

ACI 228.1R-19

Report on Methods for Estimating In-Place Concrete Strength

Reported by ACI Committee 228



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Report on Methods for Estimating In-Place Concrete Strength

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Report on Methods for Estimating In-Place Concrete Strength

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This report provides methods for estimating the in-place strength of concrete in new and existing construction. These methods include: rebound number, penetration resistance, pullout, pull-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 A 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 safety; in-place tests; nondestructive tests; 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, such as rebound number and ultrasonic pulse velocity, were noninvasive and did not damage the concrete. Over the years, however, new methods have developed that result in superficial local damage. There-

fore, the terminology “in-place tests” is used as a general name for these test methods, which includes those that do not damage the concrete and those that result in some near-surface damage. In this report, the principal application of in-place tests is to estimate the compressive strength of the concrete. The pull-off test can be used to estimate the tensile strength of concrete or evaluate bond strength between layers. 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 in-place test results and the uncertainty of the relationship between these two parameters. These sources of uncertainty are discussed in this report. An alternative approach for correlation between tests results and concrete strength is presented in EN 13791 (2007) and BS 6089 (2010).

In-place tests can be used to estimate concrete strength during construction so that operations requiring a specific strength can be performed safely or curing procedures 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 et al. 2006; Malhotra and Carino 2004). No attempt is made to review all methods in this report; only those methods that have been standardized by ASTM International are discussed. Examples of methods not covered include internal fracture tests (Chabowski and Bryden-Smith 1980; Domone and Castro 1987) and torque tests (Stoll 1985).

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 standard cylinders (ASTM C39/C39M). This test procedure is relatively easy to perform in terms of sampling, specimen preparation (ASTM C31/C31M), and strength measurement. When properly performed, this test has low single-operator variation and low interlaboratory variation and, therefore, the method lends itself to use as a standardized testing procedure. The compressive strength so obtained is used to verify that the specified strength (f'_c) used to calculate the nominal strengths of structural members has been achieved. Therefore, the compressive strength of standard cylinders is an essential parameter in design codes and project specifications.

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 assurance 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, consolidation, 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 flexural members, especially pretensioned members, can be required to support a large percentage of the design load before an age of 28 days. 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, a 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 C31/C31M**. 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 should 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 keep the project on schedule. 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. 1983a**). 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. In 1983, ACI 318 first allowed the use of other procedures instead of tests for field-cured cylinders, if approved by the building official (**ACI Committee 318 1983**). The design professional, when requested by the building official, however, was required to 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 if approved by the licensed design professional and, if requested, approved by the building official. The commentary to **ACI 318-14** (Section R26.11.2.1(e)) lists four procedures, which are covered in this report, that may be used provided there are sufficient correlation data.

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 is required. At present, there are no standard practices for developing the required relationship.

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, a specification for new concrete construction. In the 2016 version of **ACI 301**, Section 1.6.4.2 on in-place testing of hardened concrete includes the following:

Use of the rebound hammer in accordance with ASTM C805/C805M or the pulse-velocity method in accordance with ASTM C597 may be specified by Architect/Engineer to evaluate uniformity of in-place concrete or to select areas to be cored. These methods shall not be used to evaluate in-place strength.

Regarding the validity of in-place strength tests, ACI 301-16 states in Section 1.6.5.3(a):

Results of in-place strength tests will be evaluated by Architect/Engineer and are valid only if tests are conducted using properly calibrated equipment in accordance with recognized standard procedures and an acceptable correlation between test results and concrete compressive strength is established and submitted.

Section 1.6.6.4 of ACI 301-16, however, restricts the use of these tests in acceptance of concrete by stating that, "In-place tests shall not be used as the sole basis for accepting or rejecting concrete," but they may be used if specified to "evaluate" concrete if standard-cured cylinder strengths fail to meet the specified strength criteria.

ACI 301-16 also mentions in-place tests in Section 2.3.4 dealing with required strength for removal of formwork. Specifically, it is stated that if specified in Contract Documents, the following methods may be used to estimate in-place strength:

- (a) **ASTM C873/C873M** (cast-in-place cylinders)
- (b) **ASTM C803/C803M** (penetration resistance)
- (c) **ASTM C900** (pullout)
- (d) **ASTM C1074** (maturity method)

These same methods may be used, if specified, as alternatives to testing field-cured cylinders for estimating in-place strength for the purpose of terminating curing procedures.

ACI 562-16, the repair code for existing concrete buildings, allowed the use of in-place test methods for assessment of concrete strength. Section 6.4.3.2 states:

Nondestructive strength testing to evaluate in-place strength of concrete shall be permitted if a valid correlation is established with core sample compressive strength test results and nondestructive test results. Quantifications of concrete compressive strength by NDT alone shall not be permitted as a substitute for core sampling and testing.

ACI 308.1 also mentions in-place tests as acceptable methods for estimating in-place strength for the purpose of terminating curing procedures. Project specifications can, therefore, reference standard specifications that allow in-place testing as an alternative to testing field-cured cylinders. ACI 325.11R discusses the use of in-place tests for estimating early-age concrete strength in fast-track concrete paving projects.

In all cases, sufficient correlation data are required, and permission may have to be granted before using in-place test methods. This report explains how the required correlation data can be acquired and 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 and ACI 562). Historically, in-place strength has been estimated by testing cores drilled from the structure. In-place tests can supplement coring and permit more economical evaluation of concrete in the structure. The critical step in such applications is to establish the relationship between in-place test results and concrete strength. More recently, the approach is to correlate results of in-place tests performed at selected locations with the strength of corresponding cores. Although in-place testing does not eliminate the need for coring, it can reduce the total amount of coring needed to evaluate a large volume of concrete. A sound sampling plan is required to acquire the correlation data, and appropriate statistical methods should be used for reliable interpretation of test results.

