place tests. Chapter 7 discusses how in-place testing could be used for acceptance of concrete.

To arrive at a reliable estimate of the in-place compressive strength by using in-place tests, one must account for the following primary sources of uncertainty:

1. The average value of the in-place test results;

2. The relationship between compressive strength and the in-place test results; and

3. The inherent variability of the in-place compressive strength.

The first source of uncertainty is associated with the inherent variability (repeatability) of the test method. This subject is discussed in the remainder of this chapter.

3.2—Repeatability of test results

The uncertainty of the average value of the in-place test results is a function of the standard deviation of the results and the number of tests. The standard deviation is in turn a function of the repeatability of the test and the variability of the concrete in the structure.

In this Report, repeatability means the standard deviation or coefficient of variation of repeated tests by the same operator on the same material. This is often called the within-

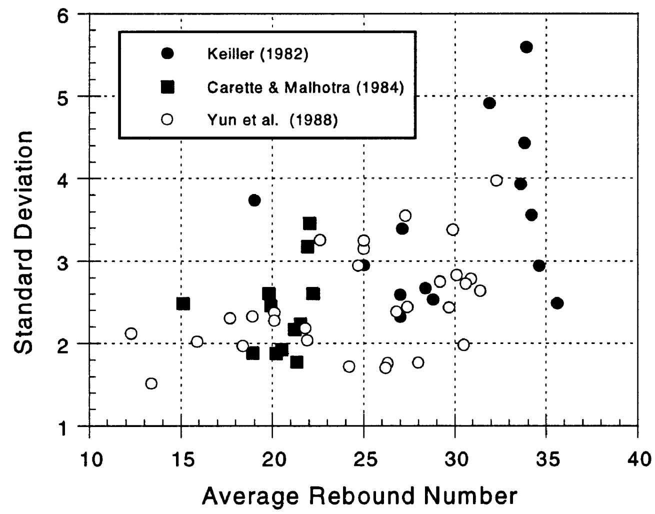

Fig. 3.1—Within-test standard deviation as a function of average rebound number.

test variation and shows the inherent scatter associated with a particular test method.

Data on the repeatability of some in-place tests are provided in the precision statements of the ASTM standards governing the tests. Some information on the repeatability of other tests may be found in published reports. Unfortunately, most published data deal with correlations with standard strength tests, rather than with repeatability. As will be seen, conclusions about repeatability are often in conflict because of differences in experiment designs or in data analysis.

3.2.1 *Rebound number*—The precision statement of ASTM C 805 states that the within-test standard deviation of the rebound hammer test is 2.5 rebound numbers. Teodoru* reported an average standard deviation of 3.75, for average rebound numbers ranging from 20 to 40, and the standard deviation was independent of the average rebound number.

The results of three studies that evaluated the performance of various in-place tests provide additional insight into the repeatability of the rebound number test. Keiller (1982) used eight different mixtures and took 12 replicate rebound readings at ages of 7 and 28 days. Carette and Malhotra (1984) used four mixtures and took 20 replicate readings at ages of 1, 2, and 3 days. Yun et al. (1988) used five mixtures of concrete and took 15 replicate readings at ages ranging from 1 to 91 days.

Figure 3.1 shows the standard deviations of the rebound numbers as a function of the average rebound number. The data from the three studies appear to follow the same pattern. In the study by Carette and Malhotra (1984), the average maximum rebound number ranged from 15 to 22 and the average standard deviation was 2.4. In the study by Keiller (1982), the average rebound number ranged from 18 to 35, and the average standard deviation was 3.4. In the work by Yun et al. (1988), the range in average rebound number was 12 to 32, and the average standard deviation was 2.5.

*Teodoru, G. V., 1970, "Quleques Aspects du Contrôle Statistique de la Qualité du Béton Basé sur le Essais Nondestructifs," meeting of RILEM NDT Committee, Slough, England.

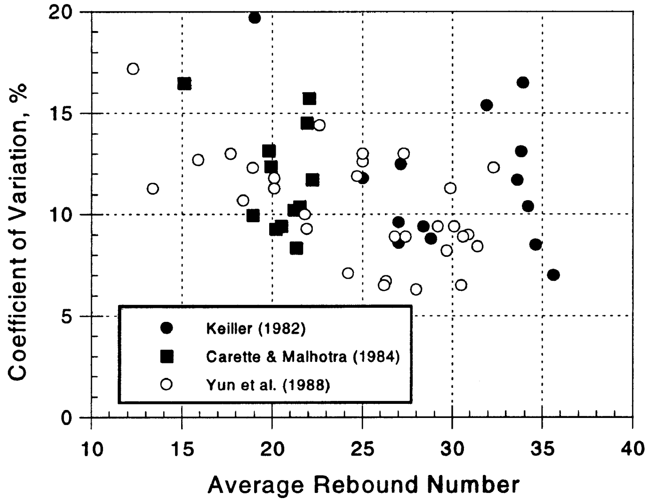

Fig. 3.2—Within-test coefficient of variation as a function of average rebound number.

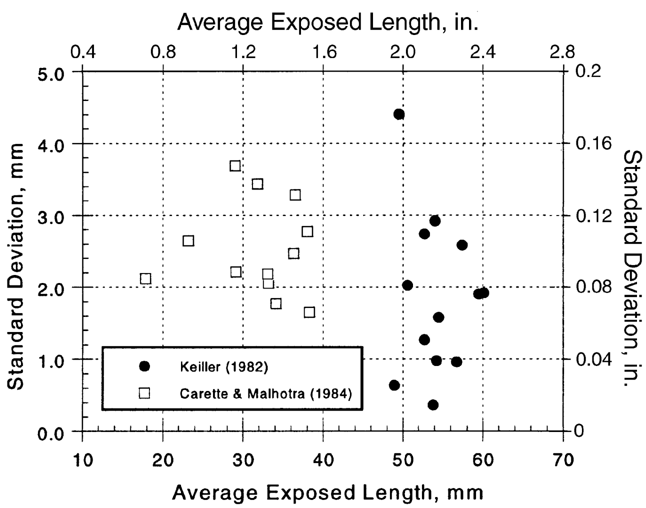

Fig. 3.3—Within-test standard deviation as a function of average exposed length of probes.

Examination of Figure 3.1 shows that there may be a trend of increasing standard deviation with increasing average rebound number, in which case the coefficient of variation is a better measure of repeatability. Figure 3.2 shows the coefficients of variation plotted as functions of average rebound number. There does not appear to be any trend with increasing rebound number. In contrast, Leshchinsky et al. (1990) found that the coefficient of variation and its variability tended to decrease with increasing concrete strength. The average coefficients of variation from the studies by Carette and Malhotra (1984) and by Keiller (1982) have equal values of 11.9, while the average value from the study by Yun et al. (1988) was 10.4 and Teodoru* reported a value of 10.2%.

In Figure 3.2, the coefficients of variation are not constant. It should be realized, however, that the values are based on

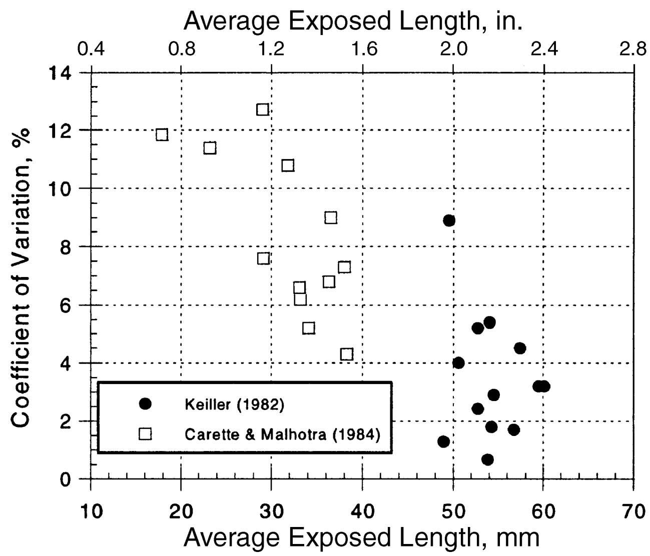

Fig. 3.4—Within-test coefficient of variation as a function of average exposed length of probes.

sample estimates of the true averages and standard deviations. With finite sample sizes there will be variations in these estimates, and a random variation in the computed coefficient of variation is expected; although, the true coefficient of variation may be constant. Thus, it appears that the repeatability of the rebound number technique may be described by a constant coefficient of variation, which has an average value of about 10%.

3.2.2 *Penetration resistance*—The precision statement in ASTM C 803/C 803M states that, for the probe penetration test, the within-test standard deviations of exposed probe length for three replicate tests are:

Maximum Size of Aggregate	Standard Deviation
Mortar—4.75 mm (No. 4)	2.0 mm (0.08 in.)
Concrete—25 mm (1 in.)	2.5 mm (0.10 in.)
Concrete—50 mm (2 in.)	3.6 mm (0.14 in.)

The data reported by Carette and Malhotra (1984) and Keiller (1982), which include concrete strengths in the range of 10 to 50 MPa (1500 to 7000 psi), give additional insight into the underlying measure of repeatability for this test. Figure 3.3 shows the standard deviations of the exposed length of the probes as a function of the average exposed length. The values from Carette and Malhotra (1984) are based on the average of six probes, while Keiller's (1982) results are based on three probes. Except for one outlying point, there is a trend of decreasing within-test variability with increasing exposed length. In Fig. 3.4, the coefficients of variation of exposed length are shown as a function of the average exposed length. The decreasing trend with increasing concrete strength is more pronounced than in Fig. 3.3. Thus, the repeatability of the exposed length is described neither by a constant standard deviation nor a constant coefficient of variation.

The customary practice is to measure the exposed length of the probes, but concrete strength has a direct effect on the

*Teodoru, G. V., 1968, "Le Contrôle Statistique de la Qualité du Béton dans les Usines de Précoulage à l'aide des Essais Nondestructifs," Report to RILEM Committee NDT, Varna, Bulgaria.

depth of penetration. A more logical approach is to express the coefficient of variation in terms of depth of penetration. Figure 3.5 shows the coefficient of variation of the penetration depth as a function of average penetration. In this case, there is no clear trend with increasing penetration. The higher scatter of the values from Keiller's (1982) tests may be due to their smaller sample size compared with the tests of Carette and Malhotra (1984). Note that the standard deviation has the same value whether exposed length or penetration depth is used. The coefficient of variation, however, depends on whether the standard deviation is divided by average exposed length or average penetration depth.

Hence, it appears that a constant coefficient of variation of the penetration depth can be used to describe the within-test variability of the probe penetration test. The work by Carette and Malhotra (1984) is the first known study that uses this method for defining the repeatability of the penetration test. Other test data using the probe penetration system, however, can be manipulated to yield the coefficient of variation of penetration depth provided two of these three quantities are given: average exposed length, standard deviation, or coefficient of variation of exposed length. Using the data given in Table 6 of Malhotra's 1976 review, the following values for average coefficients of variation for depth of penetration have been calculated

Maximum size aggregate		Coefficient of variation of penetration depth, %
mm	in.	
50	2	14
25	1	8.6
19	3/4	3.5, 4.7, and 5.6

In the study by Carette and Malhotra (1984), the maximum aggregate size was 19 mm (3/4 in.) and the average coefficient of variation was 5.4%, whereas in the study by Keiller (1982), it was 7.8% for the same maximum size aggregate. Other work (Swamy and Al-Hamad 1984) used 10 mm (3/8 in.) maximum size aggregate, and the coefficients of variation ranged between 2.7 and 7%. For commonly used 19 mm (3/4 in.) aggregate, it is concluded that a coefficient of variation of 5% is reasonable.

There are limited data on the repeatability of the pin penetration test. Nasser and Al-Manaseer (1987b) reported an average coefficient of variation of about 5% for replicate tests on 100 mm (4 in.) thick slab specimens and on the bottom surfaces of 150 x 300 mm (6 x 12 in.) cylinders. The variability was based on the best five of seven readings (the lowest and highest were deleted), and the concrete strength varied from about 3.5 to 25 MPa (500 to 3500 psi). In another study (Carino and Tank 1989), eight replicate pin tests were performed at the midheight of 100 x 200 mm (4 x 8 in.) cylinders. The compressive strengths ranged from about 7 to 40 MPa (1000 to 5800 psi). Each set of replicate pin tests was analyzed for outliers due to penetrations into large air voids or coarse aggregate particles. On average, two of the eight readings were discarded. Figure 3.6 shows the standard deviations of the valid penetration values plotted as

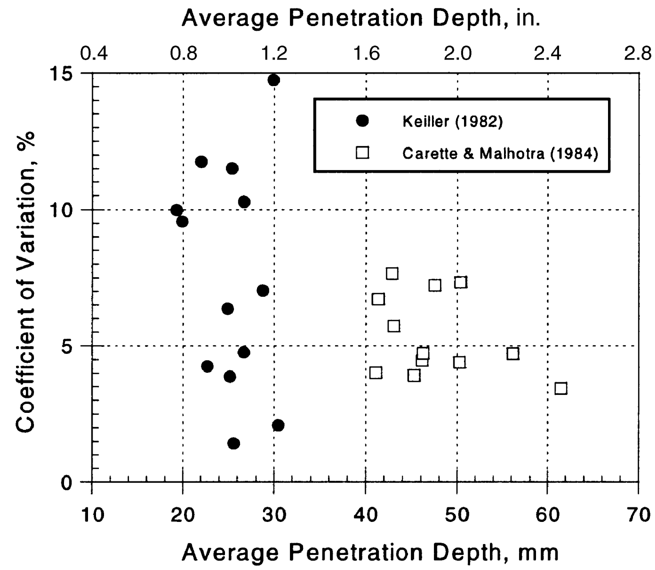

Fig. 3.5—Within-test coefficient of variation as a function of average penetration of probes.

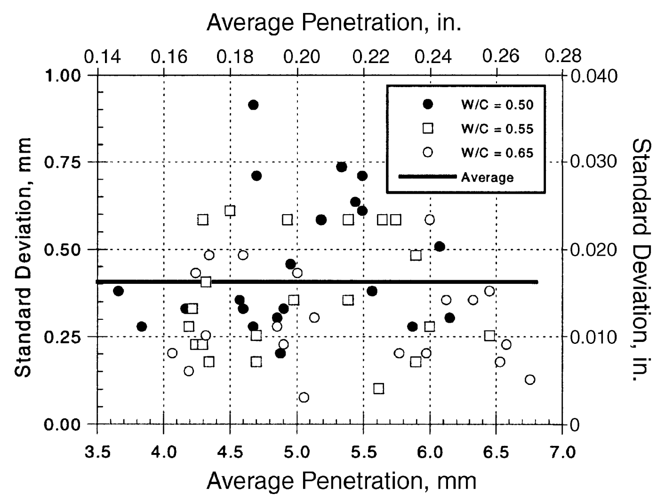

Fig. 3.6—Standard deviation of pin penetration tests on 100 x 200 m (4 x 8 in.) cylinders (Carino and Tank 1989).

a function of the average penetration. (Note that a high penetration corresponds to low concrete strength.) There is no clear trend between the standard deviation and the average penetration. The average standard deviation is 0.41 mm (0.016 in.), which is the value adopted in the precision statement of ASTM C 803/C 803M. To compare with the variability reported by Nasser and Al-Manaseer (1987b), the results in Fig. 3.6 are presented in terms of coefficient of variation in Fig. 3.7. The average coefficient of variation is 7.4%.

Additional data are needed on the repeatability of the pin penetration test. Based on available information, a coefficient of variation of 8% is recommended for planning pin penetration tests.

3.2.3 *Pullout test*—ASTM C 900 states that the average within-test coefficient of variation is 8% for cast-in-place pullout tests with embedments of about 25 mm (1 in.) in concrete with nominal maximum aggregate size of 19 mm (3/4 in.). This value is based on the data summarized as

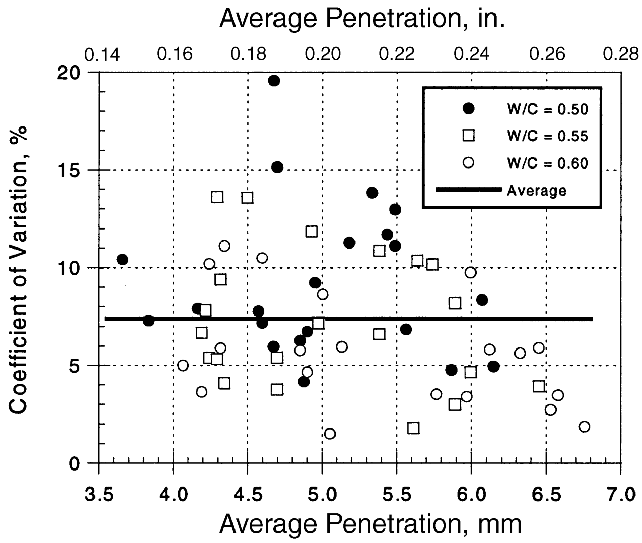

Fig. 3.7—Coefficient of variation of pin penetration tests on 100 x 200 mm (4 x 8 in.) cylinders (Carino and Tank 1989).

