ACI 214.4R-10

Guide for Obtaining Cores and Interpreting Compressive Strength Results

Reported by ACI Committee 214

American Concrete Institute®

First Printing
June 2010

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ISBN 978-0-87031-254-0

Guide for Obtaining Cores and Interpreting Compressive Strength Results

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Core testing is the most direct method to determine the compressive strength of concrete in a structure. Generally, cores may be obtained to assess whether concrete in a new structure complies with strength-based acceptance criteria or to evaluate the structural capacity of an existing structure based on the in-place concrete strength. In either case, the process of obtaining core specimens and interpreting strength test results is often confounded by various factors affecting in-place concrete strength or measured strength of test specimens. The scatter in strength test data, which is unavoidable given the inherent randomness of in-place concrete strengths and the uncertainty attributable to preparation and testing of the specimens, may further complicate compliance and evaluation decisions.

This guide summarizes practices for obtaining cores and interpreting core compressive strength test results. Factors that affect in-place concrete strength are reviewed so sampling locations that are consistent with objectives of the investigation can be selected. Strength correction factors are presented for converting the measured strength of non-standard core-test specimens to the strength of equivalent specimens with standard diameters, length-to-diameter ratios, and moisture conditioning. This guide provides direction for checking strength compliance of concrete in a structure under construction and methods for determining equivalent specified strength to assess capacity of an existing structure. The materials, processes, quality control measures, and inspections described in this document should be tested, monitored, or performed as applicable only by individuals holding the appropriate ACI Certifications or equivalent.

Keywords: compressive strength; core; hardened concrete; sampling; test.

CONTENTS

ACI 214.4R-10 supersedes ACI 214.4R-03 and was adopted and published June 2010.

CHAPTER 1—INTRODUCTION

1.1—Introduction

Core testing is the most direct method to determine the in-place compressive strength of concrete in a structure. Generally, cores are obtained to:

- Assess, if required, whether concrete in a new structure complies with strength-based acceptance criteria; or
- Determine in-place concrete strengths in an existing structure for evaluation of structural capacity.

In new construction, cylinder strength tests failing to meet strength-based acceptance criteria can be investigated using provisions given in ACI 318. These criteria specify the circumstances when core tests are permitted, the number of cores to be tested, the conditioning of the cores before testing, the limits on the time interval between coring and testing, and the basis for determining whether the concrete in the area represented by the core strengths is structurally adequate. This guide presents procedures for obtaining and testing cores and interpreting results in accordance with ACI 318.

If strength records are unavailable, the in-place strength of concrete in an existing structure can be evaluated using cores. This in-place strength determination is simplified when in-place strength data are converted into an equivalent specified compressive strength f_c' value that can be directly substituted into conventional strength equations with customary strength reduction factors. This guide presents procedures for performing this conversion in a manner consistent with the assumptions used to derive strength reduction factors for structural design.

1.2—Background

Analysis of core test data can be difficult and can subsequently lead to uncertain interpretations and conclusions. Based on 10 hypothetical core test results, 23 practitioners responding to a survey in 2000 estimated the compressive strength of in-place concrete between 3000 and 5000 psi (21 and 35 MPa) (Hanson 2007). Strength interpretations should always be made by, or with the assistance of, an investigator experienced in concrete technology. Factors contributing to the scatter of core strength test results include:

- Systematic variation of in-place strength along a member or throughout the structure;
- Random variation of concrete strength, both within one batch and among batches;
- Low test results attributable to flawed test specimens or improper test procedures;
- Effects of the size, aspect ratio, and moisture condition of the test specimen on the measured strengths; and
- Additional uncertainty attributable to testing that is present even for tests performed in strict accordance with standardized testing procedures.