1.6—Report objective

This report reviews ASTM test methods for estimating the in-place strength of concrete in new construction and 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 includes the notation and definitions of terms used in this report. Chapter 3 discusses the underlying principles and inherent limitations of in-place tests. Chapter 4 reviews the statistical characteristics of in-place tests. Chapter 5 outlines procedures to develop the relationship needed to estimate in-place compressive strength. Chapter 6 discusses factors to be considered in planning the in-place testing program. Chapter 7 presents statistical techniques to interpret in-place test results. Chapter 8 discusses in-place testing for acceptance of concrete. Appendix A provides details and an illustrative example on the statistical principles discussed.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

a	= intercept of line
b	= slope of line, used in development of strength relationship
C	= average of in-place compressive strengths
c	= individual compressive strength test results
d_{yx}	= deviation of each test point from the best-fit line
I	= in-place test result
i	= individual in-place strength test results
$\ln C$	= average of natural logarithms of compressive strengths
$\ln I$	= average of natural logarithms of in-place test results
$\ln PO$	= average of natural logarithms of pullout loads
K	= one-sided tolerance factor (Table 6.1.2a)
k	= b/λ , where λ is obtained from the single-operator variability during correlation testing as given by Eq. (A.2d).
$M(t)$	= temperature-time factor at age t , deg-days or deg-hours
m	= number of replicate in-place tests performed on an element of a structure
N	= number of test points used to establish the strength relationship
n_i	= number of replicate in-place tests
n_x	= number of replicate in-place tests at each strength level
n_y	= number of replicate compressive strength tests at each strength level
PO	= average of natural logarithms of pullout loads
Q	= apparent activation energy divided by the gas constant, K (kelvin)
S_e	= estimated residual standard deviation
S_{uu}	= modified sum of the squares as given by Eq. (A.2i(a))
S_{xx}	= sum of squares of the deviations about $\bar{X}$ of the X values used to develop the strength relationship, $S_{xx} = \sum (X - \bar{X})^2$
s_{cf}	= standard deviation of the logarithms of concrete strength in the structure
s_{cl}	= standard deviations of logarithm of compressive strength in the laboratory
s_e	= error of fit of strength relationship as given by Eq. (A.2j)
s_{il}	= standard deviation of logarithms of the in-place results in the laboratory
s_X	= standard deviation of the logarithms of the in-place tests performed on the structure
s_{xj}	= standard deviation of the logarithms of the in-place tests at strength level j
s_Y	= standard deviation of estimated value of Y (average concrete strength)
s_{yj}	= standard deviation of the logarithms of the compressive strength tests at strength level j
T_a	= average concrete temperature during time interval Δt , °C (°F) or K
T_0	= datum temperature, °C (°F)
T_s	= specified curing temperature, °C (°F) or K

- t_e = equivalent age at a specified temperature T_s , days or hours
 $t_{m-1,\alpha}$ = Student's t -value for $m-1$ degrees of freedom and confidence level α
 $t_{N-2,\alpha/2}$ = Student's t -value for $N-2$ degrees of freedom and significance level α
 V_i = coefficient of variation of in-place test
 V_s = coefficient of variation of standard test
 V_Y = coefficient of variation (expressed as a decimal)
 W = width of the $100(1 - \alpha)$ percent confidence interval for the estimated mean value of Y for the value X
 X = average of the logarithms of the in-place tests performed on the structure
 X_j = average of the logarithms of the in-place tests at strength level j
 $\bar{X}$ = grand average of X values during correlation tests, Eq. (A.2g(a))
 Y = logarithm of the estimated average in-place compressive strength
 $\bar{Y}$ = grand average of Y values during correlation tests, Eq. (A.2g(b))
 $Y_{0.10}$ = lower tenth percentile of strength (10 percent defective)
 Y_j = average of the logarithms of the compressive strength tests at strength level j
 Y_{low} = lower confidence limit of estimated average in-place strength at confidence level α
 Δt = time interval, days or hour
 λ = ratio of variance of average compressive strength test results to variance of average in-place test results

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, ACI Concrete Terminology. Definitions provided herein complement that source.

apparent activation energy—an empirical factor that indicates the temperature sensitivity of the rate of strength development of a cementitious mixture after final setting has occurred.

datum temperature—the temperature value that is used for calculating the temperature-time factor.

equivalent age—the number of days or hours of curing of a concrete mixture at a specified temperature required to produce a maturity equal to the maturity achieved by a given curing period at concrete temperatures different from the specified temperature.

lower confidence limit—the value above which the true average, in-place strength is expected to occur with a specified probability or confidence level.

lower tenth percentile strength—the strength below which 10 percent of test results are expected to occur.

maturity—the extent of the development of a property of a cementitious mixture.

maturity function—a mathematical expression that uses the measured temperature history of a cementitious mixture during the curing period to calculate an index that is indicative of the maturity at the end of that period.

maturity index—an indicator of maturity, such as equivalent age or temperature-time factor, that is calculated from the temperature history of the cementitious mixture by using a maturity function.

residual standard deviation—a measure of the scatter of test data about a regression line; the standard deviation of the test values from the values estimated from the regression line.

single-operator variability—the standard deviation or coefficient of variation of replicate test determinations obtained on identical test specimens by a single operator using the same apparatus.

strength relationship—empirical equation obtained from regression analysis of test data that relates the compressive strength of concrete to the result of an in-place test method.

temperature-time factor—the maturity index computed as the area between the concrete temperature and the datum temperature from the plot of measured concrete temperature versus time, expressed in units of degree-days or degree-hours.

CHAPTER 3—REVIEW OF METHODS

3.1—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 concrete strength. For new construction, this relationship is usually established empirically in the laboratory. For existing construction, the relationship is conventionally established by performing in-place tests at selected locations in the structure and determining the strength of cores drilled from adjacent locations. **ACI 214.4R** discusses the number of core specimens needed to obtain, at a given confidence level, an estimate of average strength that is within a certain percentage of the true average strength. Figure 3.1 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 understand 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 et al. (2006)**, and **Malhotra and Carino (2004)**.