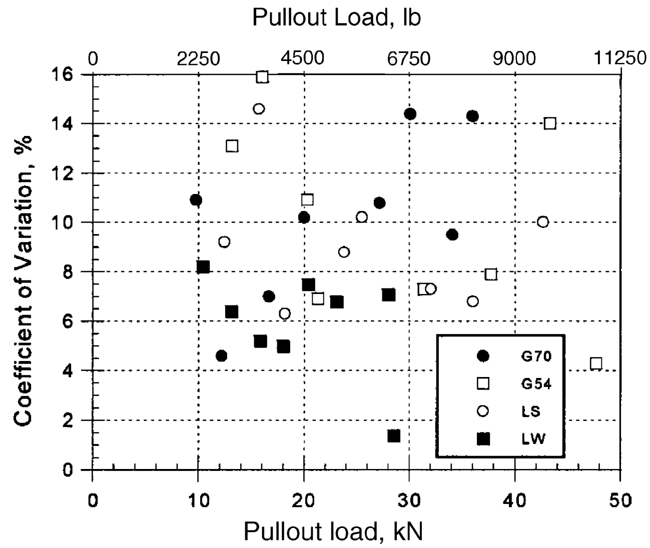

Fig. 3.9—Within-test coefficient of variation as a function of pullout load (Stone, Carino, and Reeve 1986).

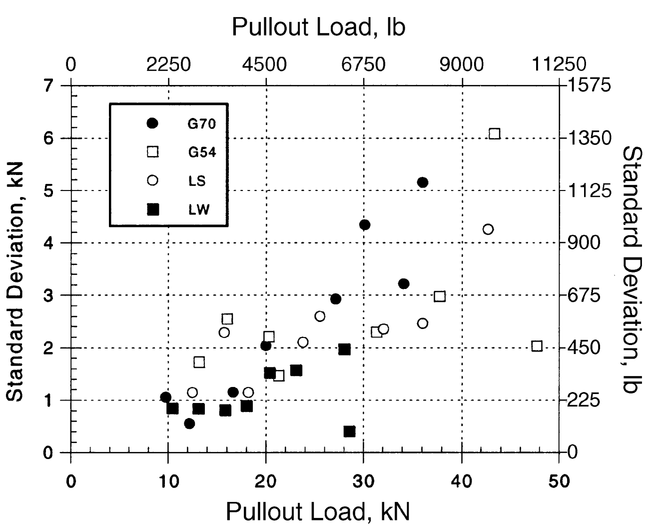

Fig. 3.8—Within-test standard deviation as a function of pullout load (Stone, Carino, and Reeve 1986).

follows. A similar within-test variability is suggested for post-installed tests of the same geometry (Petersen 1997).

Stone, Carino, and Reeve (1986) examined whether standard deviation or coefficient of variation is the best measure of repeatability. Four test series were performed. Three of them used a 70 degree apex angle but different aggregate types: siliceous river gravel, crushed limestone, and expanded low-density (lightweight) shale. The fourth series was for a 54 degree angle with river-gravel aggregate. These test series are identified as G70, LS, LW, and G54 in Fig. 3.8 and 3.9. The embedment depth was about 25 mm (1 in.), and compressive strength of concrete ranged from about 10 to 40 MPa (1500 to 6000 psi). Figure 3.8 shows the standard deviation, using 11 replications, as a function of the average pullout load. It is seen that there is a tendency for the standard deviation to increase with increasing pullout load. Figure 3.9 shows the coefficient of variation as a function of the average pullout load. In this case, there is no trend between the two quantities. Thus, it may be concluded that the coefficient of variation should be used as a measure of the repeatability of the pullout test.

Table 3.1 gives the reported coefficients of variation from different laboratory studies of the pullout test. Besides these data, the work of Krenchel and Petersen* summarizes the repeatability obtained in 24 correlation testing programs involving an insert with a 25 mm (1 in.) embedment and a 62-degree apex angle. The reported coefficients of variation ranged from 4.1 to 15.2%, with an average of 8%. The tests reported in Table 3.1 and by Krenchel and Petersen involved different test geometries and different types and sizes of coarse aggregate. In addition, the geometry of the specimens containing the embedded inserts was different, with cylinders, cubes, beams, and slabs being common shapes. Because of these testing differences, it is difficult to draw firm conclusions about the repeatability of the pullout test.

Table 3.2 summarizes the coefficients of variation obtained in a study by Stone and Giza (1985) designed to examine the effects of different variables on test repeatability. The column labeled sample size shows the number of groups of tests, with each group containing 11 replications. For the conditions studied, it was found that embedment depth and apex angle did not greatly affect repeatability. On the other hand, the maximum nominal aggregate size appeared to have some affect, with the 19 mm (3/4 in.) aggregate resulting in slightly greater variability than the smaller aggregates. The aggregate type also appears to be important. For tests with low-density aggregate, the variability was lower than for tests with normal-density aggregates. In this study, companion mortar specimens were also tested and the coefficients of variation varied between 2.8 and 10.6%,

*Krenchel, H., and Petersen, C. G., 1984, "In-Place Testing wih Lok-Test: Ten Years Experience," Presentation at International Conference on In Situ/Nondestructive Testing of Concrete, Ottawa, Ontario, Canada.

Table 3.1—Summary of within-test coefficient of variation of pullout test

Reference	Apex angle, degrees	Embedment depth, mm	Embedment depth, in.	Maximum aggregate size, mm	Maximum aggregate size, in.	Aggregate type	No. of replicate specimens	Coefficient of variation, % Range	Coefficient of variation, % Average
Malhotra and Carette (1980)	67	50	2	25	1	Gravel	2	0.9 to 14.3	5.3
Malhotra (1975)	67	50	2	6	1/4	Limestone	3	2.3 to 6.3	3.9
Bickley (1982b)	62	25	1	10	3/8	?	8	3.2 to 5.3	4.1
Khoo (1984)	70	25	1	19	3/4	Granite	6	1.9 to 12.3	6.9
Carette and Malhotra (1984)	67	50	2	19	3/4	Limestone	4	1.9 to 11.8	7.1
	62	25	1	19	3/4	Limestone	10	5.2 to 14.9	8.5
Keiller (1982)	62	25	1	19	3/4	Limestone	6	7.4 to 31	14.8
Stone, Carino, and Reeve (1986)	70	25	1	19	3/4	Gravel	11	4.6 to 14.4	10.2
	70	25	1	19	3/4	Limestone	11	6.3 to 14.6	9.2
	70	25	1	19	3/4	Low density	11	1.4 to 8.2	6.0
	54	25	1	19	3/4	Gravel	11	4.3 to 15.9	10.0
Bocca (1984)	67	30	1.2	13	1/2	?	24	2.8 to 6.1	4.3

Table 3.2—Summary of results from investigation of pullout test (Stone and Giza 1985)

Test series	Apex angle, degrees	Embedment depth, mm	Embedment depth, in.	Maximum aggregate size, mm	Maximum aggregate size, in.	Aggregate type	No. of replicate specimens*	Coefficient of variation, % Range	Coefficient of variation, % Average
Apex angle	30	25	0.98	19	3/4	Gravel	2 x 11	9.1 to 11.4	10.3
	46	25	0.98	19	3/4	Gravel	4 x 11	5.6 to 18.7	11.1
	54	25	0.98	19	3/4	Gravel	2 x 11	6.3 to 6.7	6.5
	58	25	0.98	19	3/4	Gravel	2 x 11	8.6 to 10.0	9.3
	62	25	0.98	19	3/4	Gravel	2 x 11	7.5 to 9.6	8.6
	70	25	0.98	19	3/4	Gravel	4 x 11	8.0 to 10.1	8.8
	86	25	0.98	19	3/4	Gravel	2 x 11	9.0 to 10.8	9.9
Embedment	58	12	0.47	19	3/4	Gravel	1 x 11	—	12.9
	58	20	0.78	19	3/4	Gravel	2 x 11	7.7 to 14.0	10.9
	58	23	0.91	19	3/4	Gravel	2 x 11	6.5 to 6.7	6.6
	58	25	0.98	19	3/4	Gravel	2 x 11	8.8 to 10.7	9.8
	58	27	1.06	19	3/4	Gravel	2 x 11	9.1 to 11.1	10.1
	58	43	1.69	19	3/4	Gravel	2 x 11	11.5 to 11.9	11.7
Aggregate size	70	25	0.98	6	1/4	Gravel	2 x 11	6.5 to 7.0	6.8
	70	25	0.98	10	3/8	Gravel	5 x 11	4.9 to 6.5	6.0
	70	25	0.98	13	1/2	Gravel	5 x 11	3.3 to 10.6	6.7
	70	25	0.98	19	3/4	Gravel	4 x 11	8.0 to 10.1	8.8
Aggregate type	70	25	0.98	19	3/4	Low density	2 x 11	5.6 to 5.7	5.7
	70	25	0.98	19	3/4	Gravel	4 x 11	8.0 to 10.1	8.8
	70	25	0.98	19	3/4	Crushed gneiss	2 x 11	7.2 to 16.8	12.0
	70	25	0.98	19	3/4	Porous limestone	2 x 11	7.7 to 10.9	9.3

*The term, "2 x 11" indicates two groups of 11 replicates per group.

with an average value of 6.2%. Thus, the repeatability with low-density aggregate is similar to that obtained with mortar.

Experimental evidence suggests that the variability of the pullout test should be affected by the ratio of the mortar strength to coarse-aggregate strength and by the maximum aggregate size. As aggregate strength and mortar strength become similar, repeatability is improved. This explains why the tests results by Stone and Giza (1985) with low-density aggregate were similar to test results with plain mortar. Results from Bocca (1984), summarized in Table 3.2, also lend support to this pattern of behavior. In this case, high-strength concrete was used and the mortar strength approached that of the coarse aggregate. This condition, and the use of small maximum aggregate size, may explain why the coefficients of variation were lower than typically obtained with similar pullout test configurations on lower strength concrete.

In summary, a variety of test data has been accumulated on the repeatability of the pullout tests. Differences in results are often due to differences in materials and testing conditions. In general, it appears that an average within-test coefficient of variation of 8% is typical for pullout tests conforming with the requirements of ASTM C 900 and with embedment depths of about 25 mm (1 in.). The actual value expected in any particular situation will be affected primarily by the nature of the coarse aggregate, as discussed in previous paragraphs.

3.2.4 *Break-off test*—ASTM C 1150 states that the average coefficient of variation is 9% for break-off tests in concrete with nominal maximum aggregate size of 19 and

Table 3.3—Within-test coefficient of variation of break-off test

Reference	Maximum aggregate size		Coarse aggregate type	Replicate tests	Coefficient of variation, %	
	mm	in.			Range	Average
Johansen (1976)	25	1	Unknown	5	Not available	9.7
	25	1	Gravel	5		8.7
	38	1-1/2	Unknown	5		12.3
	Sand	Sand	None	5		4.1
Keiller (1982)	19	3/4	Crushed stone	6	4.2 to 15.8	9.4
	19	3/4	Gravel	6		8.2
Nishikawa (1983)	13	1/2	Gravel	10	5.1 to 13.7	9.9
	13	1/2	Crushed stone	10	*	8.0
	10	3/8	Gravel	10	*	4.7
	19	3/4	Gravel	10	*	9.0
	25	1	Gravel	10	*	13.3
Naik et al. (1987) (sleeves)	19	3/4	Crushed stone	5,6	3.5 to 11.7	6.8
	19	3/4	Gravel	5,6	3.0 to 17.9	10.6
Naik et al. (1987) (cores)	19	3/4	Crushed stone	5	2.8 to 11.6	6.2
	19	3/4	Gravel	5	3.6 to 12.9	8.3
Barker and Ramirez (1988)	13	1/2	Gravel	4	2.4 to 13.9	6.0
	13	1/2	Crushed stone	4	2.9 to 7.2	4.8
	25	1	Gravel	4	3.8 to 14.3	6.8

*Only one test series.

25 mm (3/4 and 1 in.). This value is based on the data summarized as follows.

Failure during the break-off test is due to the formation of a fracture surface at the base of the core (refer to Fig. 2.9). The crack passes through the mortar and, usually, around coarse-aggregate particles at the base of the core. The force required to break off the core is influenced by the particular arrangement of aggregate particles within the failure region. Because of the small size of the fracture surface and the heterogeneous nature of concrete, the distribution of aggregate particles will be different at each test location. Hence, one would expect the within-test variability of the break-off test to be higher than that of other standard strength tests that involve larger test specimens. One would also expect that maximum aggregate size and aggregate shape might affect the variability.

The developer of the break-off test reported a within-test coefficient of variation of about 9% (Johansen 1979). Other investigators have generally confirmed this value. Table 3.3 summarizes some published data on within-test variability of the break-off test. The results have been grouped according to nominal maximum aggregate size and aggregate type (river gravel and crushed stone). The numbers of replicate tests are also listed. The following observations can be made:

- The variability tends to increase with increasing maximum aggregate size; and
- The variability in concrete made with river gravel tends to be higher than in concrete made with crushed stone.

In Table 3.3, the variability reported by Barker and Ramirez (1988) is lower than that reported by others. Part of the difference may be due to the experimental technique. In most of the research, break-off tests have been performed on slab specimens. Barker and Ramirez, however, inserted the plastic sleeves into the tops of 150 x 150 mm (6 x 6 in.) cylinders. It is possible that the confining effects of the cylinder mold produced more reproducible conditions at the base of the cores.

The results of Naik, Salameh, and Hassaballah (1990) suggest that the variability of break-off tests on drilled cores is comparable with that obtained on cores formed by inserting sleeves into fresh concrete; however, cores were drilled into concrete having a compressive strength greater than approximately 20 MPa (3000 psi). Thus, additional data are needed to determine the lowest concrete strength for which core drilling does not affect the integrity of the concrete at the base of the core.

In summary, the results summarized in Table 3.3 support Johansen's (1979) findings that the break-off test has a within-test coefficient of variation of about 9%. The variability is expected to be slightly higher for concrete made with nominal maximum aggregate size greater than 19 mm (3/4 in.).

Table 3.4—Within-test coefficient of variation of pulse-velocity tests

Reference	Coefficient of variation, %	
	Range	Average
Keiller (1982)	0.5 to 1.5	1.1
Carette and Malhotra (1984)	0.1 to 0.8	0.4
Bocca (1984)	0.4 to 1.2	0.7
Yun et al. (1988)	0.4 to 1.1	0.6
Leshchinsky, Yu, and Goncharova (1990)	0.2 to 4.0	1.9
Phoon, Lee, and Loi (1999)	1.1 to 1.2	1.2

3.2.5 *Pulse velocity*—In contrast to the previous test techniques that examine a relatively thin layer of the concrete in a structure, the pulse-velocity method (using through transmission) examines the entire thickness of concrete between the transducers. Localized differences in the composition of the concrete because of inherent variability are expected to have a negligible effect on the measured travel times of the ultrasonic pulses. Thus, the repeatability of this method is expected to be much better than the previous techniques.

Table 3.4 reports the within-test variability of pulse-velocity measurements obtained by different investigators. ASTM C 597 states that the repeatability of test results is within 2%, for path lengths from 0.3 to 6 m (1 to 20 ft) through sound concrete and for different operators using the same instrument or one operator using different instruments.

3.2.6 *Maturity method*—In the maturity method, the temperature history of the concrete is recorded and used to compute a maturity index. Therefore, the repeatability of the maturity indexes depends on the instrumentation used. One would expect the repeatability to be better when using an electronic maturity meter than when the maturity index is computed from temperature readings on a strip-chart recorder. There are, however, no published data on repeatability of maturity measurements using different instrumentation. The precision of temperature measurement by the instrument is not an important issue, provided that steps are taken to ensure that the instrument is operating properly before it is used. Temperature probes can be embedded in temperature-controlled water baths to verify that they are operating properly. The maturity index, after a given time in the bath, can be calculated readily and compared with the instrument reading. Of greater importance than accurate temperature measurement is using the datum temperate or Q-value that represents the temperature sensitivity of the particular concrete.

3.2.7 *Cast-in-place cylinder*—This test method involves the determination of the compressive strength of cylindrical specimens cured in the special molds located in the structure. The repeatability would be expected to be similar to other compression tests on cylinders. Little data have been published. Bloem (1968) reported a within-test coefficient of variation ranging from 2.7 to 5.2% with an average of 3.8% for three replicate tests at ages from 1 to 91 days. Richards[*] reported values from 1.2 to 5.8% with an average of 2.8% for two replicate tests at ages of from 7 to 64 days. Data from Carino, Lew, and Volz (1983), in which three replicate cylinders were tested at ages ranging from 1 to 32 days, show an average coefficient of variation of 3.8%.