1.3—Scope

This guide summarizes current practices for obtaining cores and interpreting core compressive strength test results in light of past and current research findings. Many of these findings are based on older references as the research has reached a mature state. Parallel procedures are presented for cases where cores are obtained to assess whether concrete strength in a new structure complies with strength-based acceptance criteria, and to determine a value based on the actual in-place concrete strength equivalent to the specified compressive strength f_c'. The latter can be directly substituted into conventional strength equations with customary strength reduction factors for strength evaluation of an existing structure. It is inappropriate to use procedures for determining the equivalent specified concrete strength to assess whether concrete strength in a new structure complies with strength-based acceptance criteria.

The order of contents parallels the logical sequence of activities in a typical core-test investigation. Chapter 3 describes how bleeding, consolidation, curing, and micro-cracking affect in-place concrete strength in structures so the investigator can account for this strength variation when planning the testing program. Chapter 4 identifies preferred sample locations and provides guidance on the number of specimens that should be obtained. Chapter 5 summarizes coring techniques that should result in undamaged, representative test specimens. Chapter 6 describes procedures for testing cores and detecting "outliers" by inspection of load-machine displacement curves or using statistical tests from ASTM E178. Chapter 7 summarizes the subsequent analysis of strength test data including use of ASTM C42/42M precision statements that quantify expected variability of properly conducted tests for a sample of homogeneous material, research findings concerning accuracy of empirically derived core strength correction factors, and statistical analysis techniques that can determine if the data can be grouped into unique categories. Chapter 8 briefly elaborates on criteria presented in ACI 318 for using core test results to investigate low-strength cylinder test results in new construction.

Chapter 9 presents two methods for estimating the lower tenth-percentile value of in-place concrete strength using core test data to quantify in-place strength. This value is equivalent to the specified compressive strength f_c' and can be directly substituted into conventional strength equations with customary strength reduction factors for strength evaluation of an existing structure.

Example calculations are presented in an appendix for:

- Outlier identification in accordance with ASTM E178 criteria;
- Determining whether a difference in mean strengths of cores from beams and columns is statistically significant; and
- Computing the equivalent specified strength using the two approaches presented in Chapter 9.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

C = a constant related to the number of batches, number of members, and type of construction, Alternate Method

d = diameter of core, in. (mm)

e = predetermined maximum error expressed as a percentage of the population average

F_d = correction factor for core damage

F_{dia} = correction factor for core diameter

$F_{\ell/d}$ = correction factor for length-to-diameter ratio of core

F_{mc} = correction factor for moisture content of core

$f_{0.10}$ = compressive strength of concrete at 10% fractile, psi (MPa)

f_c = equivalent in-place compressive strength of concrete, psi (MPa)

$\bar{f}_c$ = sample mean of equivalent in-place compressive strength of concrete, psi (MPa)

f_c' = specified compressive strength of concrete, psi (MPa)

$(\bar{f}_c)_{CL}$ = lower bound estimate of the sample mean equivalent in-place compressive strength of concrete at confidence limit CL, Alternate Method, psi (MPa)

$f'_{c,eq}$ = equivalent design compressive strength of concrete; applies to both methods, psi (MPa)

f_{ci} = equivalent in-place compressive strength of individual core specimen, psi (MPa)

f_{core} = compressive strength of concrete determined by core test, psi (MPa)

K = correction factor for number of cores, Tolerance Factor Method

ℓ = length of core, in. (mm)

n = number of samples

s_a = standard deviation of strength correction factors, psi (MPa)

s_c = sample standard deviation of equivalent in-place compressive strength of concrete, psi (MPa)

s_o = overall standard deviation, psi (MPa)

T = statistic related to the probability of an occurrence, Student's t test method

V = coefficient of variation, ratio of standard deviation to average, percent

V_d = coefficient of variation associated with F_d, percent

V_{dia} = coefficient of variation associated with F_{dia}, percent

$V_{\ell/d}$ = coefficient of variation associated with $F_{\ell/d}$, percent

V_{mc} = coefficient of variation associated with F_{mc}, percent

V_{WS} = coefficient of variation of in-place strengths

Z = correction factor to adjust for the uncertainty of strength correction factors; applies to both methods