The following methods are discussed:

- Rebound number
- Penetration resistance
- Pullout test
- Pull-off test
- Ultrasonic pulse velocity
- Maturity
- Cast-in-place cylinder



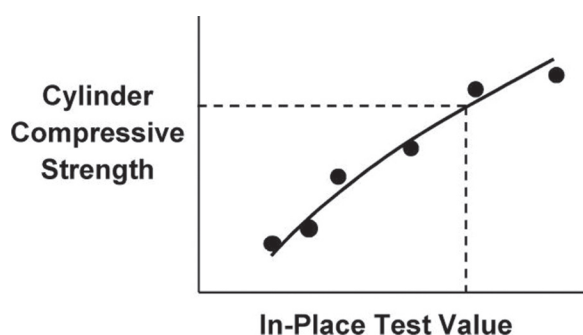


Fig. 3.1—Schematic of relationship between cylinder compressive strength and in-place test value.

3.2—Rebound number (ASTM C805/C805M)

Operation of the rebound hammer, also called the Schmidt Hammer or Swiss Hammer, is illustrated in Fig. 3.2. The device consists of the following main components: 1) outer body; 2) metal plunger and guide rod; 3) hammer; and 4) impact spring. The following sequence describes how the test is performed. From the initial locked condition, the plunger is placed in contact with the concrete surface and the plunger is extended from the body of the instrument by means of a compression spring. A latch locks the hammer in place and the instrument is ready for testing (Fig. 3.2(a)). Then, the body is pushed toward the test object, which extends the impact spring connected between the hammer and body (Fig. 3.2(b)). When the body is pushed to its limit of travel, the hammer is released and travels toward the test object due to the energy stored in the stretched impact spring (Fig. 3.2(c)). Finally, the hammer strikes the plunger and rebounds a certain distance along the guide rod (Fig. 3.2(d)). The rebound number can be based on the rebound distance expressed as a percentage of the stretch distance of the impact spring, or it can be based on the ratio of the hammer speed after impact to the hammer speed before impact with the plunger.

The key to understanding the inherent limitations of this test for estimating strength is recognizing the factors influencing the rebound number. The impact loading and resulting wave propagation within the hammer-concrete system result in a complicated dynamic system that is difficult to model from a fundamental point of view.

The rebound number depends on the kinetic energy in the hammer before impact with the plunger and the amount of that energy absorbed during 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. The latter factor is what makes the rebound number an indicator of the concrete properties. Energy absorbed by the concrete depends on the stress-strain relationship of the concrete. Therefore, absorbed energy is related to the strength and 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 likely 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.

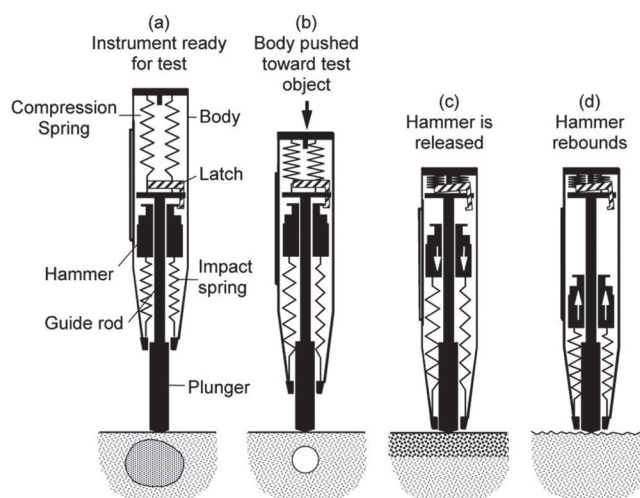


Fig. 3.2—Schematic illustrating operation of rebound hammer.

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 and the volume of coarse aggregate affect the stiffness of concrete, it is necessary to develop the strength relationship on concrete made with the same aggregates and similar proportions 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 surface conditions, local variations in concrete consolidation, and relative stiffness of the member at the location where the test is performed. If the plunger is located over a hard-aggregate particle, an unusually high rebound number will result. By contrast, if the plunger is located over a large air void or a soft aggregate particle, a lower rebound number will occur. Reinforcing bars with shallow concrete cover could also affect rebound numbers if tests are done directly over the bars. To account for these possibilities, **ASTM C805/805M** requires that 10 rebound numbers be taken for a test. If a reading differs by more than six units from the average, that reading is discarded and a new average is 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 might not represent the interior concrete. The presence of surface carbonation 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 can also influence the rebound number. When the test is performed on rough concrete, 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; the strength relationship has to be developed for the same instrument orientation as will be used for in-place testing.

In summary, while the rebound number test is simple to perform, there are many factors other than concrete strength that influence test results. As a result, estimated compressive strengths are unreliable unless a correlation is developed between rebound number and compressive strength for a given concrete mixture or from cores taken from a structure. Refer to [Chapters 5](#) and [6](#) for additional information on developing the relationship and on using the relationship to estimate in-place strength.

3.3—Penetration resistance (ASTM C803/C803M)

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

The probe-penetration technique involves the use of a specially designed gun to drive a hardened steel probe into the concrete. One well-known commercial test system is the Windsor Probe. The penetration depth of the probe is an indicator of concrete strength. This method is similar to the rebound number test, except the probe impacts the concrete with much higher energy than the plunger of the rebound hammer. The probe penetrates the concrete, compared with the plunger of the rebound hammer, which only produces 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 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 C803/C803M](#) requires that the probe exit-velocities do not have a coefficient of variation greater than 3 percent based on 10 tests by approved ballistic methods.

As the probe penetrates the concrete, some energy is absorbed by friction between the probe and the concrete, and some is absorbed by crushing and fracturing the concrete. There are no rigorous studies of factors affecting the geometry of the fracture zone, but its general shape is most likely similar to Fig. 3.3a. There is usually a cone-shaped region in which the concrete is heavily fractured, and a significant portion of the probe energy is absorbed in this zone.

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 penetration distance. This contrasts with the

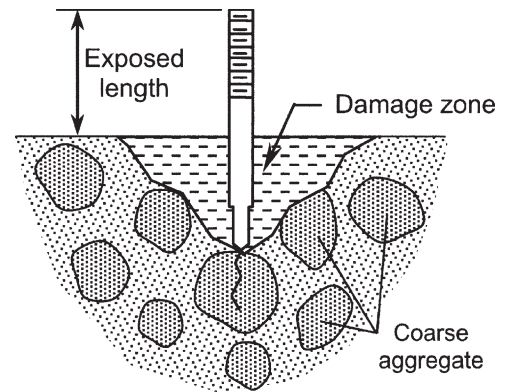


Fig. 3.3a—Approximate shape of failure zone in concrete during probe penetration test.