ASTM C 873 states that the single-operator coefficient of variation is 3.5% for a range of compressive strength between 10 and 40 MPa (1500 and 6000 psi).

CHAPTER 4—DEVELOPMENT OF STRENGTH RELATIONSHIP

4.1—General

Manufacturers of in-place testing equipment typically provide generalized relationships in the form of graphs or equations that relate the property measured by the particular test device to the compressive strength of standard concrete specimens. These relationships, however, often do not accurately represent the specific concrete being tested. These relationships should not be used unless their validity has been established through correlation testing on concrete similar to that being investigated and with the specific test instrument that will be used in the investigation. The general approach in correlation testing is to perform replicate in-place tests and standard strength tests at various strength levels and then to use statistical procedures to establish the strength relationship. The details, however, will depend on whether the in-place tests are to be used in new construction or in existing structures.

The standard specimen may be the standard cylinder, standard cube, or beam. The in-place tests are often correlated with the compressive strength of cores because core strength is the most established and accepted measure of in-place strength. Cast-in-place cylinders are also useful in determining the in-place strength of new concrete, and their use does not require a pre-established correlation. The statistical techniques for establishing the strength relationship are independent of the type of standard specimen. The specimen type, however, is important when interpreting the results of in-place tests.

[*]Personal communication from former committee member Owen Richards.

4.2—New construction

4.2.1 *General*—For new construction, the preferred approach is to establish the strength relationship by a laboratory-testing program that is performed before using the in-place test method in the field. The testing program typically involves preparing test specimens using the same concrete mixture proportions and materials to be used in construction. At regular intervals, measurements are made using the in-place test technique, and the compressive strengths of standard specimens are also measured. The paired data are subjected to regression analysis to determine the best-fit estimate of the strength relationship.

For some techniques it may be possible to perform the in-place test on standard specimens without damaging them, and the specimens can be subsequently tested for compressive strength. Usually, in-place tests are carried out on separate specimens, and it is extremely important that the in-place tests and standard tests are performed on specimens having similar consolidation and at the same maturity. This may be achieved by using curing conditions that ensure similar internal temperature histories. Alternatively, internal temperatures can be recorded and test ages can be adjusted so that the in-place and standard tests are performed at the same maturity index.

In developing the test plan to obtain a reliable strength relationship, the user should consider the following questions:

- How many strength levels (test points) are needed?
- How many replicate tests should be performed at each strength level?
- How should the data be analyzed?

4.2.2 *Number of strength levels*—The number of strength levels required to develop the strength relationship depends on the desired level of precision and the cost of additional tests. Section A.1 in the Appendix discusses how the number of test points used to develop the strength relationship affects the uncertainty of the estimated strength. From that discussion in Section A.1, it was concluded that in planning the correlation testing program, six to nine strength levels should be considered. Using fewer than six strength levels may result in high uncertainties in the estimated strength and using more than nine levels may not be justifiable economically.

The range of strengths used to establish the correlation should cover the range of strengths that are to be estimated in the structure. This will ensure that the strength relationship will not be used for extrapolation beyond the range of the correlation data. Therefore, if low in-place strengths are to be estimated, such as during slipforming, the testing program must include these low strength levels. The chosen strength levels should be evenly distributed within the strength range.

4.2.3 *Number of replications*—The number of replicate tests at each strength level affects the uncertainty of the average values. The standard deviation of the computed average varies with the inverse of the square root of the number of replicate tests used to obtain the average. The effect of the number of tests on the precision of the average is similar to that shown in Fig. A.1 (Appendix).

Statistics show (ASTM E 122) that the required number of replicate tests depends on: 1) the within-test variability of the method; 2) the allowable error between the sample average and the true average; and 3) the confidence level that the allowable error is not exceeded. The number of replicate tests is, however, often based upon customary practice. For example, in acceptance testing, ACI 318 considers a test result as the average compressive strength of two molded cylinders. Therefore, in correlation testing, two replicate standard compression tests can be assumed to be adequate for measuring the average compressive strength at each level.

The number of companion in-place tests at each strength level should be chosen so that the averages of the in-place tests and compressive strengths have similar uncertainty. To achieve this condition, the ratio of the number of tests should equal the square of the ratio of the corresponding within-test coefficients of variation. If the number of replicate compression tests at each strength level is two, the required number of replicate in-place tests is

$$n_i = 2\left(\frac{V_i}{V_s}\right)^2 \qquad (4\text{-}1)$$

where
n_i = number of replicate in-place tests;
V_i = coefficient of variation of in-place test; and
V_s = coefficient of variation of standard test.

For planning purposes, the coefficients of variation given in Chapter 3 may be used for the in-place tests. For molded cylinders prepared, cured, and tested according to ASTM standards, the within-test coefficient of variation can be assumed to be 3% (ASTM C 39/C 39M). For cores a value of 5% may be assumed (ASTM C 42/C 42M).

4.2.4 *Regression analysis*—After the data are obtained, the strength relationship should be determined. The usual practice is to treat the average values of the replicate compressive strength and in-place test results at each strength level as one data pair. The data pairs are plotted using the in-place test value as the independent value (or X variable) and the compressive strength as the dependent value (or Y variable). Regression analysis is performed on the data pairs to obtain the best-fit estimate of the strength relationship.

Historically, most strength relationships have been assumed to be straight lines, and ordinary least-squares (OLS) analysis has been used to estimate the corresponding slopes and intercepts. The use of OLS is acceptable if an estimate of the uncertainty of the strength relationship is not required to analyze in-place test results, such as if the procedures in Sections 6.2.1 and 6.2.2 are used. If more rigorous methods, such as those in Sections 6.2.3 and 6.2.4, are used to analyze in-place test results, a procedure that is more rigorous than OLS should be used to establish the strength relationship and its associated uncertainty.

The limitations of OLS analysis arise from two of its underlying assumptions:

- There is no error in the X value; and
- The error (standard deviation) in the Y value is constant.

Except for measured maturity indexes, the first of these assumptions is violated because in-place tests (X value) generally have greater within-test variability than compression tests (Y value). In addition, it is generally accepted that the within-test variability of standard cylinder compression tests is described by a constant coefficient of variation (ACI 214R). Therefore, the standard deviation increases with increasing compressive strength, and the second of the aforementioned assumptions is also violated. As a result, OLS analysis will underestimate the uncertainty of the strength relationship (Carino 1993). There are, however, approaches for dealing with these problems.

First, the problem of increasing standard deviation with increasing average strength is discussed. If test results from groups that have the same coefficient of variation are transformed by taking their natural logarithms, the standard deviations of the logarithm values in each group will have the same value* (Ku 1969). Thus, the second assumption of OLS can be satisfied by performing regression analysis using the average of the natural logarithms of the test results at each strength level. If a linear relationship is used, its form is as follows

$$\ln C = a + B \ln I \qquad (4\text{-}2)$$

where
$\ln C$ = average of natural logarithms of compressive strengths;
a = intercept of line;
B = slope of line; and
$\ln I$ = average of natural logarithms of in-place test results.

By obtaining the antilogarithm of $\ln C$, one can transform Eq. (4-2) into a power function

$$C = e^a I^B = AI^B \qquad (4\text{-}3)$$

The exponent B determines the degree of nonlinearity of the power function. If $B = 1$, the strength relationship is a straight line passing through the origin with a slope = A. If $B \neq 1$, the relationship has positive or negative curvature, depending on whether B is greater than or less than one. Regression analysis using the natural logarithms of the test results provides two benefits:

1. It satisfies an underlying assumption of OLS analysis (constant error in Y value); and
2. It allows for a nonlinear strength relationship, if such a relationship is needed.

Use of the transformed data implies that concrete strength is distributed as a lognormal rather than a normal distribution. It has been argued that, for the usual variability of concrete strength, the possible errors from this assumption are not significant (Stone and Reeve 1986).

Next, a method for dealing with the problem of error in the X values is discussed. Fortunately, regression analysis that accounts for X error can be performed with little additional computational effort compared with OLS analysis. One such procedure was proposed by Mandel (1984) and was used by

*In fact, the standard deviation of the transformed values will be approximately the same as the coefficient of variation of the original values, when the coefficient of variation is expressed as a decimal fraction. For example, if the coefficient of variation of a group of numbers equals 0.05, the standard deviation of the transformed values will be approximately 0.05.

Stone and Reeve (1986) to develop a rigorous procedure to analyze in-place test results (discussed in Section 6.2.3). Mandel's approach involves the use of a parameter λ defined as the variance (square of the standard deviation) of the *Y* variable divided by the variance of the *X* variable. For the correlation-testing program, the value of λ is obtained from the standard deviations of the average compressive strength and in-place test results. If the numbers of replicates for compressive tests and in-place tests are chosen so that average values are measured with comparable precision, the value of λ should be close to one.

The parameter λ and the correlation testing results, that is, the averages of the logarithms of the in-place results (*X* values) and the averages of the logarithms of compressive strengths (*Y* values), are used to determine the strength relationship using the calculations outlined in Section A.2 (Appendix). The calculations involve the usual sums of squares and cross-products used in OLS analysis (Mandel 1984). The procedure is well suited for application on a personal computer with a spreadsheet program.

Figure 4.1 is a graphical representation of the difference between OLS analysis and Mandel's procedure. In OLS analysis, the best-fit straight line is the one that minimizes the sum of squares of the vertical deviations of the data points from the line, as shown in Fig. 4.1(a). Mandel's analysis minimizes the sum of squares of the deviations along a direction inclined to the straight line, as shown in Fig. 4.1(b). The direction of minimization depends on the value of λ, which in turn depends on the ratio of the errors in the *Y* and *X* values. As the error in the *X* value increases, the value of λ decreases and the angle θ in Fig. 4.1(b) increases. An important feature of Mandel's analysis is that the estimated standard deviation of the predicted value of *Y* for a new value of *X* accounts for error in the new *X* value and the error in the strength relationship (refer to Section A.3 in the Appendix).

In summary, regression analysis should be performed using the natural logarithms of the test results to establish the strength relationship. This approach will accommodate the increase in within-test variability with increasing strength. Using a straight line to represent the relationship between logarithm values is equivalent to assuming a power function strength relationship. The power function can accommodate a nonlinear relationship, if necessary. To be rigorous, the regression analysis procedure should account for the uncertainty in the in-place test results (*X* error). Failure to account for the *X* error will underestimate the uncertainty of future estimates of in-place compressive strength. This rigorous procedure, however, is justified only when an equally rigorous method will be used to interpret in-place test results (see Chapter 6); otherwise, OLS analysis is acceptable.

4.2.5 *Procedures for correlation testing*—Ideally, it is desirable to determine the compressive strength and the in-place test result on the same specimen so that companion test results are obtained at the same maturity. Unfortunately, this is only possible with those methods that are truly nondestructive, such as pulse velocity and rebound number. For methods that cause local damage to the concrete, separate specimens are needed for obtaining compressive strength and the in-place test result. In such cases, it is important that companion specimens are tested at the same maturity. This is especially critical for early-age tests when strength at a given age depends highly on the thermal history. The problem arises because of differences in early-age temperatures in specimens of different geometries. An approach for moderating temperature differences is to cure all specimens under laboratory conditions in the same water bath.

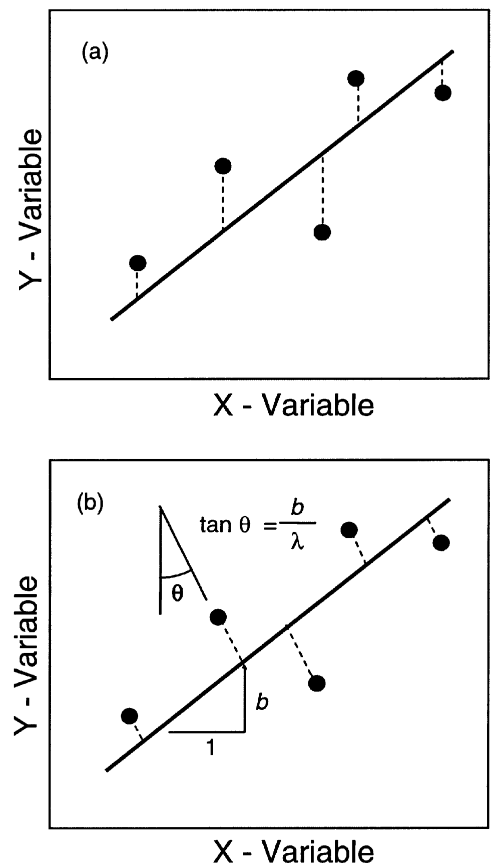

Fig. 4.1—Direction of error minimization in: (a) ordinary least-squares analysis; and (b) Mandel's procedure (Carino 1992).

Alternatively, internal temperatures can be monitored and test ages adjusted so that compression tests and in-place tests are performed at equal values of the maturity index. Failure to perform companion tests on specimens that are at equal maturity will result in an inaccurate strength relationship that will cause systematic errors (or bias) when it is used to estimate the in-place strength in a structure. The following recommendations should be used in correlation testing programs.

4.2.5.1 *Rebound number* —At least 12 standard cylinders should be cast. At each test age, a set of 10 rebound numbers (ASTM C 805) should be obtained from each pair of cylinders held firmly in a compression testing machine or other suitable device at a pressure of about 3 MPa (500 psi). The rebound tests should be made in the same direction relative to gravity as they will be made on the structure. The cylinders should then be tested in compression. If it is not feasible to test the cylinders with the hammer in the same orientation that will be used to test the structure, the correction factors supplied by the equipment manufacturer can be used to account for differences in orientation. As mentioned in Section 2.2, the surface produced by the material of the cylinder molds can differ from the surface produced by the form material for the structure. This factor should also be considered in the correlation testing. If considerable difference is expected between the surfaces of the structure and the cylinders, additional prismatic specimens should be prepared for rebound tests. These specimens should be formed with the same type of forming materials that will be used in construction, and they should be similar in size to the cylinders so that they will experience similar thermal histories. When the rebound number is determined on these specimens, ensure that the specimens are rigidly supported so that they do not move during testing. If the specimens move, lower rebound numbers will be recorded, and the strength relationship will be biased.

For accurate estimates of in-place strength, the moisture content and texture of the surfaces of the cylinders at the time of the correlation tests should be similar to those anticipated for the concrete in the structure at the time of in-place testing. Practically, the only easily reproducible moisture condition for concrete surfaces is the saturated condition.

4.2.5.2 *Penetration resistance*—For the probe penetration test, at least 12 standard cylinders and a test slab large enough for at least 18 probe penetration tests should be cast. For in-place testing of vertical elements, the recommended procedure is to cast a wall specimen and take cores next to the probe tests. All test specimens should be cured under identical conditions of moisture and temperature. At each test age, two compression tests and three probe penetration tests should be made. The recommended minimum thickness for the test slab is 150 mm (6 in.). The minimum spacing between probe penetrations is 175 mm (7 in.), and the minimum distance from a probe to a slab edge is 100 mm (4 in.).

For the pin penetration test, it may be possible to perform penetration tests on the sides of cylinders and subsequently test the cylinders for compressive strength. Carino and Tank (1989) showed that the surface damage produced by pin penetrations into 100 x 200 mm (4 x 8 in.) cylinders did not result in strength reductions. Comparative tests, however, were not performed on specimens with concrete strength less than 25.5 MPa (3700 psi). Until further studies are conducted to confirm that pin penetrations do not affect the compressive strength of cylinders for a wide range of concrete strength, it is recommended that slab specimens be used for pin penetration tests. A minimum of six penetration readings should be performed at each test age. Discard a result when it is obvious that an aggregate particle or a large air void was penetrated. In addition, according to ASTM C 803/C 803M, if the range of penetration values exceeds 1.6 mm (0.064 in.), the result with the maximum deviation from the average should be discarded and a new test performed. Individual penetrations should be spaced between 50 and 150 mm (2 and 6 in.), and the minimum distance from an edge should be 50 mm (2 in.).

4.2.5.3 *Pullout test*—Several techniques have been used. Pullout inserts have been cast in the bottom of standard cylinders, and a pullout test was performed before testing the standard cylinder in compression (Bickley 1982b). In this case, the pullout test is stopped when the maximum load (indicated by a drop in the load with further displacement) has been attained. The insert is not extracted and the cylinder can be capped and tested in compression. Alternatively, companion cylinders have been cast with and without inserts, and the pullout test has been performed on one standard cylinder and the other cylinder tested in compression. Investigators have had problems with both procedures, particularly at high strengths, because radial cracking occurs at the end of the cylinder containing the pullout insert. This cracking is believed to result in lower ultimate pullout loads.