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, "ACI Concrete Terminology," http://terminology.concrete.org. Definitions provided herein complement that resource.

core—a cylindrical sample of hardened concrete obtained by means of a core drill.

core test—compression test on a concrete sample cut from hardened concrete by means of a core drill.

strength, concrete compressive—the measured maximum resistance of a concrete specimen to axial compressive loading; expressed as force per unit cross-sectional area.

strength, in-place concrete compressive—the estimated in-place compressive strength, f_c, computed by adjusting the core test result for length-to-diameter ratio, diameter, moisture condition, and drilling direction.

strength, specified concrete compressive—the compressive strength of concrete used in design, f_c'.

strength, specified concrete equivalent—in-place concrete compressive strength, $f'_{c,eq}$, adjusted by correction factors that can be directly substituted into conventional strength equations with customary strength reduction factors.

CHAPTER 3—VARIATION OF IN-PLACE CONCRETE STRENGTH IN STRUCTURES

Chapter 3 discusses the variation of in-place concrete strength in structures so that the investigator can anticipate relevant factors in early stages of planning the testing program. Selecting core extraction locations and analyzing and interpreting data obtained are simplified and streamlined when pertinent factors are identified beforehand.

The quality of "as-delivered" concrete depends on the quality and relative proportions of the constituent materials and on the care and control exercised during batching, mixing, and handling. The final in-place quality depends on placing, consolidation, and curing practices. Recognizing that delivery of high-quality concrete does not ensure high-quality in-place concrete, some project specifications require minimum core compressive strength results for concrete acceptance (Ontario Ministry of Transportation 1998). Core test results may not represent the quality of concrete as delivered to the site if mixing water was added at the site, or poor placing, consolidation, or curing practices were followed.

Generally, the in-place concrete strength at the top of a member as cast is less than the strength at the bottom (Bloem 1965; Bungey 1989; Dilly and Vogt 1993).

3.1—Bleeding

Shallow voids under coarse aggregate caused by bleeding can reduce the compressive strength transverse to the direction of casting and consolidation (Johnson 1973), which is typically vertical, but would be horizontal for a precast concrete column or tilt-up panel cast in a horizontal orientation. The strength of cores with axes parallel to the casting direction can therefore be greater than that of cores with axes perpendicular to the casting direction. Experimental findings are contradictory because some investigators observed appreciable differences (Sanga and Dhir 1976; Takahata et al. 1991) whereas others did not (Bloem 1965). Although bleeding varies greatly with mixture proportions and constituent materials, available core strength data do not demonstrate a relationship between bleeding and the top-to-bottom concrete strength differences.

3.2—Consolidation

Concrete is usually consolidated by vibration to expel entrapped air after placement, unless it is a self-consolidating concrete (SCC). Strength is reduced by approximately 7% for each percent by volume of entrapped air remaining when concrete is insufficiently consolidated (Popovics 1969; Concrete Society 1987; ACI 309.1R). The investigator can assess the extent of poor consolidation in the concrete in question by simply inspecting the cores or using nondestructive techniques reported in ACI 228.2R.

Consolidation of fresh concrete in the lower portion of a column or wall is enhanced by static pressure of fresh concrete in the upper portion. These consolidation pressures can cause a strength increase (Ramakrishnan and Li 1970; Toossi and Houde 1981), so the lower portions of cast vertical members can have relatively greater strengths.

3.3—Curing

Proper curing procedures, which control the temperature and moisture environment of the concrete, are essential for high-quality concrete. Low initial curing temperatures reduce initial strength development rate but can result in higher long-term strength. Conversely, high initial-curing temperatures increase initial strength development but reduce long-term strength (ACI 308R).