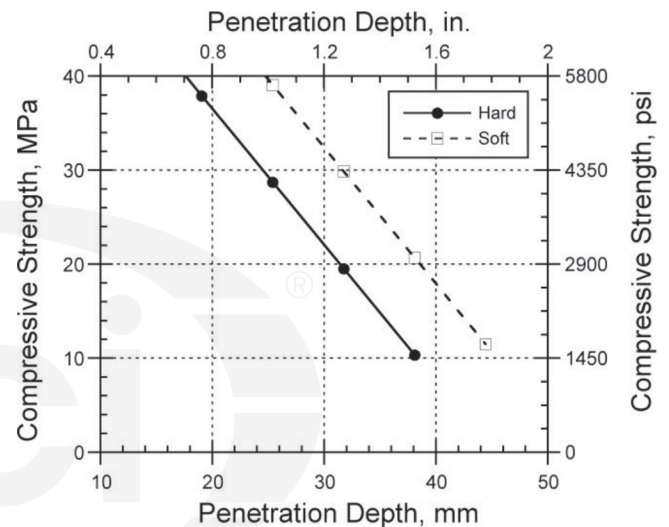


Fig. 3.3b—Effect of aggregate type on relationship between concrete strength and depth of probe penetration.

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 affects greatly the relationship between concrete strength and depth of probe penetration. For example, Fig. 3.3b 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 concrete with hard aggregate. More detailed information on the influence of aggregate type on strength relationships is found in [Malhotra \(1976\)](#), [Bungey et al. \(2006\)](#), and [Malhotra and Carino \(2004\)](#).

Penetration values may be affected by surfaces with coarse textures. ASTM C803/C803M requires grinding of surfaces that are coarser than burlap-dragged finishes before testing. A hard surface layer, as would occur with trowel finishing, can result in low penetration values and excessive scatter of data. The orientation in which the test is performed is not critical provided the probe is driven perpendicular to the

surface. The penetration, however, 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. Cover meters 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 resistance to penetration. Therefore, when assessing the variability of test results (Chapter 4), 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 approximately 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, which uses 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 C803/C803M. A spring-loaded device is used to drive a pointed 3.56 mm (0.140 in.) diameter hardened steel pin into the concrete. Penetration by the pin creates a small indentation (or hole) in the concrete surface. The pin is removed from the hole, the hole cleaned with an air jet, and the hole depth measured with a suitable depth gauge. 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 approximately 1.3 percent of the energy delivered by the Windsor Probe system (Carino and Tank 1989). Because of low energy level, 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 C803/C803M). A pin can become blunted during penetration. Because the degree of blunting affects penetration depth, ASTM C803/C803M requires that a new pin be used for each penetration test.

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 an expected 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.

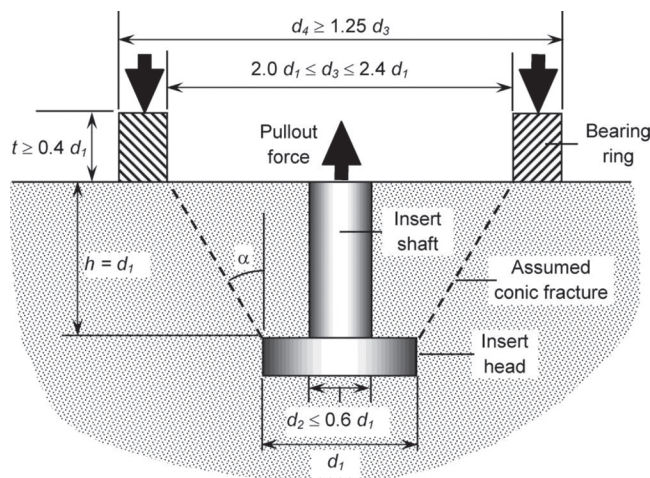


Fig. 3.4a—Dimensional requirements of pullout test in accordance with ASTM C900.

3.4—Pullout test (ASTM C900)

The pullout test measures the maximum force required to pull an embedded metal insert with an enlarged head from a concrete specimen or structure. A positive feature of the pullout test is that it produces a well-defined failure surface in the concrete and measures a static, rather than dynamic, strength property of the concrete.

The test requires a metal insert that is either cast into fresh concrete or installed into hardened concrete. The pullout force is applied by a loading system that reacts against the concrete surface through a reaction ring concentric with the insert (Fig. 3.4a). 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 given in ASTM C900. The embedment depth and head diameter have to 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 (2α) 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 has to 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; Ferretti 2004). 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 3.4b shows the approximate directions (trajectories) of the principal stresses acting in radial plane through the center of the insert before cracking for apex

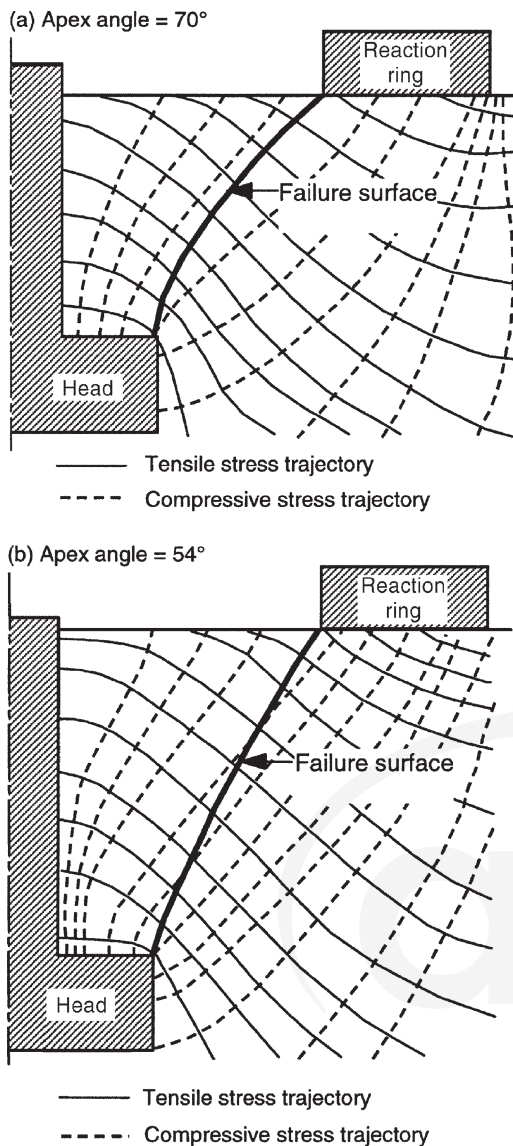


Fig. 3.4b—Principal stress trajectories before cracking for pullout test in a homogeneous material and measured fracture surfaces in physical tests (Stone and Carino 1984).