A third alternative has been to cast standard cylinders for compression testing and to place pullout inserts in cubes (or slabs or beams) so that the pullout tests can be made in the companion specimen when the standard cylinders are tested. The latter approach is the preferred method, providing consolidation is consistent between the standard cylinders and the cubes or other specimens containing the pullout inserts, and the maturity of all specimens at the time of testing is the same. The recommended minimum size for cubes is 200 mm (8 in.) when 25 mm (1 in.) diameter inserts are used. Four inserts can be placed in each cube, one in the middle of each vertical side. For each test age, two standard cylinders should be tested and eight pullout tests performed. The same procedure applies to post-installed pullout tests. Install the inserts on the same day that pullout tests will be done.

4.2.5.4 *Break-off test*—The procedure for correlation testing depends on how the system will be used in practice. If the break-off specimens will be formed by inserting sleeves, the correlation testing should involve the fabrication of a slab specimen (or specimens) and companion cylinders. The slabs should have a minimum thickness of 150 mm (6 in.). The sleeves should be inserted into the top surface of the slab after the concrete has been consolidated and screeded. The slabs and cylinders should be subjected to identical curing conditions. When tests are performed, the break-off test locations should be chosen randomly from the available locations. For applications in which the sleeves are to be attached to the sides of formwork, the laboratory specimens should simulate the conditions that will be encountered in the field. For example, if the sleeves will be used on vertical faces of the formwork, the laboratory specimens should be made with the sleeves on the vertical faces of the forms.

When the in-place break-off test specimens will be prepared by core drilling, the correlation testing should involve core drilling into a slab or wall specimen. At each test age, the location of the drilled cores for break-off specimens should be selected randomly. The recommended minimum thickness of the slab or wall is 150 mm (6 in.).

For either specimen preparation method, at least eight break-off specimens and two cylinders should be tested at each test age. The center-to-center spacing of the break-off specimens should be at least 150 mm (6 in.), and the distance from the edge of the slab or wall and the counter bore should be at least 4 in. (100 mm).

4.2.5.5 *Ultrasonic pulse velocity*—It is preferable to develop the strength relationship from concrete in the structure.

Tests should be on cores obtained from the concrete being evaluated. Tests with standard cylinders can lead to unreliable correlations because of different moisture conditions between the cylinders and the in-place concrete.

The correlation data should be obtained from a testing configuration that is similar to the one used in the field because the geometry of the test specimen may affect the determination of the pulse velocity. The recommended procedure is to select certain areas in the structure that represent different levels of pulse velocity. At these locations, it is recommended that five velocity determinations be made to ensure a representative average value of the pulse velocity. For each measurement, the transducers should be uncoupled from the surface and then recoupled to avoid systematic errors due to poor coupling (ASTM C 597). Then obtain at least two cores from each of the same locations for compressive strength testing. Pulse velocity measurements on these cores, once they have been removed from the structure, will usually not be the same as the velocities measured in the structure and are not representative of the pulse velocity of the structure.

4.2.5.6 *Maturity method*—The following procedure is given in ASTM C 1074.

Prepare cylindrical concrete specimens according to ASTM C 192/C 192 M using the mixture proportions for the concrete intended for the structure. Embed temperature sensors at the centers of at least two specimens. Connect the sensors to maturity instruments or to a suitable temperature recording device(s).

Moist cure the specimens in a water bath or in a moist room meeting the requirements of ASTM C 511. Perform compression tests according to ASTM C 39/C 39M at 1, 3, 7, 14, and 28 days. Test at least two specimens at each age.

At each test age, record the average maturity index for the instrumented specimens. On graph paper, plot the average compressive strength as a function of the average maturity index. Draw a best-fit curve through the data. The resulting curve is the strength-maturity relationship to be used for estimating in-place strength. Alternatively, a suitable empirical equation may be fitted to the data using least-squares curve fitting. (Refer to Malhotra and Carino 1991 for possible equations.)

4.2.5.7 *Cast-in-place cylinder*—If necessary, test results should be corrected for the height-diameter ratio using the values given in ASTM C 42/C 42M. No other correlation is needed because the specimens represent the concrete in the placement and the test is a uniaxial compression test.

4.3—Existing construction

4.3.1 *General*—There is often a need to evaluate the in-place strength of concrete in existing structures. For example, planned renovation or change in the use of a structure may require determination of the concrete strength for an accurate assessment of structural capacity. There also may be a need to evaluate concrete strength after a structural failure, fire damage, or environmental degradation has occurred. Sometimes, errors or unforeseen conditions occur during new construction and an evaluation is needed to resolve questions about concrete strength. These situations are similar because the need to determine the in-place strength of the concrete was not preplanned. In-place testing methods can be helpful in these evaluations.

In-place tests can be used in two ways to evaluate existing construction. First, they can be used qualitatively to locate those portions of the structure where the concrete appears to be different from other portions. In this case, the in-place tests can be used without a strength relationship for the concrete in the structure. The main purpose of the in-place testing is to establish where cores should be taken for strength determinations and other pertinent tests (ACI 437R). The rebound number and the pulse velocity method are widely used for this purpose. Second, in-place methods can be used for a quantitative assessment of the strength. In this case, a strength relationship must be established for the concrete in the structure. The relationship can be developed only by performing in-place tests at selected locations and taking companion cores for strength testing. Thus, the use of in-place testing does not eliminate the need for coring, but it can reduce the amount of coring required to gain an understanding of the variations of strength in a structure, and it can give a higher degree of confidence that the cores taken truly represent the conditions being investigated.

4.3.2 *Developing strength relationship*—Because in-place testing for evaluations of existing construction is not preplanned, the techniques that have traditionally been used are ultrasonic pulse velocity, rebound number, and probe penetration. The break-off test is also applicable, but it has not been widely used in North America. In the United Kingdom, the pull-off test is also used (Long and Murray 1984; Murray and Long 1987). The pull-off test involves gluing a steel disk to the concrete surface and measuring the force required to pull off the disk. In Scandinavia and other parts of Europe, a post-installed pullout test is widely used (Petersen 1984, 1997). This test involves drilling a hole into the concrete and cutting out a cylindrical slot to accommodate an expandable ring that functions as the insert head (Fig. 2.8). In 1999, this type of post-installed pullout test was incorporated into ASTM C 900.

For some test methods, certain factors should be considered when testing existing structures. For example, for surface tests (rebound number, penetration resistance, and pull-off), the user must pay special attention to those factors that may affect the near-surface strength, such as carbonation, moisture content, or surface degradation from chemical or physical processes. Surface grinding may be necessary to expose concrete that represents the concrete within the structure.

To develop the strength relationship, it is generally necessary to correlate the in-place test parameter with the compressive strength of cores obtained from the structure. In selecting the core locations, it is desirable to include the widest range of concrete strengths in the structure that is possible. Often, rebound numbers or pulse velocity values are determined at points spread over a grid pattern established on the area being evaluated. When the data are plotted on a map, contour lines can be sketched in to outline the variations in the concrete quality (Murphy 1984). Based on this initial survey, six to nine different locations should be selected for coring and measurement of the in-place test parameter. At each location, a minimum of two cores should be obtained to establish the in-place compressive strength. The number of replicate in-place tests at each location depends on the test method and economic considerations, as discussed in Chapter 5. Because at least 12 cores are recommended to develop an adequate strength relationship, the use of in-place testing may only be economical if a large volume of concrete is to be evaluated.

Cores should be tested in a moisture condition that is representative of the in-place concrete. The recommended procedure is to wipe off excess drilling water, allow the cores to surface dry, and place the cores in sealed plastic bags. Refer to ASTM C 42/C 42M for additional guidance on the handling and testing of cores.

After the averages and standard deviations of the in-place test parameter and core strength are determined at each test location, the strength relationship is developed using the same approach as for new construction (Section 4.2.4).

In evaluating the average and standard deviation of the replicate in-place results, the recorded values should be checked for outliers (ASTM E 178). In general, test results that are more than two standard deviations from the average should be scrutinized carefully. Outliers may occur due to an improperly performed test or a localized, abnormal condition. If an obvious cause of the outlier is identified, that result should be ignored and the average and standard deviation recalculated.

CHAPTER 5—IMPLEMENTATION OF IN-PLACE TESTING

5.1—New construction

5.1.1 *Preconstruction consensus*—Before starting construction of the components of the structure that are to be tested in-place, a meeting should be held among the parties who are involved. The participants typically include the owner, construction manager, structural engineer, testing company, general contractor, subcontractors (such as formwork contractor or post-tensioning contractor), and concrete supplier. The objective of the preconstruction meeting is to clarify the test procedures to be used, the access requirements, the criteria for interpretation of test data, and the interaction among the parties. A mutual understanding among the involved parties will reduce the potential for dispute during construction.

The meeting should achieve a consensus on the following critical issues:

- Agreement on type of formwork material that will be used because it may affect the correlation testing;
- The test procedure(s) to be used, number and locations of tests, the access requirements for testing, and the assistance to be provided by the contractors in preparing and protecting test locations and testing equipment;
- The criteria for acceptable test results for performing critical operations, such as form removal, post-tensioning, removal of reshores, or termination of accelerated or initial curing;
- Procedures for providing access and any modifications to formwork required to facilitate testing;
- Procedures and responsibilities for placement of testing hardware, where required, and protection of test sites;
- Procedures for the timing and execution of testing;
- Reporting procedures to provide timely information to site personnel;
- Approval procedures to allow construction operations to proceed if adequate strength is shown to have been achieved; and
- Procedures to be followed if adequate strength is not shown to have been achieved.

5.1.2 *Number of test locations*—It is important that the tests provide a reliable measure of the strength of the tested component at the time the tests are made. Therefore, sufficient test locations need to be provided so that there are sufficient test results to adequately characterize the concrete strength within the portion of the structure being evaluated. The term "test location" means a region on the structure where an in-place test procedure is to be executed. At a test location, one or more single or replicate in-place tests may be performed.

The number of test locations should account for the following considerations:

- Because tests will be performed at early ages when strength gain of concrete depends highly on temperature, the initial tests may show that adequate strength has not yet been achieved. It will then be necessary to stop testing after the initial tests have been made and to retest at a later age. Sufficient test locations have to be provided to allow for repeat tests and to satisfy the criterion for number of tests required to allow critical operations to proceed; and
- If tests are made at ages under 12 h after the concrete is cast, it is expected that the in-place strength will have high variability due to variations in temperature at the test locations. In this case, it is advisable to increase the number of provided test locations by 10 to 25%.

Table 5.1 to 5.4 provide recommendations for testing various structural components. For each test method, the tables show:

- The number of test locations or access points that should be provided per stated volume of concrete; and
- The minimum number of test locations that should be available for statistical analysis to determine concrete strength.

The numbers in these tables are based on experience considering the criticality of the structural component and practicality.

5.1.3 *Number of tests per location*—The number of in-place tests to be performed at a test location could, in theory, be determined based on the within-test repeatability of the test method, as discussed in Section 4.2.3. Consideration, however, should be also given to practicality; otherwise, in-place testing programs will be avoided because of the financial burden. Table 5.5 lists the minimum number of individual determinations per test location. A lower number is recommended for those in-place test methods that require installation of hardware compared with those methods that do not.

5.1.4 *Providing access to test locations*—To perform in-place tests during construction, it is necessary to provide access to the hardening concrete. The specific details will

Table 5.1—Recommendations for slabs, shearwalls, and core walls*

Test method	Number of test locations provided		Number of locations to test	
	First 75 m^3 (100 yd^3)	Each additional 15 m^3 (20 yd^3)	First 75 m^3 (100 yd^3)	Each additional 15 m^3 (20 yd^3)
Rebound number	20	2	10	1
Probe penetration	8	1	6	1
Pin penetration	15	2	10	1
Pullout	15	2	10	1
Ultrasonic pulse velocity	15	2	10	1
Break-off	10	2	8	1
Maturity	5	2	5	1
Cast-in-place cylinder†	5	1	5	1

*Core walls that typically surround elevator shafts are usually located at the center of a building and form the structural backbone of the building.
†For slabs only.

Table 5.2—Recommendations for other walls per 150 m^3 (200 yd^3)

Test method	Number of test locations provided		Number of locations to test	
	Walls thinner than 300 mm (1 ft)	Walls 300 mm (1 ft) thick or thicker	Walls thinner than 300 mm (1 ft)	Walls 300 mm (1 ft) thick or thicker
Rebound number	20 to 25	15 to 20	10	8
Probe penetration	8 to 10	6 to 8	8	6
Pin penetration	10 to 15	8 to 12	10	8
Pullout	10 to 15	8 to 12	10	8
Ultrasonic pulse velocity	10 to 15	8 to 12	10	8
Break-off	10 to 12	8 to 12	10	8
Maturity	5	5	5	5

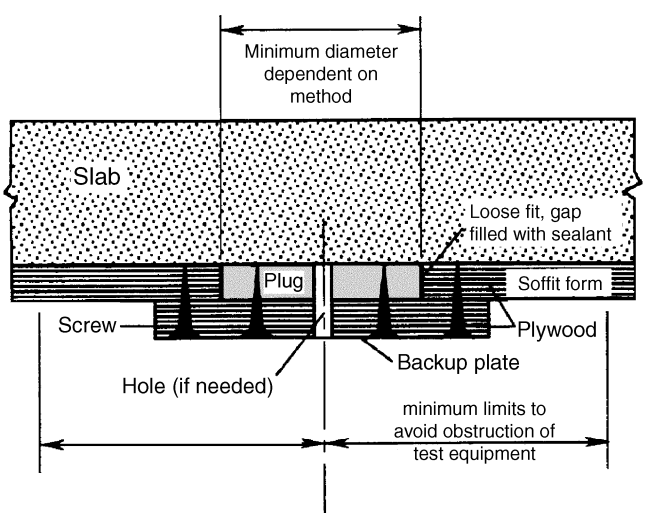

Fig. 5.1—Access for use on vertical surfaces and soffits with wooden forms.

Table 5.3—Recommendations for individual columns*

Test method	Number of test locations provided	Minimum number of locations to test
Rebound number	5 to 8	5
Probe penetration	5 to 8	5
Pin penetration	5 to 8	5
Pullout	5 to 8	6
Ultrasonic pulse velocity	5 to 8	6
Break-off	5 to 8	6
Maturity	5	5

*Recommendations are based on the assumptions that there are six to 10 columns in each test area and that each column contains approximately 1 m^3 (1.5 yd^3) of concrete. Greater numbers of tests should be provided and tested for larger columns or where the test area contains more than 10 columns.

Table 5.4—Recommendations for columns with spandrel beams per for 40 m^3 (50 yd^3)*

Test method	Number of test locations provided	Minimum number of locations to test
Rebound number	6 to 9	5
Probe penetration	6 to 9	5
Pin penetration	6 to 9	5
Pullout	6 to 9	6
Ultrasonic pulse velocity	6 to 9	6
Break-off	6 to 9	6
Maturity	5	5

*Recommendations apply to the number of test locations provided/tested before removal of forms and again before application of construction loading from next level of construction. It is assumed that corbels, if present, are cast integrally with columns or spandrel beams.

Table 5.5—Number of replicate tests at each location

Test method	Minimum number of locations to test
Rebound number	10
Probe penetration	3
Pin penetration	6
Pullout	1
Ultrasonic pulse velocity	2
Break-off	1
Maturity	1
Cast-in-place cylinder	2

depend on the test method, the type of structural component, and the type of formwork. Test locations should be selected to avoid reinforcing steel. Finally, it should be kept in mind that the water absorption characteristics of the form surface at the location of the in-place testing might affect the results of surface tests, such as the rebound number and pin-penetration methods. Form materials for the in-place test specimens in the correlation testing must be similar to those used in construction.

For tests on the soffits of slabs formed with plywood, an access configuration as shown in Fig. 5.1 can be used. A circular hole is cut in the form and the plug that is cut is attached to a backup plate that is temporarily fastened to the formwork with screws. Test hardware, such as a pullout insert, is attached to the removable assembly. When a test is to be performed, test hardware, if it exists, is loosened and the backup plate and plug are removed to expose the test surface. To provide a smooth test surface, a sheet metal plate

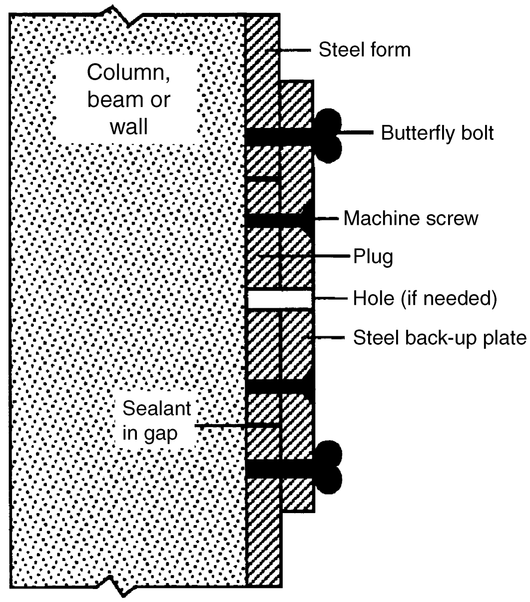

Fig. 5.2—Access for use on vertical surfaces and soffits with steel forms.