High initial temperatures generated by hydration can significantly reduce strength of the interior regions of massive elements (ACI 305R). For example, results in Fig. 3.1 indicate the strength of cores obtained from the middle of mock 30 x 30 in. (760 x 760 mm) columns is consistently less than the strength of cores obtained from exterior faces (Cook 1989). Mock columns were cast using high-strength concrete with an average 28-day standard cylinder strength exceeding 11,200 psi (77 MPa). Similarly, data analysis from large specimens reported by Yuan et al. (1991), Mak et al. (1990, 1993), Burg and Ost (1992), and Miao et al. (1993) indicate a strength loss of approximately 3% of the average strength in the specimen for every 10°F (5°C) increase of average maximum temperature sustained during early hydration (Bartlett and MacGregor 1996a). Maximum temperatures recorded in these specimens varied between 110 and 200°F (45 and 95°C).

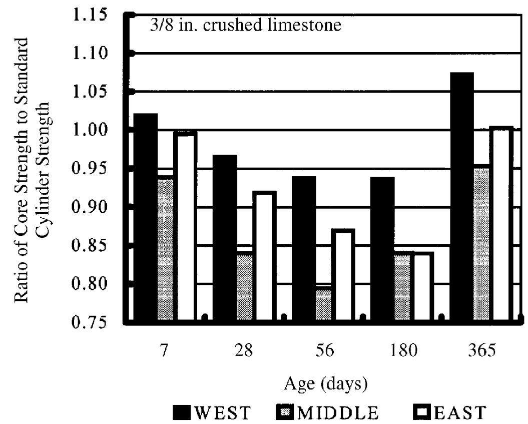

Fig. 3.1—Relationships between compressive strengths of column core samples and standard-cured specimens cast with high-strength concrete (Cook 1989).

The in-place strength of slabs or beams is more sensitive to the presence of moisture than the in-place strength of walls or columns because the unformed top surface is a relatively large fraction of the total surface area. Data from four studies (Bloem 1965, 1968; Meynick and Samarin 1979; Szypula and Grossman 1990) indicate the strength of cores from poorly cured shallow elements averages 77% of the strength of cores from properly cured elements for concrete ages of 28, 56, 91, and 365 days (Bartlett and MacGregor 1996b). Data from two studies investigating walls and columns (Bloem 1965; Gaynor 1970) indicate that strength loss at 91 days, attributable to poor curing, averages approximately 10% (Bartlett and MacGregor 1996b).

3.4—Microcracking

Microcracks in a core reduce strength (Szypula and Grossman 1990), and their presence explains why average strengths of cores from two ends of a beam cast from a single batch of concrete with a cylinder strength of 7850 psi (54.1 MPa) differed by 11% of their average (Bartlett and MacGregor 1994a). Microcracks can be present in regions of the structure that were subjected to stress from applied loads or restraint of imposed deformations. Rough handling of the core specimen can also cause microcracking. Strength

Table 3.1—Coefficient of variation due to in-place strength variation within structure V_{WS}

Structure composition		One member	Many members
One batch of concrete		7%	8%
Many batches of concrete	Cast-in-place	12%	13%
	Precast	9%	10%

results of cores taken where microcracking has occurred or where cracks are visible should be interpreted with caution.

In massive concrete elements, hydration causes thermal gradients between the interior, which becomes hot, and the element surfaces, which remain relatively cool. In this case, surfaces are restrained from contracting by the interior of the element, which can cause microcracking that reduces strength at the surface. This phenomenon has been clearly observed in some investigations (Mak et al. 1990) but not in others (Cook et al. 1992).

3.5—Overall variability of in-place strengths

Estimates of overall variability of in-place concrete strengths reported by Bartlett and MacGregor (1995) are presented in Table 3.1. Variability is expressed as the coefficient of variation V_{WS}, which is the ratio of standard deviation of in-place strength to average in-place strength. Overall variability depends on the number of members in the structure, number of concrete batches present, and whether construction is precast or cast-in-place. Values shown are for concrete produced, placed, and protected in accordance with normal industry practice, for example, ASTM C94/C94M, ACI 304R, and ACI 308R, and may not pertain to concrete produced to high or low standards of quality control.