angles of 54 and 70 degrees. Because of symmetry, only one-half 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. There

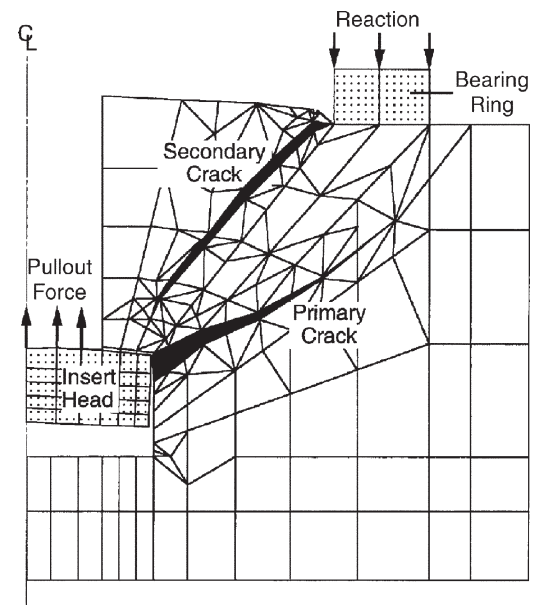


Fig. 3.4c—Circumferential cracks predicted by nonlinear fracture mechanics analysis of pullout test by Hellier et al. (1987).

is no agreement 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 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 et al. (1986) concluded that ultimate pullout 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 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 other studies, using nonlinear fracture mechanics and discrete cracking models, Hellier et al. (1987) and Ferretti (2004) showed excellent agreement between the predicted and observed internal cracking in the pullout test. Figure 3.4c shows the displaced shape of the finite-element model used by Hellier et al. (1987). The analyses by Hellier et al. and Ferretti indicate 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 appears

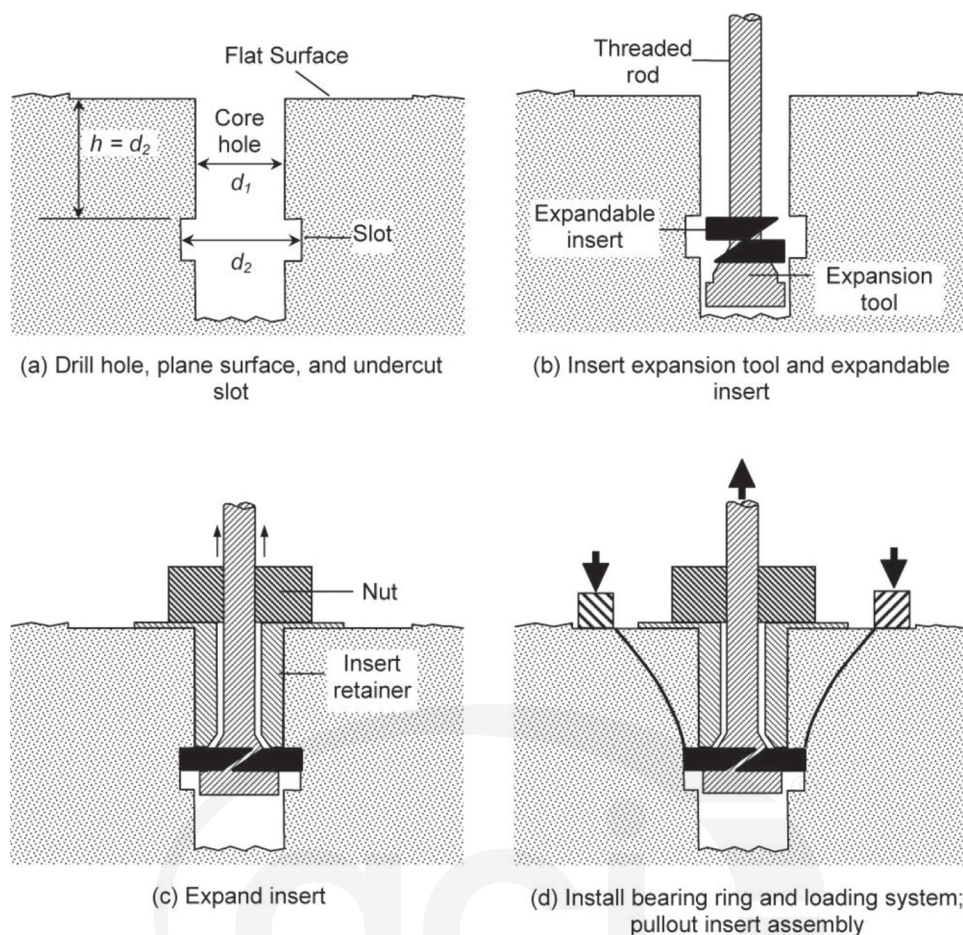


Fig. 3.4d—Technique for post-installed pullout test (adapted from ASTM C900).

to coincide with the final fracture surface observed when the conical fragment is extracted from the concrete mass during pullout testing. These studies also concluded that the ultimate pullout load is not governed by uniaxial compressive failure in the concrete.

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 approximately 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 single-operator coefficient of variation of these pullout tests has been found to be between approximately 4 and 15 percent.