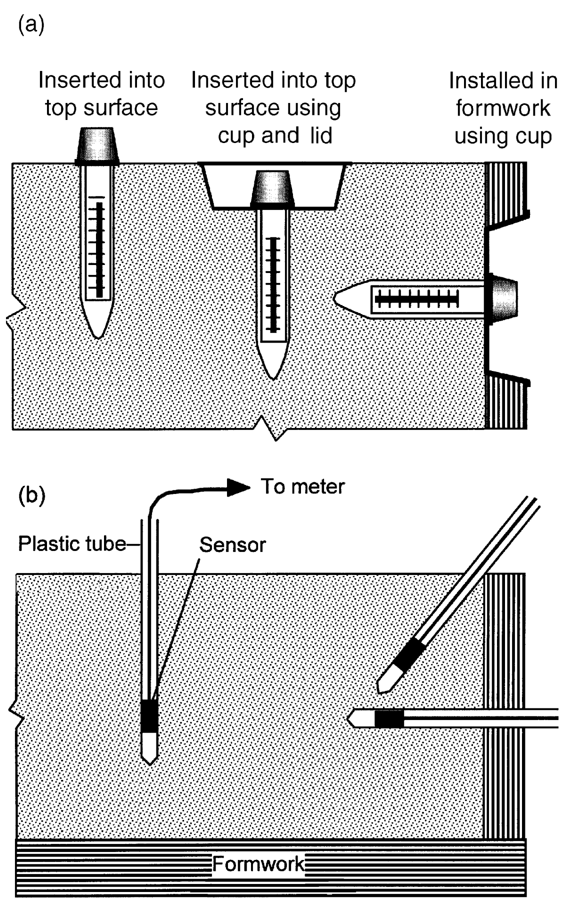

Fig. 5.3—Installation of maturity meters into fresh concrete: (a) disposable mini-meter; and (b) sensor of electronic meter.

can be attached to the plug. A sealant should be used to seal the gap between the plug and backup plate to prevent leakage of fresh cement paste. The diameter of the plug will depend on the specific spacing requirements for the test method, as discussed in Section 5.1.6, and it should provide at least 25 mm (1 in.) of clear space around the perimeter of the plug to avoid testing concrete near the edge of the plug. For access through metal forms, a similar backup plate assembly can be fabricated of metal plate. A typical access configuration for use on the vertical surface of a metal form is shown in Fig. 5.2.

The access types shown in Fig. 5.1 and 5.2 are applicable to all the in-place testing methods except for the maturity method, the break-off test, and cast-in-place cylinders. Figure 5.3 illustrates typical techniques for installing maturity meters. The disposable mini-maturity meters can be inserted directly into the top surfaces of slabs, or they can be embedded deeper into the slab using a cup-lid assembly to avoid interference with finishing operations. The cup may also be placed within openings on the sides of vertical forms. For electronic maturity meters, temperature probes are inserted into the structural elements. For meters with reusable probes, the usual practice is to embed an expendable plastic tube into the fresh concrete and to place the probe within the tube (Fig. 5.3(b)). A thermal couplant (a type of grease) should be applied to the probe before insertion into the tube to ensure accurate measurement of the concrete temperature. For meters that use thermocouple wires as sensors, the wires are fastened to reinforcing bars before concreting. After testing is completed, the thermocouple wires are cut flush with the concrete surface, and the excess wires can be reused.

For break-off tests in the top surfaces of slabs, special access provisions are not necessary. The plastic sleeves are inserted into the fresh concrete after the slab has been screeded. Sleeves can also be attached to the sides of formwork, using the access types shown in Fig. 5.1 and 5.2, and filled during concrete placement. Care is needed to avoid disrupting the sleeves during subsequent finishing operations.

Cast-in-place cylinders do not require special access provisions. The supporting sleeve for the cylinder mold is nailed directly to the formwork. It is only necessary to ensure that the top surface of the specimen will coincide with the top surface of the slab. If the top of the specimen is too low, it will be difficult to locate and extract the cylinder. If the top of the specimen is too high, finishing operations will disrupt the molds.

5.1.5 *Distribution of tests*—Test locations should be distributed throughout the component being tested so that the results provide an accurate indication of the strength distribution within the component. In selecting the testing locations, consideration should be given to the most critical locations in the structure in terms of strength requirements (such as post-tensioning stressing locations) and exposure conditions (such as slab edges), especially during cold weather. When a large number of tests are required for structural components such as slabs, it is advisable to distribute the test locations in a regular pattern. For test methods that require few tests, such as cast-in-place cylinders, it is advisable to choose locations that are critical in each concrete placement.

For tests on vertical members, such as columns, walls, and deep beams, the vertical location within the placement is important. For vertical members, there is a tendency for the concrete strength to be higher at the bottom of the placement than at the top of the placement. The magnitude of this variation is influenced by many factors such as mixture composition, type and degree of consolidation, aggregate shape, and environmental conditions (Murphy 1984; Munday and Dhir 1984; Bartlett and MacGregor 1999). It is not possible to predict accurately the magnitude of strength variation expected in a given component. Also, code writing committees have not addressed these strength variations. As a result, engineering judgment is needed in planning and interpreting the results of in-place tests on vertical members, particularly when testing members with depths greater than 300 mm (12 in.). Similar engineering judgments will also need to be made when testing deep slab sections.

5.1.6 *Critical dimensions*—Tests such as rebound number, penetration resistance, pullout, and break-off produce some surface damage to the concrete, and test results are affected by the conditions within the zone of influence of the particular test. As a result, the ASTM standards prescribe minimum dimensions to assure that test results are not influenced by neighboring tests, specimen boundaries, or reinforcing steel. Test locations should be positioned to conform with the dimensional requirements in Table 5.6.

Table 5.6—Dimensional requirements for in-place tests according to ASTM standards*

Test method	Requirements
Rebound number	*Minimum dimensions* Thickness of member: 100 mm (4 in.) Diameter of test area: 300 mm (12 in.) *Minimum distance* Between test points: 25 mm (1 in.)
Probe penetration	*Minimum distance* Between probes: 175 mm (7 in.) To edge of concrete: 100 mm (4 in.)
Pin penetration	*Minimum distance* Between pins: 50 mm (2 in.) To edge of concrete: 50 mm (2 in.) *Maximum distance* Between pins: 150 mm (6 in.)
Pullout	*Minimum clear spacing* Between inserts: 10 times insert head diameter To edge of member: Four times head diameter From edge of failure surface to reinforcing bar: One insert head diameter or maximum aggregate size, whichever is larger
Break-off	*Minimum clear spacing* Clear spacing between inserts: 100 mm (4 in.)

*The current version of the ASTM test methods should be consulted before planning in-place tests to ensure that proper spacing and clearance requirements are satisfied.

5.2—Existing construction

5.2.1 *Pretesting meeting*—As discussed in Section 4.3, there are many reasons for determining the in-place strength of concrete in existing structures. In-place testing is often one facet of an overall investigation to establish structural adequacy. The guidelines in ACI 437R should be followed to develop the complete plan of the investigation and to identify other aspects of the field study to complement concrete strength determination.

The plan for the in-place testing program will depend on the purpose of the investigation. A pretesting meeting should be held among the members of the team who share a common interest in the test results. At the conclusion of the meeting, there should be a clear understanding of the objective of the investigation; there should be agreement on the responsibilities of the team members in acquiring the test data; and there should be agreement on the procedures for obtaining and analyzing the test results. When access to the concrete for testing is restricted by architectural coverings, detailed plans should be developed to accomplish this access.

5.2.2 *Sampling plan*—In developing the testing program, consideration should be given to the most appropriate sampling plan for the specific situation. ASTM C 823 provides guidelines for developing the sampling plan. Although the standard deals primarily with the drilling of cores or sawn samples, there is a section addressing in-place testing.

In general, two sampling situations may be encountered. In one situation, all of the concrete is believed to be of similar composition and quality. For this case, random sampling should be spread out over the entire structure and the results treated together. ASTM E 105 should be consulted to understand the principles of random sampling. The structure should be partitioned into different regions and a random number table used to determine objectively which areas to test. Objective random sampling is necessary to apply probability theory and make valid inferences about the properties of the population (all of the concrete in the structure) based upon the sample test results.

The second sampling situation arises when available information suggests that the concrete in different sections of the structure may be of different composition or quality, or when the purpose of the investigation is to examine failure or damage in a specific section of a structure. In this case, random sampling should be conducted within each section of the structure where the concrete is suspected of being nominally identical. Test results from different sections of the structure should not be combined unless it is shown that there are no statistically significant differences between the average test results in the different sections.

5.2.3 *Number of tests*—As was discussed in Section 4.3, the in-place testing program for an existing structure involves two phases. First, the strength relationship must be established by testing drilled cores and measuring the corresponding in-place test parameter near the core locations. The locations for correlation testing should be chosen to provide a wide range in concrete strength. As mentioned in Section 4.3.2, a minimum of six to nine test locations should be selected for obtaining the correlation data. In general, cores should be drilled after the in-place tests are performed. At each location, two cores should be drilled, and the following number of replicate in-place tests should be performed to provide the average value of the companion in-place test parameter:

Test method	Replicates at each location
Rebound number	10
Probe or pin penetration	3 to 6
Break-off	5
Ultrasonic pulse velocity	5
Pullout	3

Table 6.1—One-sided tolerance factor for 10% defective level (Natrella 1963)

	Confidence level		
Number of tests n	75%	90%	95%
Column 1	Column 2	Column 3	Column 4
3	2.501	4.258	6.158
4	2.134	3.187	4.163
5	1.961	2.742	3.407
6	1.860	2.494	3.006
7	1.791	2.333	2.755
8	1.740	2.219	2.582
9	1.702	2.133	2.454
10	1.671	2.065	2.355
11	1.646	2.012	2.275
12	1.624	1.966	2.210
13	1.606	1.928	2.155
14	1.591	1.895	2.108
15	1.577	1.866	2.068
20	1.528	1.765	1.926
25	1.496	1.702	1.838
30	1.475	1.657	1.778
35	1.458	1.623	1.732
40	1.445	1.598	1.697
50	1.426	1.560	1.646

The number of replicate in-place tests are based on considerations of the within-test variability of the method and the cost of additional testing. For example, the within-test repeatability of the ultrasonic pulse velocity test is low, and the cost of replicate readings at one location is low. Therefore, five replicate readings are recommended to ensure that a representative value will be obtained because of the variability in the efficiency of the coupling of the transducer to the structure. In making the replicate pulse velocity determinations, the transducers should be moved to nearby locations to evaluate the area where cores will be taken. The dimensional requirements presented in Table 5.6 should be observed for all test methods.

The second phase of the in-place testing program involves performing the in-place tests at other locations and estimating the compressive strength based upon the strength relationship. The number of test locations for this phase will depend on several factors. First, there are the statistical factors. According to the principles set forth in ASTM E 122, the number of tests depends on the variability of the concrete strength, the acceptable error between the true and sample average, and the acceptable risk that the error will be exceeded. Among these factors, the variability of the concrete is a predominant factor in determining the number of required tests. For a given acceptable error and level of risk, the number of tests increases with the square of the variability (ASTM E 122).

Economic considerations also influence the testing plan. For some cases, the cost of an extensive investigation might outweigh the economic benefit. Because the cost of an investigation is related to the amount of testing performed, a high degree of confidence, due to a large sample size, is obtained at a higher cost. The selection of a testing plan involves tradeoffs between economics and degree of confidence.

CHAPTER 6—INTERPRETATION AND REPORTING OF RESULTS

6.1—General

Standard statistical procedures should be used to interpret in-place tests. It is not sufficient to simply average the values of the in-place test results and then compute the equivalent compressive strength by means of the previously established strength relationship. It is necessary to account for the uncertainties that exist. While no procedure has yet been agreed upon for determining the tenth-percentile in-place strength based on the results of in-place tests, proponents of in-place testing have developed and are using statistically based interpretations.

Four statistical methods for evaluating in-place test results are reviewed in the following sections. The first two methods are similar and are based on the idea of statistical tolerance factors. These two methods are simple to use, requiring only tabulated statistical factors and a calculator. Because of their underlying assumptions, however, the statistical rigor of these methods has been questioned. As a result, more rigorous methods have been proposed. The rigorous methods are more complex and require an electronic spreadsheet or computer program for practical implementation.

6.2—Statistical methods

6.2.1 *Danish method (Bickley 1982b)*—This method has been developed for analysis of pullout test results. The pullout strengths obtained from the field tests are converted to equivalent compressive strengths by means of the strength relationship (correlation equation) determined by regression analysis of previously generated data for the particular concrete being used at the construction site. The standard deviation of the converted data is then calculated. The tenth percentile compressive strength of the concrete is obtained by subtracting the product of the standard deviation and a statistical factor K (which varies with the number of tests made and the desired level of confidence) from the mean of the converted data. Although Bickley (1982b) did not state it explicitly, the statistical factor is a one-sided tolerance factor (Natrella 1963), as discussed further in Section 6.2.2. The K factors for different number of tests and a 75% confidence level are given in Column 2 of Table 6.1. The example in Table 6.2 illustrates how the Danish method is applied. The first column shows the equivalent compressive strengths corresponding to the 10 individual pullout test results. The second column shows the values and calculations used to obtain the tenth percentile strength at a 75% confidence level. The example uses 10 test results, but another appropriate number may be used in larger placements.

6.2.2 *General tolerance factor method (Hindo and Bergstrom 1985)*—The acceptance criteria for strength of concrete cylinders in ACI 214 are based on the assumption that the probability of obtaining a test with strength less than f_c' is less than approximately 10%. A suggested method for evaluating in-place tests of concrete at early ages is to determine the lower tenth percentile of strength, with a prescribed confidence level.

It has been established that the variation of cylinder compressive strength can be modeled by the normal or the lognormal distribution function depending upon the degree of quality control. In cases of excellent quality control, the distribution of compressive strength results is better modeled by the normal distribution; in cases of poor control, it is better modeled by a lognormal distribution (Hindo and Bergstrom 1985).

In the tolerance factor method, the lower tenth percentile compressive strength is estimated from in-place test results by considering quality control, number of tests n, and the required confidence level p. Three quality control levels are considered: excellent, average, and poor, with the distribution function of strength assumed as normal, mixed normal-lognormal, and lognormal, respectively. Suggested values of p are 75% for ordinary structures, 90% for very important buildings, and 95% for crucial parts of nuclear power plants (Hindo and Bergstrom 1985). Because safety during construction is the primary concern, it may be adequate to use the same p value for all structures. A value of p equal to 75% is widely used in practice.

The tolerance factor K, the sample average Y, and standard deviation s_Y are used to establish a lower tolerance limit, that is, the lower tenth percentile strength. For a normal distribution function, the estimate of the tenth percentile strength $Y_{0.10}$ can be determined as follows

$$Y_{0.10} = Y - Ks_Y \tag{6-1}$$

where

$Y_{0.10}$ = lower tenth percentile of strength (10% defective);
Y = sample average strength;
K = one-sided tolerance factor (Table 6.1); and
s_Y = sample standard deviation.

The tolerance factor is determined from statistical characteristics of the normal probability distribution and depends on the number of tests n, the confidence level p, and the defect percentage. Values of K are found in reference books such as that by Natrella (1963). Table 6.1 provides one-sided tolerance factors for confidence levels of 75, 90, and 95% and a defect level of 10%.

For the lognormal distribution, the lower tenth percentile of strength can be calculated in the same manner, but using the average and standard deviation of the logarithms of strengths in Eq. (6-1).

By dividing both sides of Eq. (6-1) by the average strength Y, the following is obtained

$$\frac{Y_{0.10}}{Y} = 1 - KV_Y \tag{6-2}$$

where V_Y = coefficient of variation (expressed as a decimal).

In Eq. (6-2), the tenth-percentile strength is expressed as a fraction of the average strength. Figure 6.1 is a plot of Eq. (6-2) for p = 75% and for coefficients of variation of 5, 10, 15, and 20%. This figure shows that as the variability of the test results increases or as fewer tests are performed, the tenth-percentile strength is a smaller fraction of the average strength.