CHAPTER 4—PLANNING THE TESTING PROGRAM

The procedure for planning a core-testing program depends on the objective of the investigation. Section 4.1 presents procedures for checking whether concrete in a new structure complies with strength-based acceptance criteria, whereas Section 4.2 presents procedures for evaluating the strength capacity of an existing structure using in-place strengths.

As noted in Chapter 3, the strength of concrete in a placement usually increases with depth. In single-story columns, cores should be obtained from the central portion, where strength is relatively constant, and not in the top 18 to 24 in. (450 to 600 mm), where it can decrease by 15%, or in the bottom 12 in. (300 mm), where it can increase by 10% (Bloem 1965).

4.1—Investigating concrete in a new structure using strength-based acceptance criteria

To investigate low-strength test results in accordance with ACI 318, three cores are required from that part of the structure cast from concrete represented by the low-strength test result. The investigator should only sample areas where suspect concrete was placed.

In some situations, such as thin or heavily reinforced sections, it is difficult or impracticable to obtain cores meeting all the length and diameter requirements of ASTM C42/C42M. Nevertheless, cores can allow a relative comparison of two or more portions of a structure representing different concrete batches. For example, consider two sets of columns placed with the concrete with the same mixture proportions: one acceptable based on standard strength tests and one questionable because of low strength test results. Nondestructive testing methods (ACI 228.1R) may indicate that the concrete quality in suspect columns exceeds that in acceptable columns. Alternatively, it is appropriate to take 2 in. (50 mm) diameter cores from columns where 1 in. (25 mm) maximum size aggregate was used. After trimming the cores, however, ℓ/d will be less than 1.0 where the cover is only 2 in. (50 mm) and reinforcement bars cannot be cut. Acknowledging that strength tests of short cores may not produce results that accurately reflect column concrete strength, a relative comparison of the two concrete placements can sufficiently determine if concrete strength in question is comparable to the other placement or if more investigation is warranted.

4.2—Evaluating existing structure capacity using in-place strengths

To establish in-place strength values for existing structures, the sample size and locations from which cores are extracted need to be selected using procedures such as those described in ASTM E122 and ASTM C823/C823M.

As the sample size increases, the accuracy of the result improves. The likelihood of detecting a spurious value in the data set also improves but greater costs are incurred and the risk of weakening the structure increases. ASTM E122 recommends computing sample sizes using Eq. (4-1) to achieve a 1-in-20 chance that the difference between the measured sample average and population average, expressed as a percentage of population average, will be less than some predetermined error.

$$n = \left(\frac{2V}{e}\right)^2 \tag{4-1}$$

where

n = recommended sample size;

e = predetermined maximum error expressed as a percentage of the population average; and

V = estimated coefficient of variation of the population, in percent, and may be estimated from values shown in Table 3.1 or from other available information.

For example, if the estimated coefficient of variation of in-place strength is 15%, and it is desired that the measured average strength should be within 10% of the true average strength approximately 19 times out of 20, Eq. (4-1) indicates that (for V = 15% and e = 10%) a total of nine cores should be obtained. If a higher confidence level is desired, or if smaller percentage error is necessary, then a larger sample size is required. Statistical tests for determining whether extreme values should be rejected, such as those in ASTM E178, become more effective as sample size increases. As indicated by relationships between the percentage error and the recommended number of specimens shown in Fig. 4.1, the benefits of larger sample sizes diminish. ASTM C823/C823M recom-

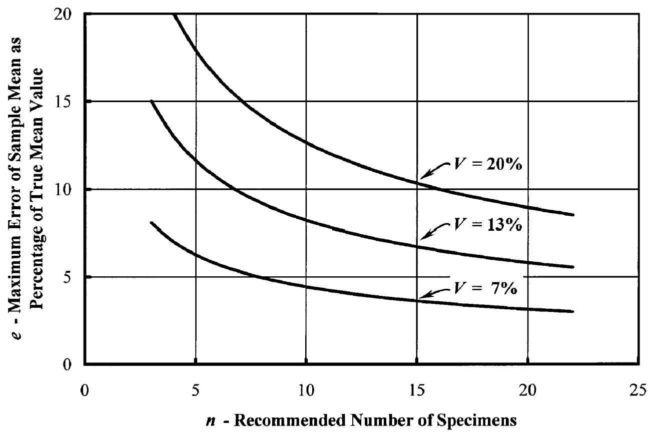

Fig. 4.1—Maximum error of sample mean for various recommended number of specimens.