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. If inserts are placed manually, care is required to maintain representative concrete properties at placement locations and to reduce the amount of air that is entrapped on the underside of the plates. In an early

study, [Vogt et al. \(1984\)](#) reported higher-than-expected single-operator variability when using manually placed inserts. Subsequent work by [Dilly and Vogt \(1988\)](#) 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 taken 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, which was added to [ASTM C900](#) in 1999, is summarized in Fig. 3.4d. The procedure involves five basic steps:

- (1) Drilling a hole perpendicular to the surface of the concrete using a core drill
- (2) Grinding the test area to make it flat
- (3) Undercutting a slot to engage an expandable insert
- (4) Expanding an insert into the milled slot
- (5) Pulling the insert out of the concrete

The test geometry is the same as for the [ASTM C900](#) insert. In a commercial test system, known as the CAPO

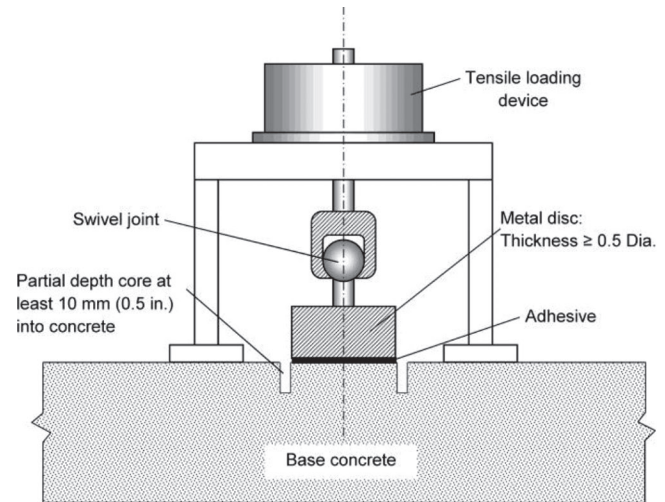
(Cut and Pull Out) test, the insert is a coiled, split ring that is expanded with specially designed hardware. The CAPO system performs similarly to the cast-in-place system of the same geometry (Petersen 1984, 1997). Care is required during preparation to ensure that the hole is drilled perpendicular to the test surface. The surface has to 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 can result in an incomplete circle for the top surface of the extracted frustum. If this occurs, the test result must be rejected (ASTM C900). 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).

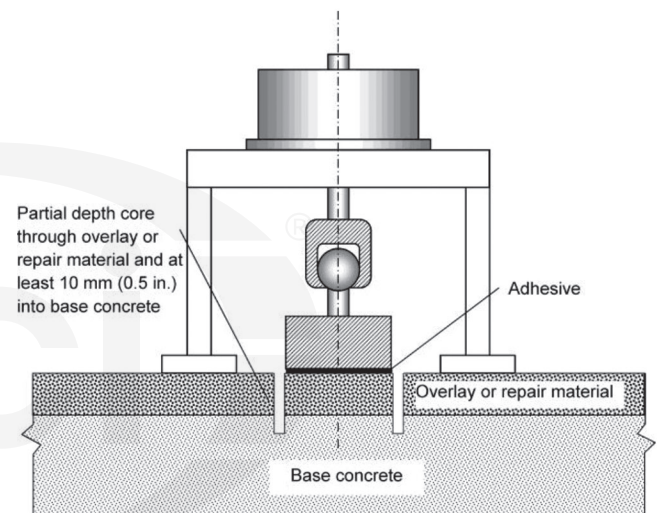
In summary, the pullout test can be used to estimate the strength of concrete by measuring the force required to extract an insert embedded in fresh concrete or installed in hardened concrete. The test results in a complex, three-dimensional state of stress in the concrete. Although there is no consensus on the exact failure mechanism of the pullout test, there is a strong relationship between the compressive strength and pullout strength of concrete.

3.5—Pull-off test (ASTM C1583/C1583M)

The pull-off test, as described in ASTM C1583/C1583M, is used to determine the near-surface tensile strength of the substrate as an indicator of the adequacy of surface preparation before application of a repair or overlay material; the bond strength of a repair or an overlay material to the substrate; and the tensile strength of a repair or overlay material, or an adhesive used in repairs, after the material has been applied to a surface (Long and Murray 1984). The pull-off method is illustrated in Fig. 3.5. In the ASTM test procedure, the surface to be tested is cored to a depth of at least 10 mm (0.5 in.), as shown in Fig. 3.5(a). If testing the bond strength of an overlay or a concrete repair, the core depth is extended to at least 10 mm (0.5 in.) below the concrete overlay interface, as shown in Fig. 3.5(b). A metal disc is then bonded to the test surface using a suitable adhesive. A temporary packing material can be placed in the kerf to prevent the adhesive from filling in the annular cut. The disc is then pulled from the surface at a constant rate of 35 ± 15 kPa/s (5 ± 2 psi/s) using a device that ensures the force is parallel to and coincident with the axis of the core. The system to provide reaction to the applied load can be a ring



(a) Test of base concrete



(b) Test of material applied to base concrete

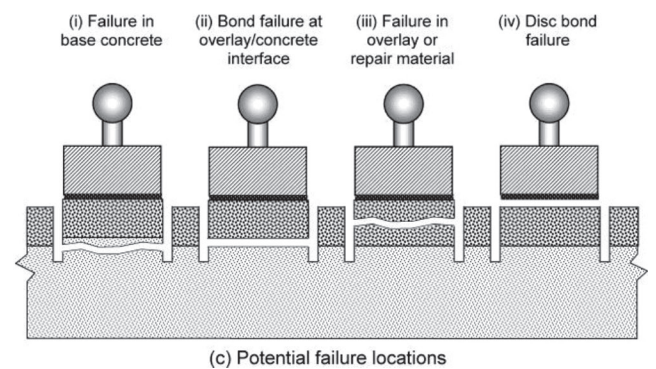


Fig. 3.5—Schematic of pull-off test and potential failure locations (adapted from ASTM C1583/C1583M).

or tripod. Courard et al. (2014) investigated the influence of pull-off force misalignment and concluded that a 4-degree misalignment could reduce the measured pull-off strength by approximately 15 percent.

An alternative procedure (not standardized as of 2016) is to first prepare a flat surface using a planer or grinder without water, remove the dust, bond the disc, and then cut

the partial-depth core. This avoids waiting for the surface to dry after cutting the core and before bonding the disc. This procedure, however, requires a core barrel that is properly sized for the disc being used so that the partial-depth core is centered with the disc.