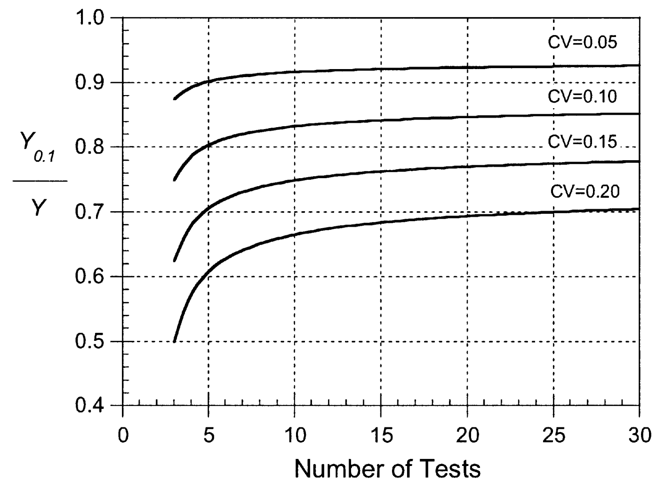

Fig. 6.1—Ratio of tenth-percentile strength to average strength as a function of coefficient of variation and number of tests (normal distribution assumed).

Table 6.2—Example of Danish method

Individual equivalent compressive strength, MPa (psi)*	Calculations
27.5 (3990)	
25.0 (3620)	Mean Y = 25.7 MPa (3730 psi)
24.5 (3550)	
25.0 (3620)	Standard deviation s_Y = 2.3 MPa (330 psi)
22.5 (3260)	
24.0 (3480)	K = 1.671†
25.5 (3700)	
28.5 (4130)	Tenth percentile strength $= Y - Ks_Y$
25.0 (3620)	= 21.9 MPa (3180 psi)
30.0 (4350)	

*Converted from pullout force measurements using strength relationship.
†The values of the constant K for the 75% confidence level are given in Column 2 of Table 6.1.

The tolerance factor method is similar to the Danish method. The results of the in-place tests are converted to equivalent compressive strengths using the strength relationship, and the equivalent compressive strengths are used to compute the sample average and standard deviation.

The example in Table 6.3 illustrates the application of the tolerance factor method for probe-penetration tests. The question in the example is whether the in-place strength of concrete in a slab is sufficient for the application of post-tensioning, if the compressive strength requirement for post-tensioning is 20 MPa (2900 psi). The numbers in the first column are the measured exposed lengths of each of eight probes, and the second column gives the corresponding compressive strengths based on the previously established strength relationship for the concrete being evaluated. For eight tests and a confidence level of 75%, the tolerance factor is 1.74. It is assumed that the normal distribution describes the variation of concrete strength. Thus, by substituting the coefficient of variation and the tolerance factor into Eq. (6-2), the ratio of the tenth-percentile strength to the average strength is 0.838. Therefore, the tenth-percentile in-place strength is 18.6 MPa (2700 psi). Because the tenth percentile

Table 6.3—Example of general tolerance factor method

Strength relationship: Y (MPa) = –1 + 0.69L (mm) Y (psi) = –145 + 2540L (in.)	
Exposed length L, mm (in.)	Compressive strength Y, MPa (psi)
30 (1.18)	19.7 (2850)
35 (1.38)	23.2 (3360)
34 (1.34)	22.5 (3260)
35 (1.38)	23.2 (3360)
38 (1.50)	25.2 (3660)
36 (1.42)	23.9 (3460)
31 (1.22)	20.3 (2950)
30 (1.18)	19.7 (2850)

Mean (Y) = 22.2 MPa (3220 psi).
Standard deviation (s_Y) = 2.1 MPa (300 psi).
Coefficient of variation (V_Y) = 9.3%.
For n = 8 and 75% confidence level: K = 1.74.
$Y_{0.10} = (1 - KV_Y)Y = (1 - 1.74 \times 0.093) \times 22.2$ = 18.6 MPa (2700 psi).

strength is greater than 0.85 × 20 MPa (2900 psi) = 17 MPa (2465 psi), post-tensioning may be applied.*

6.2.3 *Rigorous method (Stone and Reeve 1986)*—The preceding methods convert each in-place test result to an equivalent compressive strength value by means of the strength relationship. The average and standard deviation of the equivalent compressive strength are used to compute the tenth-percentile in-place strength. Two major objections have been raised to these methods (Stone, Carino, and Reeve 1986; Stone and Reeve 1986):

1. The strength relationship is presumed to have no error; and

2. The variability of the compressive strength in the structure is assumed to be equal to the variability of the in-place test results.

The first factor will make the estimates of in-place tenth-percentile strength not conservative, whereas the second factor will make the estimates overly conservative.

Stone and Reeve (1986) developed a comprehensive technique for statistical analysis of in-place test results that attempted to address the perceived deficiencies of the tolerance factor methods. Only a general summary of the method is given herein. This rigorous method encompasses the following procedures:

1. Regression analysis to establish the strength relationship;

2. Estimating the variability of the in-place compressive strength based on the results of the correlation tests and tests on the structure; and

3. Calculating the probability distribution of the estimated in-place, tenth-percentile strength.

For the reasons given in Section 4.2.4, the logarithms of the test results are used in the analysis, and the strength relationship is assumed to be a power function. Regression analysis is performed using Mandel's procedure discussed in Section 4.2.4 and in Appendix A.2. The errors associated with the best-fit strength relationship are used to estimate the in-place, tenth-percentile strength at any desired confidence level.

A novelty of the rigorous method is the approach used to estimate the variability of the in-place compressive strength. In Chapter 3, it was shown that the within-test variability of in-place test results is generally greater than compressive-test results. This is why objections have been raised against assuming that the variability of the in-place compressive strength equals the variability of the in-place test results. In the rigorous method, it is assumed that the variability of compressive strength divided by the variability of the in-place test results is a constant. Thus, the ratio obtained during correlation testing is assumed to be valid for the tests conducted in the field. This provides a means for estimating the variability of the in-place compressive strength based on the results of the in-place tests (see Section 6.2.4).

The in-place tenth-percentile strength computed by the rigorous procedure accounts for the error associated with the strength relationship. The user can determine the tenth-percentile strength at any desired confidence level for a particular group of field test results. In addition, the user can choose the percentile to be a value other than the tenth percentile.

Stone, Carino, and Reeve (1986) computed the tenth-percentile strengths by the rigorous method and compared them with those computed by the Danish and tolerance factor methods. These calculations used simulated in-place test data having different mean values and standard deviations. It was found that, for an assumed confidence level, the strengths estimated by the Danish and tolerance factor methods were lower than the values based on the rigorous method. The differences were as high as 40% when the in-place tests had high variability (coefficient of variation = 20%). Compared with the rigorous method, the Danish and tolerance factor methods give more conservative estimates of in-place compressive strength, but they do not appear to provide a consistent confidence level. One reason for the inconsistency of the tolerance factor method is the assumption that the variability of the in-place compressive strength is the same as the variability of the in-place test results. Experimental field studies are needed to compare the in-place, tenth-percentile strengths estimated by these methods with the values obtained from many core tests. Only then can the reliability of these methods be evaluated.

6.2.4 *Alternative method (Carino 1993)*—The rigorous method developed by Stone and Reeve (1986) has not received widespread acceptance among concrete technologists because of its complexity. Carino (1993) proposed an alternative method that retains the main features of the rigorous method but can be implemented easily with spreadsheet software.

The basic approach of the alternative method is illustrated in Fig. 6.2. Mandel's procedure (as outlined in Appendix A.2) is used to obtain the strength relationship from correlation data. The results of the in-place tests and the strength relationship are used to compute the lower confidence limit of the estimated average in-place strength at a desired confidence level. Finally, the tenth-percentile strength is determined assuming a lognormal distribution for the in-place concrete strength. Calculations are performed using natural-logarithm values.

*Refer to Section 3.1 for discussion of the 0.85 factor.

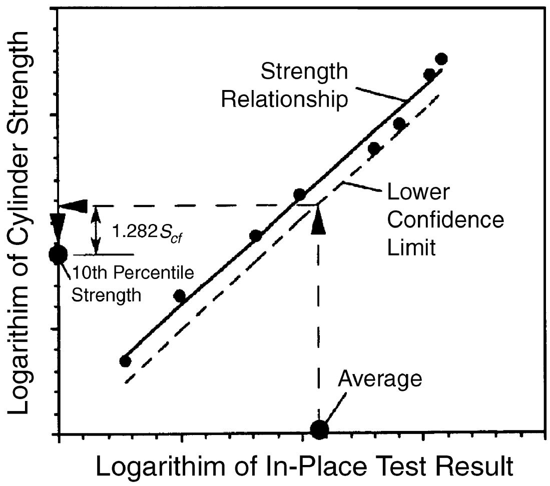

Fig. 6.2—Alternative method to estimate compressive strength based on in-place tests (Carino 1993).

In the following paragraphs, the procedure for estimating the in-place strength is explained further. When the in-place strength is to be estimated, replicate tests are performed on the structure. The average of the logarithms of the in-place tests is used to compute the logarithm of the average in-place compressive strength using the strength relationship

$$Y = a + bX \tag{6-3}$$

where

Y = the logarithm of the estimated average in-place compressive strength;

X = the average of the logarithms of the in-place tests performed on the structure; and

a,b = the intercept and slope of the strength relationship.

Next, the lower confidence limit for the estimated average strength is computed. This lower limit is obtained using Eq. (A-16) in Appendix A.3 for the standard deviation s_Y of an estimated value of Y for a new X. The lower confidence limit for the average concrete strength is as follows

$$Y_{low} = Y - (t_{m\text{-}1,\alpha} s_Y) \tag{6-4}$$

where

Y_{low} = lower confidence limit at confidence level α;

$t_{m\text{-}1,\alpha}$ = Student t-value for m-1 degrees of freedom and confidence level α; and

m = the number of replicate in-place tests.

Table 6.4 lists Student t-values for m-1 degrees of freedom and risk (or confidence) levels of 5 and 10%. The choice of risk level depends on the criticality of in-place concrete strength in the overall assessment. When strength is critical, a lower risk level, such as 5%, should be used.

The distribution of in-place compressive strength is described by a lognormal distribution, and the tenth-percentile strength is computed as follows

$$Y_{0.10} = Y_{low} - 1.282 s_{cf} \tag{6-5}$$

Table 6.4—Student t-values for m-1 degrees of freedom and risk levels of 0.05 and 0.10 (Natrella 1963)

m-1	$t_{0.05}$	$t_{0.10}$
2	2.920	1.886
3	2.353	1.638
4	2.132	1.533
5	2.015	1.476
6	1.943	1.440
7	1.895	1.415
8	1.860	1.397
9	1.833	1.383
10	1.812	1.372
11	1.796	1.363
12	1.782	1.356
13	1.771	1.350
14	1.761	1.345
15	1.753	1.341
16	1.746	1.337
17	1.740	1.333
18	1.734	1.330
19	1.729	1.328

where

$Y_{0.10}$ = logarithm of strength expected to be exceeded by 90% of the population; and

s_{cf} = standard deviation of the logarithms of concrete strength in the structure.

The value of s_{cf} is obtained from the assumption (Stone and Reeve 1986) that the ratio of the standard deviation of compressive strength to the standard deviation of in-place test results has the same value in the field as was obtained during the laboratory correlation testing. Thus the following relationship is assumed

$$s_{cf} = \frac{s_{cl}}{s_{il}} s_X \tag{6-6}$$

where

s_{cf}, s_{cl} = standard deviations of logarithm of compressive strength in the structure and laboratory, respectively; and

s_X, s_{il} = standard deviation of logarithms of the in-place results in the structure and laboratory, respectively.

The final step is to convert the result obtained from Eq. (6-5) into real units by taking the antilogarithm.

A close examination of the alternative procedure shows that the average compressive strength estimated by the strength relationship (Eq. (6-3)) is reduced by two factors. The first factor, which is given by Eq. (6-4), accounts for the uncertainty of the strength relationship and the uncertainty of the average of the in-place test results. The second factor, which is given by Eq. (6-5), accounts for the variability of the in-place compressive strength. Thus, it is felt that the alternative procedure strikes a balance between statistical rigor and practicality of use. As mentioned, the procedure is

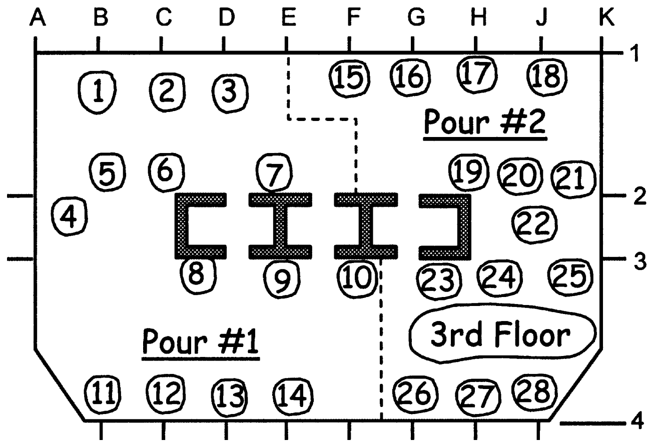

Fig. 6.3—Example of form used to identify locations of in-place tests in a floor slab of multistory building.

TESTING COMPANY

Field Record of In-Place Testing

Test Number	Test result	Estimated compressive strength
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		

Project Number ____________
Project Name ____________
Location in structure ____________

Placement
Date ________
Time ________
Size ________
Mix. No. ________

Curing Conditions

Maturity _____°C-h
Temperature at test:
Ambient ____°C
Within enclosure ____°C

Appearance of top surface:
____

Calculations:

Number of tests: ______

Estimated strength:
Mean:________

Standard deviation:________
K-Value:________
Minimum Strength:
Mean - (K * sd):________

Remarks:
____
____
____
____

Technician____________
Checked by____________
Instrument number________

Fig. 6.4—Sample form for on-site recording of in-place test results.

well suited for implementation using a computerized spreadsheet or a specialized computer program (Chang and Carino 1998). Appendix A.4 gives examples that compare the estimated in-place strength using the tolerance factor and alternative methods.

6.2.5 *Summary*—With the exception of cast-in-place cylinder tests, in-place tests provide indirect measures of concrete strength. To arrive at a reliable estimate of the in-place strength, the uncertainties involved in the estimate must be considered. This section has discussed some techniques

TESTING COMPANY LETTERHEAD

Project No.____________
Client:____________
Address ____________
____
____
Attention: ____________

Report No. ________
(in sequence)

Testing of In-Place Strength
Project Name:____________
Address:____________

Dear Sir:
The following are the results of in-place tests of ______ MPa concrete at the above site.

Location in structure: ____________

	Date	Time
Pour:		
Test:		
Proposed time of form removal		

Test Results Summary

Number of tests made:	
Mean in-place-strength (MPa):	
Standard deviation (MPa):	
Minimum in-place strength (MPa):	

Individual tests results (MPa)

Remarks:
Requirements of ______ MPa mean and ______ MPa minimum strength prior to stripping and reshoring are/are not met by the above results.

Yours very truly,

Signed: ____________
(Testing Co. Engineer)

Copy given to site Superintendent
Date: ____________
Time: ____________
Signed ____________
(for Testing Co.)
____
(for Contractor)

White: Testing Co. Yellow: Contractor Pink: Structural Engineer

Fig. 6.5—Sample form for reporting in-place test results.

developed for this purpose. The tolerance factor methods discussed in Sections 6.2.1 and 6.2.2 have been used successfully in the analysis of pullout test data. Therefore, they may be adequate for test methods that have good correlation with compressive strength, such as the pullout test.

The tolerance factor methods, however, do not account for the main sources of uncertainty in a rational way. This has led to the development of more rigorous procedures as discussed in Sections 6.2.3 and 6.2.4. These new methods are designed to provide reliable estimates of in-place strength for any test procedure. These rigorous methods, however, need to be incorporated into easy-to-use computer programs for practical use.

6.3—Reporting results

Report forms for the different tests and different purposes will vary. A variety of report forms will be appropriate. Usually, relevant ASTM standards describe the information required on a report. Where in-place testing is made at early ages, some particular reporting data are desirable. A set of forms, similar to those developed by an engineer for use in pullout testing, is shown in Fig. 6.3 to 6.5. These may serve as useful models for developing forms to report the results of other in-place tests.

Briefly, the three forms provide for the following:

1. *Record of test locations (Fig. 6.3)*—This form gives a plan view of a typical floor in a specific multistory building. The location of each test is noted. The location of maturity meters, if installed, can also be shown. Location data are

important in case of low or variable results. Where tests are made at very early ages and the time to complete a placement is long, there may be a significant age-strength variation from the start to the finish of the placement.

2. *Record of field-test results (Fig. 6.4)*—This is the form on which test data, the calculated results, and other pertinent data are recorded at the site. The form shown in Fig. 6.4 has been designed for evaluating the data with the Danish or tolerance-factor methods (minimum strength is the tenth-percentile strength). It includes provisions for entering information on maturity data, protection details, and concrete appearance to corroborate the test data during cold weather. Due to the critical nature of formwork removal, a recommended procedure is for the field technician to phone the data to a control office and obtain confirmation of the calculations before giving the results to the contractor.

3. *Report of test results (Fig. 6.5)*—This form is used to report the in-place test results. The example shown in Fig. 6.5 is a multicolor self-carbon form designed to be completed at the site by the technician, with copies given to the contractor's and structural engineer's representatives when the results have been checked. It provides for identification of the placement involved, the individual results, and the calculated mean and minimum strengths. It records the engineer's requirements for form removal and states whether these requirements have been met. It requires the contractor's representative's signature on the testing company's copy.

CHAPTER 7—IN-PLACE TESTS FOR ACCEPTANCE OF CONCRETE

7.1—General

Traditionally, acceptance testing for new construction has been limited to judging the acceptability of the concrete delivered to the project on the basis of slump, air content, and compressive strength. Acceptable concrete that is placed, consolidated, and cured according to standards of good practice will perform according to design assumptions. Exceptions occur when there is clear evidence of inadequate consolidation or distress, such as cold joints and excessive cracking, or when inadequate protection was provided in cold weather.

The durability of exposed structures depends strongly on the curing history of the concrete. Therefore, it is desirable to have assurance that the concrete in the finished structure has the necessary properties to attain the desired level of performance. In-place testing offers the opportunity to obtain this assurance when used as a component in a comprehensive quality assurance program. The Great Belt Link project in Denmark is one of the first large-scale construction projects in which the owners relied on in-place testing (pullout tests) instead of standard laboratory strength tests to assess the acceptability of the concrete layer protecting the reinforcement (Vincentsen and Henriksen 1992). This major construction effort serves as a model for future projects where in-place quality assurance is important.

In North America, there is a reluctance to abandon traditional acceptance procedures that have served their purpose. In-place testing, however, offers the opportunity to lessen the reliance on testing of standard-cured cylinders as the sole method to judge acceptability of concrete delivered to the site. The added benefit of in-place testing is that it provides assurance that the finished construction has the properties specified by the designer. This chapter discusses the potential for in-place testing as an alternative tool for acceptance testing.

7.2—Acceptance criteria

The following reviews the current acceptance criteria in North American practice and proposes how in-place testing may be used as an alternative to testing standard-cured cylinders.

7.2.1 *Molded cylinders*—According to ACI 318M-02 (ACI 318-02), the evaluation and acceptance of concrete are based on tests of cylinders molded at the job site and subjected to standard laboratory curing in accordance with ASTM C 31/C 31M. Section 5.6.3.3 of ACI 318M-02 (ACI 318-02) states as follows:

"Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met:

(a) Every arithmetic average of any three consecutive strength tests equal or exceed f_c'.

(b) No individual strength test (average of two cylinders) falls below f_c' by more than 3.5 MPa (500 psi) when f_c' is 35 MPa (5000 psi) or less; or by $0.10f_c'$ when f_c' is more than 35 MPa (5000 psi)."

In addition, according to 5.6.4.1 of ACI 318M-02 (ACI 318-02), the building official may require testing of field-cured cylinders to check the adequacy of curing and protection of the concrete in the structure. The acceptability of curing, as indicated by the field-cured cylinder strengths, is defined in section 5.6.4.4:

"Procedures for protecting and curing concrete shall be improved when strength of field-cured cylinders at test age designated for determination of f_c' is less than 85 percent of that of companion laboratory cylinders. The 85 percent limitation shall not apply if field-cured strength exceeds f_c' by more than 3.5 MPa (500 psi)."

7.2.2 *Cores*—In the event that a strength test of standard-cured cylinders is more than 3.5 MPa (500 psi) below f_c', ACI 318M-02 (ACI 318-02) requires that steps be taken to ensure adequacy of the structure. Cores may have to be drilled to verify the in-place strength. Three cores are required for each strength test failing to meet the specified criteria. In judging the acceptability of the core strengths, Section 5.6.5.4 of ACI 318M-02 (ACI 318-02) states the following:

"Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of f_c' and if no single core is less than 75 percent of f_c'. Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted."

7.2.3 *In-place tests*—Based on the aforementioned requirements for judging the acceptability of in-place concrete based on core strengths, the following acceptance criteria based on in-place testing are proposed:

The concrete in a structure is acceptable if the estimated average, in-place, compressive strength based on an ASTM

Table 7.1—Results of standard-cured cylinder and in-place tests at 28 days (f_c' = 30 MPa)

	Project 1		Project 2	
	Pullout tests	Standard cylinders	Pullout tests	Standard cylinders
No. of results*	84	84	15	15
Mean strength, MPa (psi)	34.4† (4990)	38.8 (5630)	35.9† (5210)	38.2 (5540)
Standard deviation s, MPa (psi)	2.7†(390)	3.9 (570)	2.7† (390)	3.5 (510)
Range, MPa (psi)	30.5 to 44.5 (4420 to 6450)	29.9 to 40.5 (4340 to 6920)	32.5 to 40.5† (4710 to 5870)	30.9 to 43.5 (4480 to 6310)
Mean strength $-f_c'$	1.63s	2.23s	2.18s	2.34s
Expected percentage of results below f_c'	4.9	1.2	1.4	1
Actual percentage of results below f_c'	None	1.2	None	None

*A result is the average of two cylinder tests or the average of two or more pullout tests.
†Mean and standard deviation of estimated compressive strength based on strength relationship.

standard in-place test procedure equals at least 85% of f_c' and no test result estimates the compressive strength to be less than 75% of f_c'.

Before these criteria can be put into effect, however, a standard practice for statistical analysis of in-place test data needs to be adopted.

7.3—Early-age testing

The primary reason for using in-place tests in new construction is to determine whether it is safe to perform critical operations, such as form removal or post-tensioning. The in-place tests provide estimates of compressive strength at ages that are usually much earlier than the age for attaining the specified strength. The criterion frequently used to judge the acceptability of early-age strengths to permit critical construction operations is that the estimated in-place compressive strength should be at least 75% of f_c'. In this case, the estimated strength should be an estimate of the tenth-percentile strength. When such a requirement is specified, early-age testing may facilitate final acceptance of concrete.

In high-rise construction, economic factors result in accelerated schedules in which critical operations may be planned as early as 1 to 3 days after concrete placement. To meet the early-age strength requirements, the contractor may choose to use a concrete mixture that will exceed the specified design strength. Experience has shown that requiring a minimum strength of 75% of f_c' at early ages (1 to 3 days) will usually ensure that the in-place strength will be at least f_c' at 28 days, if proper curing is used and the specifications do not allow mixtures that achieve all their strength gain at the time of form removal.

For example, for a specified design strength of 28 MPa (4000 psi), the in-place strength to permit form removal may have to be at least 21 MPa (3000 psi). Allowing for the inherent variation of concrete strength, the average in-place strength may have to be 25.5 MPa (3700 psi) to ensure that the early-age strength criterion is satisfied. In this example, the average early-age, concrete strength has to equal 93% of the specified strength. Therefore, it is reasonable to assume that if the early-age (1 to 3 days) strength requirement is satisfied, then at 28 days the specified design strength will undoubtedly be achieved. For additional assurance, in-place tests can be made on the structure at 28 days.

Bickley (1984) reported on two demonstration projects where in-place testing was used not only for early-age strength determination of horizontal elements but also for confirmation of the 28-day design strength. Permission to waive standard cylinder testing was obtained from the building official. Innovative project specifications defined the frequency of in-place tests and the procedures to follow in doing the tests and reporting the results. Acceptance of the concrete was based on the results of pullout tests performed on the structure at 28 days. For comparison, standard-cured cylinders were also tested at 28 days, but these strengths were not reported. Table 7.1 summarizes the results. The specified design strength for both projects was 30 MPa (4350 psi). Individual pullout test results were converted to compressive strengths based on the strength relationships, and these estimated strengths were used to compute the statistics shown in the second and fourth columns of the table. Based on the standard deviations, the expected percentages of strength below f_c' were computed. In all cases, these percentages were less than 10%, which is the approximate value implied in ACI 318. For both projects, the in-place test results clearly showed that the concrete had acceptable strength.

In conclusion, current legal contracts for the sale and purchase of ready-mixed concrete are usually based on the 28-day strength of standard-cured cylinders. For the time being, therefore, these cylinders have to be cast. When in-place tests are made at an early age, however, the acceptability of the concrete can be assessed at that time. If the concrete is satisfactory, there is no need to test the standard cylinders. If the early in-place tests indicate a problem with concrete in a particular placement, the related standard cylinders are available for testing.

CHAPTER 8—REFERENCES

8.1—Referenced standards and reports

The standards and reports listed as follows 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 Concrete Institute

214 Evaluation of Strength Test Results of Concrete
228.2R Nondestructive Test Methods for Evaluation of Concrete in Structures
301 Standard Specifications for Structural Concrete
306R Cold Weather Concreting
308R Guide to Curing Concrete
308.1 Standard Specification for Curing Concrete
318/ Building Code Requirements for Structural
318M Concrete and Commentary
437R Strength Evaluation of Existing Concrete Buildings

ASTM International

C 31/C 31M	Practice for Making and Curing Concrete Test Specimens in the Field
C 39/C 39M	Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 42/C 42M	Test Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 192/C 192M	Practice for Making and Curing Concrete Test Specimens in the Laboratory
C 511	Specification for Moist Cabinets, Moist Rooms, and Water Storage Tanks Used in the Testing of Hydraulic Cements and Concretes
C 597	Test Method for Pulse Velocity Through Concrete
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 873	Test Method for Compressive Strength of Concrete Cylinders Cast in Place in Cylindrical Molds
C 900	Test Method for Pullout Strength of Hardened Concrete
C 1074	Practice for Estimating Concrete Strength by the Maturity Method
C 1150	Test Method for the Break-Off Number of Concrete
E 105	Recommended Practice for Probability Sampling of Materials
E 122	Recommended Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process
E 178	Practice for Dealing with Outlying Observations

British Standards Institution

BS 1881-Part 207	Recommendations for the Assessment of Concrete Strength by Near-to-Surface Tests

These publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA19428

British Standards Institution
389 Chiswick High Road
London W4 4AL
United Kingdom

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APPENDIX

A.1—Minimum number of strength levels

The minimum number of strength levels needed to develop the strength relationship depends on statistical considerations and cost. To gain some insight, it is useful to examine how the confidence interval for an estimate obtained from a strength relationship is affected by the number of points used to establish that relationship (Carino 1993). Because the strength relationship is used to estimate compressive strength from in-place test results, compressive strength is treated as the dependent variable (Y value) and the in-place result as the independent variable (X value).

The residual standard deviation (also called standard error of estimate) is the basic parameter used to quantify the uncertainty of a best-fit strength relationship for a given set of data. For a linear relationship, an estimate of the residual standard deviation is as follows

$$S_e = \sqrt{\frac{\Sigma(d_{yx})^2}{N-2}} \tag{A-1}$$

where

S_e = estimated residual standard deviation;

d_{yx} = deviation of each test point from the best-fit line; and

N = number of test points used to establish the strength relationship.

When the strength relationship is used to estimate the mean value of Y at a new value of X, the width of the confidence interval for the mean is related to the residual standard deviation by the following expression* (Natrella 1963; Snedecor and Cochran 1967)

$$W = 2\, t_{N\text{-}2,\alpha/2}\, S_e \sqrt{\frac{1}{N} + \frac{(X-\bar{X})^2}{S_{xx}}} \tag{A-2}$$

where

W = width of the 100(1-α)% confidence interval for the estimated mean value of Y for the value X;

$t_{N\text{-}2,\alpha/2}$ = student t-value for N-2 degrees of freedom and significance level α;

$\bar{X}$ = average of X values used to develop strength relationship; and

S_{xx} = sum of squares of deviations about $\bar{X}$ of the X values used to develop the strength relationship, $S_{xx} = \Sigma(X-\bar{X})^2$.

The second term under the square root sign in Eq. (A-2) shows that the width of the confidence interval increases as the distance between X and $\bar{X}$ increases. This means that the uncertainty of the estimated strength is greater at the extreme limits of the strength relationship than at its center.

To examine how the width of the confidence interval is affected by the number of test points, consider the case where $X = \bar{X}$, so that the second term under the square root sign in Eq. (A-2) equals zero. The width of the confidence interval relative to the residual standard deviation is as follows

$$\frac{W(\bar{X})}{S_e} = 2\, t_{N-2,\,\alpha/2}\sqrt{\frac{1}{N}} \tag{A-3}$$

Equation (A-3) is plotted in Fig. A.1 to show how the width of the 95% confidence interval (relative to S_e) is affected by the number of test points used to establish the strength relationship. It is seen that, for few test points (say, less than 5), by including an additional test point there is a significant reduction in the relative width of the confidence interval. For many points, however, the reduction obtained by using an additional test point is small. Therefore, the appropriate number of strength levels is determined by considerations of precision and cost. The user must answer the question: "Is the additional precision obtained by using another test point worth the additional expense?" From Fig. A.1, it is reasonable to conclude that the minimum number of test points is about six, while more than nine tests would probably not be justified economically.

*Strictly speaking, Eq. (A-2) is applicable only for the case where the assumptions of ordinary least-squares analysis are satisfied. It is used here to demonstrate, in a simplified way, the effects of the number of test points on the width of the confidence interval. When using in-place testing, Eq. (A-16) in Appendix Section A.3 should be used to determine the lower confidence limit of the established mean value of Y for a new value of X.

A.2—Regression analysis with *X*-error (Mandel 1984)

If the procedures in Section 6.2.3 or 6.2.4 are to be used to estimate the in-place characteristic strength, the least-squares regression analysis procedure to determine the strength relationship should account for error in the *X*-variable. The method proposed by Mandel (1984) can be used for this purpose. This section provides a step-by-step procedure for carrying out Mandel's method.

At each strength level for the correlation tests there are n_x replicate in-place test results and n_y replicate compressive test results. The number of strength levels is N. The objective is to find the best-fit values of a and B (and their uncertainties) for the straight line, strength relationship

$$\ln C = a + B \ln I \quad \text{(A-4)}$$

where

a = intercept of straight line;
B = slope of straight line;
$\ln C$ = the natural logarithm of compressive strength; and
$\ln I$ = the natural logarithm of the in-place test result.

After the correlation test data have been obtained, the following sequence of calculations is used to establish the strength relationship and its uncertainty:

1. Transform the data by taking the natural logarithm of each test result

$$x = \ln i \quad \text{(A-5a)}$$

$$y = \ln c \quad \text{(A-5b)}$$

where i and c are the individual in-place and compressive strength test results, respectively.

2. For each strength level j, compute the average and standard deviation* of the logarithms of the in-place and compressive test results:

X_j = the average of the logarithms of the in-place tests at strength level j;
Y_j = the average of the logarithms of the compressive strength tests at strength level j;
s_{xj} = the standard deviation of the logarithms of the in-place tests at strength level j; and
s_{yj} = the standard deviation of the logarithms of the compressive strength tests at strength level j.

3. Calculate $(s_x)^2$ and $(s_y)^2$, which are the average variances (squares of the standard deviations) of the logarithms of the in-place tests and of the compressive tests, respectively.†

$$(s_y)^2 = \frac{\Sigma(s_{yj})^2}{N} \quad \text{(A-6a)}$$

*For a small number of replicate tests, the standard deviation may be estimated by multiplying the range by the following factors: 0.886 for two replicates, 0.591 for three replicates, and 0.486 for four replicates (Snedecor and Cochran 1967).

†Equations (A-6a) and (A-6b) assume that the same number of replicates were used at each strength level. If some test results were discarded because they were found to be outliers, the pooled variances should be computed to account for different numbers of replicates at each strength level (refer to Stone and Reeve [1986] or a textbook on introductory statistics.)

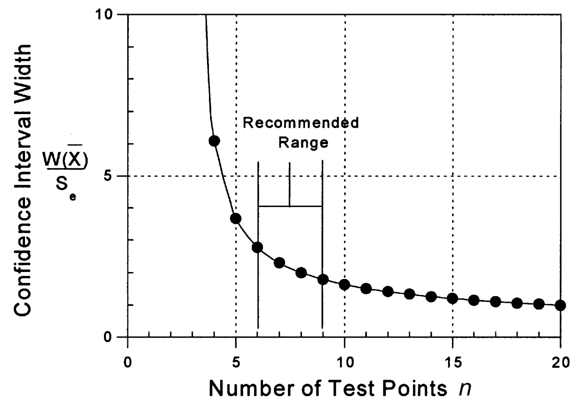

Fig. A.1—Effect of number of points used to establish strength relationship on the confidence interval width (in terms of residual standard deviation).

$$(s_x)^2 = \frac{\Sigma(s_{xj})^2}{N} \quad \text{(A-6b)}$$

4. Compute the value of λ as follows

$$\lambda = \frac{\dfrac{(s_y)^2}{n_y}}{\dfrac{(s_x)^2}{n_x}} \quad \text{(A-7)}$$

where

n_x = number of replicate in-place tests at each strength level; and
n_y = number of replicate compressive strength tests at each strength level.