The failure location in the pull-off test will be in the material with the lowest strength. As shown in Fig. 3.5(c), the failure location can be at the concrete surface (disc bond failure), at the interface between materials (bond failure), in the overlay material (overlay failure), or in the base material (substrate failure). Because the failure location cannot be predicted before the test is complete, both the failure load and location are recorded after completion of the test. The average strength in the pull-off test is based on test results with the same failure location.

The pull-off test was initially developed at Queen's University Belfast in the 1970s for strength testing of in-place concrete. Initial research resulted in the development of LIMPET test equipment (Bungey and Madandoust 1992), although several types of commercial equipment are available. Early research by Bungey and Madandoust (1992) examined the effect of the disc material, disc size, effect of coring depth, reaction system, and loading rate. These factors were examined in a series of experimental studies and finite-element simulations. This research showed that test results are sensitive to different disc materials (steel or aluminum), disc diameter and thickness, and core depth. Experimental results showed that pull-off strengths were not influenced by whether the loading system used a ring or tripod arrangement to provide reaction to the applied load. The results indicated that a 50 mm (2 in.) diameter disc with a disc thickness of 30 mm (1.2 in.) will produce a reasonably uniform concrete stress distribution regardless of the elastic properties of the concrete or disc material. ASTM C1583/C1583M requires a steel disc diameter of 50 mm (2 in.) with a thickness of at least 25 mm (1 in.).

The pull-off test has been used widely in evaluating the bond strength between materials in repair applications (ICRI 210.3R). In this application, the test can be used to assess the surface preparation quality, as well as bond strength, between a repair material and substrate. The ICRI reference provides a description of the various failure mechanisms that can occur when used in this application. The pull-off test was used extensively to assess the effect of different concrete removal or surface preparation methods on the bond of repair materials to the concrete substrate (Hindo 1990). The research by Hindo identified that "bruising," or micro-fracturing, of the substrate surface layer during concrete removal has a significant effect on the bond strength of repair materials, with lower bond strengths observed with more aggressive removal methods. The more aggressive removal methods result in damage to the concrete substrate, often resulting in bond failures immediately below the interface between the repair material and substrate.

Results from three different pull-off test devices were compared by Vaysburd and McDonald (1999). Two commercially available pull-off devices and a modified device were used in the research. The modified device was

a commercially available instrument designed to assess the pull-out strength of adhesive anchors and had been modified for pull-off testing. Pull-off tests were performed on a concrete specimen that included a simulated concrete repair. The research program included an examination of the effects of coring depth, with linear elastic, finite-element analyses performed to examine the stress concentrations that occur with different coring depths. Test results from the two commercially available pull-off test devices were found to be similar, with lower pull-off strengths obtained from the modified device. The overall results of the pull-off tests had high variability, with a preferential failure location observed in the concrete substrate, rather than in the repair material or at the bond line between materials. The results obtained in this study were used for the precision statement in ASTM C1583/1583M (4.2.4).

In summary, the pull-off test can be used to assess the bond between new materials and a concrete substrate, and for quality of surface preparation before overlay placement. Based on the nature of the test, both the failure location and failure load are used to assess the test results. Test results can be affected significantly by surface preparation methods and test specimen geometry.

3.6—Ultrasonic pulse velocity (ASTM C597)

The ultrasonic pulse velocity test, as prescribed in ASTM C597, determines the propagation velocity of a pulse of compressional (P-wave) energy through a concrete member (Jones 1949; Leslie and Cheesman 1949). The operational principle of modern testing equipment is illustrated in Fig. 3.6a. A pulser sends a short-duration, high-voltage signal to a transducer, causing the transducer to vibrate at its resonant frequency and generate a stress wave pulse in the concrete (refer to ACI 228.2R for a description of stress waves in solids). 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 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.

From the principles of elastic wave propagation, the P-wave 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 can 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 3.6b shows a typical graph of pulse velocity between compressive strength and pulse velocity. Note that

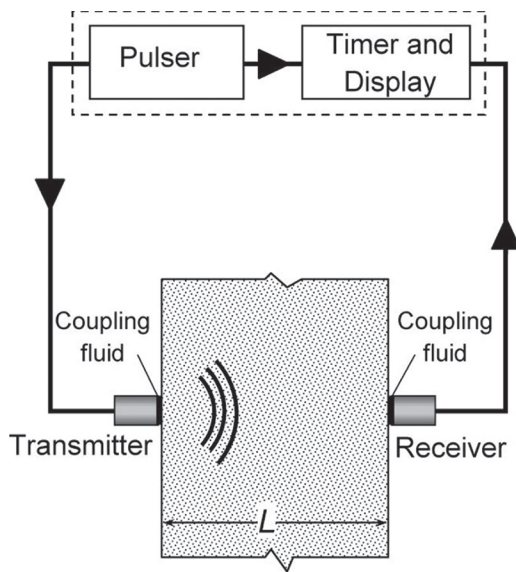


Fig. 3.6a—Schematic of apparatus to measure ultrasonic pulse velocity.

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 is accompanied by 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) can be associated with a velocity increase from approximately 2400 to 3040 m/s (7900 to 10,000 ft/s roughly). However, a strength increase from 25 to 30 MPa (3600 to 4400 psi roughly) could correspond to 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 could overshadow changes due to strength (Sturup et al. 1984). For example, although the pulse velocity depends strongly on the type and amount of aggregate in the concrete, the strength of normal-strength concrete (less than approximately 40 MPa [6000 psi]) is less sensitive to these factors. As the volumetric aggregate content of concrete increases, the pulse velocity may increase or decrease, depending on the aggregate type, but the compressive strength may not be affected appreciably (Bogas et al. 2013; Jones 1962).