The numerator and denominator in Eq. (A-7) are the variances of the average compressive strength and in-place results, respectively. If there are different numbers of replicate tests at each strength level, the average numbers of replications should be used for n_x and n_y (refer to Stone and Reeve [1986]).

5) Find the values of b and k by solving the following simultaneous equations‡

$$b = \frac{S_{xy} + kS_{yy}}{S_{xx} + kS_{xy}} \quad \text{(A-8a)}$$

$$k = \frac{b}{\lambda} \quad \text{(A-8b)}$$

In Eq. (A-8a), the terms S_{xx}, S_{yy}, and S_{xy} are calculated according to the following

$$S_{xx} = \Sigma(X_j - \bar{X})^2 \quad \text{(A-9a)}$$

‡An iterative procedure can be used to solve for k and b (Mandel 1984). First, assume a value of k, such as $k = 0$, and solve for b in Eq. (A-8a). Using this value of b, solve for a new value of k in Eq. (A-8b). Substitute the new value of k into Eq. (A-8a) and solve for b. Repeat the procedure until the values of k and b converge, which will usually occur in less than five iterations.

$$S_{yy} = \Sigma(Y_j - \bar{Y})^2 \quad \text{(A-9b)}$$

$$S_{xy} = \Sigma(X_j - \bar{X})(Y_j - \bar{Y}) \quad \text{(A-9c)}$$

The terms $\bar{X}$ and $\bar{Y}$ are the grand averages of the logarithms of the in-place and compressive strength test results.

$$\bar{X} = \frac{\Sigma X_j}{N} \quad \text{(A-10a)}$$

$$\bar{Y} = \frac{\Sigma Y_j}{N} \quad \text{(A-10b)}$$

6. The best-fit estimates of B and a are as follows

$$B = b \quad \text{(A-11a)}$$

$$a = \bar{Y} - b\bar{X} \quad \text{(A-11b)}$$

7. Use the following steps to compute the standard errors of the estimates of a and B.

a) Compute these modified sums of squares

$$S_{uu} = S_{xx} + 2kS_{xy} + k^2S_{yy} \quad \text{(A-12a)}$$

$$S_{vv} = b^2S_{xx} - 2bS_{xy} + S_{yy} \quad \text{(A-12b)}$$

b) Compute the following error of fit, s_e

$$s_e = \sqrt{\frac{S_{vv}}{N-2}} \quad \text{(A-13)}$$

c) The error in a is given by the following

$$s_a = s_e\sqrt{\frac{1}{N} + \frac{\bar{X}^2(1+kb)^2}{S_{uu}}} \quad \text{(A-14)}$$

d) The error in B is given by the following

$$s_B = s_e\frac{|1+kb|}{\sqrt{S_{uu}}} \quad \text{(A-15)}$$

In summary, the following general steps are used to obtain the best-fit strength relationship and account for the error in the X variable (in-place test results):

- Transform the correlation data by taking their natural logarithms;
- At each strength level, compute the average and standard deviation of the transformed values (logarithms);
- Compute the value of λ based on the average (or pooled) variances of the mean compressive and in-place results;
- Compute the values of b and k;
- Compute the slope and intercept of the best-fit relationship; and
- Compute the error of the fit.

The error of the fit s_e is needed to calculate the uncertainty in the estimated mean compressive strength when the strength relationship is used with in-place tests of the structure. This is explained in the next section.

A.3—Standard deviation of estimated *Y*-value (Stone and Reeve 1986)

The strength relationship is used to estimate the in-place compressive strength based on the results of the in-place tests done on the structure. Typically, several in-place tests are done on the structure, the average result is computed, and the strength relationship is used to estimate the average compressive strength. To obtain a reliable estimate of the average strength, that is, a value that has a high probability of being exceeded, the standard deviation of the estimate must be known.

The approach developed by Mandel (1984) can be used to estimate the standard deviation of an estimated value of Y (average compressive strength) for a new value of X (average in-place test results) when there is X-error. Mandel's method was modified by Stone and Reeve (1986) so that it also incorporates the uncertainty of the average in-place result from tests on the structure. This modification accounts for the fact that the uncertainty in the average of the in-place results is typically greater for tests on the structure compared with that from the laboratory tests used to develop the strength relationship. The standard deviation of the estimated value of Y (average of the logarithm of compressive strength) is obtained by the following equation

$$s_Y = \sqrt{\left[\frac{1}{N} + (1+kb)^2\frac{(X-\bar{X})^2}{S_{uu}}\right]s_e^2 + b^2\frac{s_x^2}{m}} \quad \text{(A-16)}$$

where

s_Y = standard deviation of estimated value of Y (average concrete strength);
N = number of points used to obtain the strength relationship;
b = estimated slope of the strength relationship;
k = b/λ, where λ is obtained from the within-test variability during correlation testing, Eq. (A-7);
X = average* of in-place tests done on the structure;
$\bar{X}$ = average of X values during correlation tests, Eq. (A-10a);
s_e = error of fit of strength relationship, Eq. (A-13);
S_{uu} = modified sum of the squares as given by Eq. (A-12a);
s_X = standard deviation* of in-place tests done on the structure; and
m = number of replicate in-place tests done on the structure.

It is seen that there are two sources of the uncertainty in the estimated value of Y:

1) the uncertainty of the strength relationship (s_e); and

*The average and standard deviation of the in-place results refer to the average and standard deviation of the logarithms of the test results.

Table A.1—Average, standard deviation, and variance of correlations data from Stone et al. (1986)

Average $\ln PO$ (PO in kN [lb])	Real value of PO, kN (lb)	Standard deviation $\ln PO$	Variance $\ln PO$	Average $\ln C$ (C in MPa [psi])	Real value of C, MPa (psi)	Standard deviation $\ln C$	Variance $\ln C$
1	2	3	4	5	6	7	8
2.2689 (7.6842)	9.67 (2174)	0.1085	0.0118	2.3413 (7.3183)	10.39 (1508)	0.0474	0.0022
2.4985 (7.9138)	12.16 (2735)	0.0459	0.0021	2.6522 (7.6292)	14.19 (2057)	0.0435	0.0019
2.8076 (8.2229)	16.57 (3725)	0.0700	0.0049	2.9273 (7.9043)	18.68 (2709)	0.0451	0.0020
2.9888 (8.4040)	19.36 (4465)	0.1065	0.0114	3.1275 (8.1047)	22.82 (3310)	0.0103	0.0001
3.2945 (8.7098)	26.97 (6062)	0.1162	0.0135	3.3440 (8.3209)	28.33 (4109)	0.0343	0.0012
3.3948 (8.8100)	29.81 (6701)	0.1488	0.0222	3.4551 (8.4321)	31.66 (4592)	0.0048	0.0005
3.5244 (8.9397)	33.93 (7629)	0.0953	0.0091	3.6890 (8.6660)	40.00 (5802)	0.507	0.0026
3.5725 (8.9877)	35.60 (8004)	0.1598	0.0255	3.7588 (8.7358)	42.90 (6222)	0.0303	0.0009
		Average variance of $\ln PO$	0.0125			Average variance of $\ln C$	0.0014

2) the uncertainty (s_X) of the in-place test results obtained from testing the structure.

Because Eq. (A-16) is the sum of two variances, which may have different degrees of freedom, a formula has been suggested for computing the effective degrees of freedom for s_Y (Stone and Reeve 1986). For simplicity, it can be assumed that there are (m-1) degrees of freedom associated with s_Y, where m is the number of in-place tests done on the structure. These degrees of freedom are used in choosing the t-value to calculate a lower confidence limit for the average value, as discussed in Section 6.2.4.

A.4—Example

An example is presented to show the application of Mandel's method and to illustrate the evaluation of in-place tests using the tolerance factor method discussed in Section 6.2.2 and the alternative method discussed in Section 6.2.4. The correlation data are taken from the study of the pullout test by Stone et al. (1986). The pullout test geometry had an apex angle of 70 degrees and the concrete was made using river gravel aggregate. Eight strength levels were used to develop the strength relationship. At each strength level, 11 replicate pullout tests and five replicate cylinder compressive tests were done. A soft conversion of the inch-pound values reported by Stone, Carino, and Reeve (1986) was used to obtain the corresponding SI values.

The data from the cited reference were converted by taking the natural logarithm of the individual pullout loads and compressive strengths. The average, standard deviation, and variance (square of standard deviation) of the transformed pullout loads at each strength level are shown in Columns 1, 3, and 4 of Table A.1. (SI and inch-pound versions of some tables are presented in this Appendix to reduce clutter.) The average, standard deviation, and variance of the transformed compressive strengths at each strength level are shown in Columns 5, 7, and 8. For information, Columns 2 and 6 give the averages of the logarithm values transformed into real units.

The average values in Columns 1 and 5 of Table A.1 were used to calculate the various parameters to establish the strength relationship according to the procedure in Appendix A.2. A computer spreadsheet was set up to do these calculations. Table A.2 summarizes the calculated values.

Table A.2—Summary of results of regression calculations using values in Table A.1 and procedure in Appendix A.2

Parameter	Value, SI units (in.-lb units)	Parameter	Value, SI units (in.-lb units)
N	8 (8)	k	4.287 (4.284)
n_x	11 (11)	$b = B$	1.030 (1.030)
n_y	5 (5)	a	0.0268 (–0.5747)
$\bar{X}$	3.0438 (8.4590)	e^a	1.027 (0.563)
$\bar{Y}$	3.1619 (8.1389)	S_{uu}	48.155 (48.104)
λ	0.240 (0.240)	S_{vv}	0.0180 (0.0180)
S_{xx}	1.6530 (1.6528)	s_e	0.0548 (0.0548)
S_{yy}	1.7423 (1.7424)	s_a	0.1317 (0.3622)
S_{xy}	1.6883 (1.6883)	s_B	0.0428 (0.0428)

The calculated values of a and B are shown in the last column of Table A.2. Therefore, the equation of the strength relationship is as follows

$$\text{SI units: } C = 0.0268 + 1.030PO \quad \text{(A-17a)}$$

$$\text{Inch-pound units: } C = -0.5747 + 1.030PO \quad \text{(A-17b)}$$

where

C = average of natural logarithms of compressive strengths; and

PO = average of natural logarithms of pullout loads.

Figure A.2 shows the correlation data (average of logarithms) and the best-fit line.

Finally, the strength relationship and the procedures in Section 6.2 are used to estimate the in-place compressive strength based on in-place test results. Table A.3 shows two sets of in-place pullout test results. Both cases have approximately the same average value, but Case 2 has higher variability. In each case, there are 10 replicate test results, that is, $m = 10$. The pullout loads are transformed by taking their natural logarithms. The averages of the logarithms, $\ln PO$, are substituted into Eq. (A-17) to obtain the average of the logarithm of in-place compressive strength, $\ln C$. Estimates of the tenth percentile strength ($Y_{0.10}$) corresponding to the two cases are obtained using the tolerance factor method (Section 6.2.2) and the alternative method (Section 6.2.4). The values of the various parameters used in

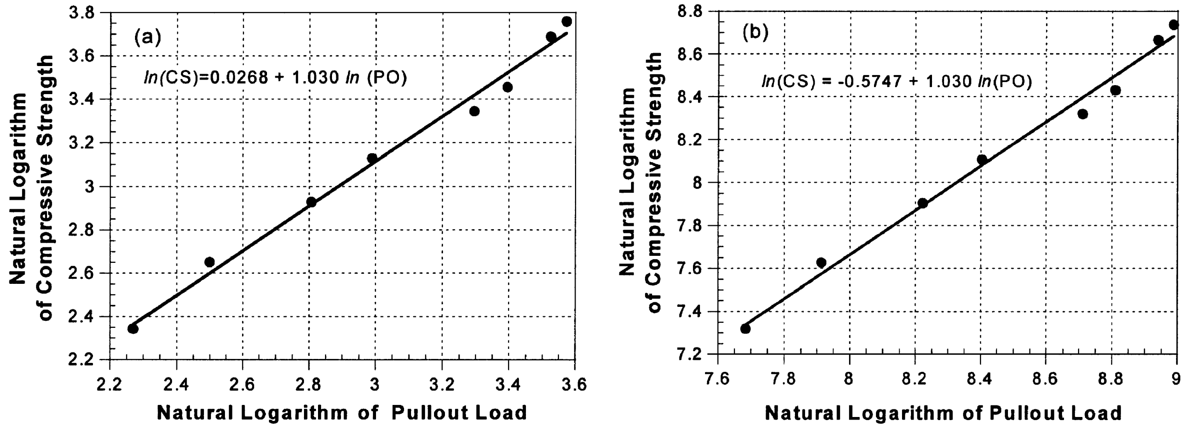

Fig. A.2—Data for strength relationship and best-fit line: (a) SI units; and (b) inch-pound units.

Table A.3—Values of pullout force obtained from tests on structures

In SI units:

Case 1		Case 2	
Pullout force, kN	ln*PO*	Pullout force, kN	ln*PO*
13.39	2.5944	17.37	2.8545
14.86	2.6985	12.78	2.5479
15.57	2.7453	14.25	2.6569
13.70	2.6174	11.87	2.4742
11.02	2.4000	10.37	2.3392
13.34	2.5911	13.75	2.6210
14.63	2.6834	17.10	2.8390
13.66	2.6142	13.97	2.6367
11.83	2.4708	11.35	2.4294
11.83	2.4708	14.84	2.6973
Average (X)	2.5886	Average (X)	2.6096
Standard deviation (s_X)	0.1108	Standard deviation (s_X)	0.1670

In in.-lb units:

Case 1		Case 2	
Pullout force, lb	ln*PO*	Pullout force, lb	ln*PO*
3010	8.0097	3904	8.2698
3340	8.1137	2873	7.9631
3500	8.1605	3204	8.0722
3080	8.0327	2669	7.8895
2478	7.8152	2332	7.7545
3000	8.0064	3091	8.0362
3290	8.0986	3844	8.2543
3070	8.0294	3140	8.0520
2660	7.8861	2552	7.8446
2660	7.8861	3336	8.1125
Average (X)	8.0038	Average (X)	8.0249
Standard deviation (s_X)	0.1108	Standard deviation (s_X)	0.1670

the calculations are summarized in Table A.4, and, where appropriate, the corresponding equation numbers are shown. For the alternative method, the standard deviation of the in-place compressive strength (s_{cf}) was computed using Eq. (6-6), while for the tolerance factor method it was taken to equal the standard deviation of the transformed in-place test results. For each method, the value of $Y_{0.10}$ is a smaller fraction of the average strength for Case 2 due to the higher variability of the in-place tests. In this example, the strength relationship has relatively low scatter, and the estimates of $Y_{0.10}$ are lower for the tolerance factor method, which does not consider this.

Table A.4—Estimate of in-place compressive strength using results in Table A.3

In SI units:

	Alternative approach (Section 6.2.4)			Tolerance factor approach (Section 6.2.2.)	
	Case 1	Case 2		Case 1	Case 2
Y (Eq. (A-17a))	2.6930	2.7147	Y	2.6930	2.7147
exp(Y), MPa*	14.78	15.10	exp(Y), MPa	14.78	15.10
s_Y (Eq. (A-16))	0.0454	0.0607	K (p = 0.75)	1.671	1.671
$t_{9,0.05}$	1.833	1.833	s_{cf}	0.111	0.167
Y_{low} (Eq. (6-4))	2.6098	2.6034	$Y_{0.10}$ (Eq. (6-1))	2.5075	2.4356
s_{cf} (Eq. (6-6))	0.037	0.055	exp($Y_{0.10}$), MPa	**12.27**	**11.42**
exp($Y_{0.10}$) (Eq. (6-5))	2.5628	2.5326			
exp($Y_{0.10}$), MPa	**12.97**	**12.59**			

In in.-lb units:

	Alternative approach (Section 6.2.4)			Tolerance factor approach (Section 6.2.2.)	
	Case 1	Case 2		Case 1	Case 2
Y (Eq. (A-17b))	7.6700	7.6917	Y	7.6700	7.6917
exp(Y), psi*	2143	2190	exp(Y), psi	2143	2190
s_Y (Eq. (A-16))	0.0454	0.0607	K (p = 0.75)	1.671	1.671
$t_{9,0.05}$	1.833	1.833	s_{cf}	0.111	0.167
Y_{low} (Eq. (6-4))	7.5870	7.5804	$Y_{0.10}$ (Eq. (6-1))	7.4845	7.4126
s_{cf} (Eq. (6-6))	0.037	0.055	exp($Y_{0.10}$), psi	**1780**	**1657**
exp($Y_{0.10}$) (Eq. (6-5))	7.5395	7.5099			
exp($Y_{0.10}$), psi	**1881**	**1826**			

*exp(Y) = e^Y.