Another factor is moisture content. As the moisture content of concrete increases from the air-dry to saturated condition, pulse velocity could increase up to 5 percent (Bungey et al. 2006). If the effects of moisture are not considered, erroneous conclusions can 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 steel reinforcement will also influence pulse velocities. Because the pulse velocity through steel is approximately 40 percent greater than

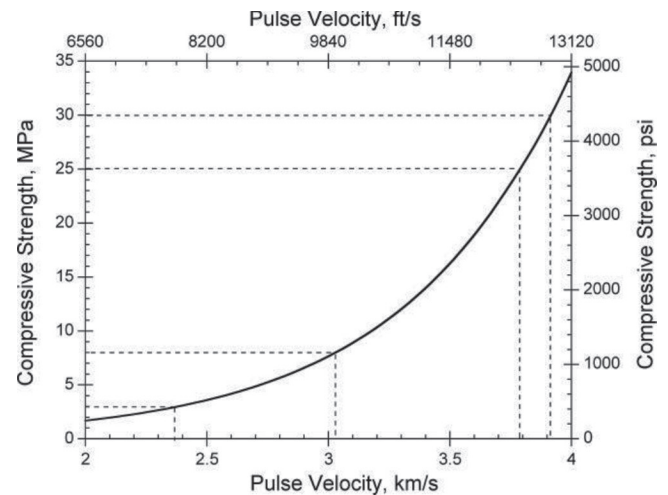


Fig. 3.6b—Schematic of typical relationship between pulse velocity and compressive strength of a given concrete mixture.

through concrete, the pulse velocity through a heavily reinforced concrete member could 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. Although correction factors have been proposed, such as those discussed in Malhotra (1976) and Bungey et al. (2006), their accuracy has not been established conclusively.

The measured pulse velocity can also be affected by the presence of cracks or voids along the propagation path from transmitter to receiver. The pulse could 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 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 a low single-operator coefficient of variation. This does not mean, however, that strength estimates are necessarily highly reliable.

The use of the velocity of other ultrasonic wave modes (for example, surface waves [Gudra and Stawiski 2000; Gallo and Popovics 2005]) and other wave characteristics (for example, P-wave wave energy attenuation [damping] [Teodoru 1988; Ismail et al. 1996; Tesfamariam et al. 2006]) to estimate in-place concrete strength have also been reported. The application of these methods to concrete structures has not, however, been evaluated extensively; furthermore, standard testing procedures for these methods have not yet been developed.

In summary, pulse velocity can be used to estimate strength in new and existing construction, provided a strength-rela-

tionship for the concrete mixture has been developed. For a given concrete, a change in P-wave 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.

3.7—Maturity method (ASTM C1074)

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 reaction rates are influenced by the concrete temperature; an increase in temperature causes an increase in the reaction rates. The extent of the reactions and, therefore, strength at any age depends on the thermal history of the concrete.

The maturity method (ASTM C1074) is a technique to estimate in-place strength by considering the relationship 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 specific 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 Commission 42-CEA (1981), and Carino (2004). As explained by Carino (2004), the maturity function is related to the rate of strength development after final setting has occurred. Therefore, a key feature of a maturity function is 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, which leads to the simple maturity function, commonly known as the Nurse-Saul function (Nurse 1949; Saul 1951), shown in Fig. 3.7a. 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

$$M(t) = \sum (T_a - T_0) \Delta t \quad (3.7a)$$

where $M(t)$ is temperature-time factor at age t , deg-days or deg-hours; Δt is a time interval, days or hours; T_a is average concrete temperature during time interval Δt , °C (°F); and T_0 is datum temperature, °C (°F).

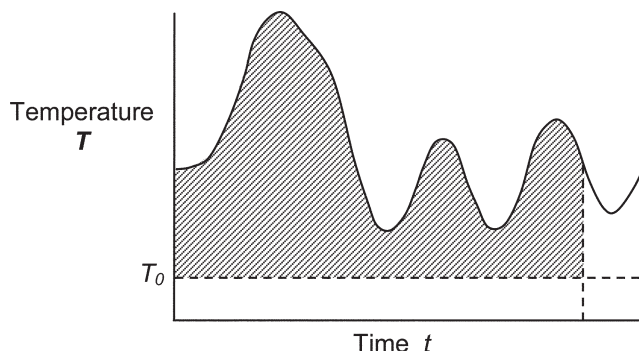


Fig. 3.7a—Maturity function based on assumption that the initial rate of strength gain varies linearly with temperature; shaded area is the temperature-time factor (Eq. (3.7a)).

Traditionally, the datum temperature used in Eq. (3.7a) has been taken as the temperature below which strength gain ceases, which has been assumed to be approximately -10°C (14°F). It has been suggested, however, that a single value for the datum temperature is not the most accurate approach and that for better accuracy, the datum temperature should be evaluated for the specific cementitious materials and admixtures in the concrete mixture (Carino 1984). ASTM C1074 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 C1074 provides a procedure to determine experimentally the datum temperature for other types of cementitious materials and for different ranges of curing temperature.

In the second approach, it is assumed that the initial rate of strength gain varies exponentially with concrete temperature in accordance with the Arrhenius function (Freiesleben-Hansen and Pedersen 1977). This second maturity function is used to compute an equivalent age of the concrete at a specified curing temperature as follows

$$t_e = \sum e^{-Q \left(\frac{1}{T_a} - \frac{1}{T_s} \right)} \Delta t \quad (3.7b)$$

where t_e is equivalent age at a specified temperature T_s , days or hours; Q is apparent activation energy divided by the gas constant, K (kelvin); T_a is average absolute temperature of concrete during time interval Δt , K ; T_s is specified curing temperature, K ; and Δt is time interval, days or hours.

In Eq. (3.7b), the exponential function can be considered as an age conversion factor that converts a time interval Δt at the actual concrete temperature to an equivalent interval (in terms of strength gain) at a specified curing temperature. In North America, the specified curing temperature is typically taken to be 23°C (296 K), whereas in Europe, 20°C (293 K) is typically used. To calculate the equivalent age of a concrete mixture, the value of a parameter (known as the apparent activation energy) is required; this depends primarily on the types of cementitious materials and, to a lesser extent, on the water-cementitious materials ratio (w/cm) (Carino and Tank 1992). The Q -value in Eq. (3.7b) is the activation energy divided by the gas constant ($8.314\text{ J/mole}\cdot\text{K}$). ASTM