mends a minimum of five core test specimens be obtained for each concrete category with a unique condition or specified quality, specified mixture proportion, or specified material property. ASTM C823/C823M also provides guidance for repeating the sampling sequence for large structures.

The investigator should select core extraction locations based on the overall objective of the investigation by characterizing the overall in-place strength for general evaluation purposes or determining the in-place strength of a specific group of components, not the ease of obtaining samples. To characterize the overall in-place strength of an existing structure for general evaluation purposes, cores should be drilled from randomly selected locations throughout the structure using a written sampling plan. When the in-place strength for a specific component or group of components is sought, the investigator should extract cores at randomly selected locations from those specific components.

When determining sample locations, the investigator should consider whether different categories of strength or type of concrete may be present in the structure. For example, in-place strengths of walls and slabs cast from a single batch of concrete may differ (Meininger 1968) or concrete with different required strengths may have been used for the footings, columns, and floor slabs in a building. If the concrete volume under investigation contains two or more concrete categories, the investigator should objectively select sample locations so as not to bias the outcome. Alternatively, the investigator should randomly select a sufficient number of sampling locations for each concrete category with unique composition or properties. The investigator can use nondestructive testing methods (ACI 228.1R) to perform a preliminary survey to identify regions in a structure that have different concrete properties.

ACI 311.1R (SP-2) and ASTM C823/C823M contain guidance concerning sampling techniques.

CHAPTER 5—OBTAINING TEST SPECIMENS

Coring techniques should result in undamaged, representative test specimens. The investigator should delay coring until the concrete being cored has sufficient strength to allow core removal without disturbing the bond between mortar and aggregate. ASTM C42/C42M suggests that concrete should not be cored before it is 14 days old, unless other information indicates the concrete can withstand the coring process without damage. ASTM C42/C42M further suggests in-place nondestructive tests (ACI 228.1R) can be performed to estimate the level of concrete strength development before coring is attempted.

Core specimens for compression tests should not contain reinforcing bars. Several nondestructive methods are available to locate reinforcement embedded in concrete, including magnetic induction using a pachometer or cover meter, X-ray or gamma-ray radiography, and short-pulse radar (ground-penetrating radar) (Malhotra and Carino 2004). Cutting sections containing conduit, ductwork, or prestressing tendons should be avoided.

As described in Chapter 7, the specimen strength is affected by the core diameter and the ratio of length-to-diameter, ℓ/d, of the specimen. Strength correction factors for these effects are derived empirically from test results (Bartlett and MacGregor 1994b) and so are not universally accurate. Therefore, it is preferable to obtain specimens with nominal diameters of 4 to 6 in. (100 to 150 mm) and ℓ/d ratios between 1.5 and 2 to minimize error introduced by the strength correction factors (Neville 2001).

Core drilling should be performed by an experienced operator using a diamond-impregnated bit attached to the core barrel. The drilling apparatus should be rigidly anchored to the member to avoid bit wobble, which results in a specimen with variable cross section and introduces large strains in the core. The drill bit should be lubricated with water and resurfaced or replaced when worn. The operator should be informed beforehand that the cores are for strength testing and require proper handling and storage.

Because a damaged specimen will not accurately represent in-place concrete strength, core specimens require protection from freezing and damage during transit.

Drilling a core with a water-cooled bit results in a moisture gradient between exterior and interior of the core that adversely affects the compressive strength (Fiorato et al. 2000; Bartlett and MacGregor 1994c). ASTM C42/C42M presents moisture protection and scheduling requirements intended to achieve moisture distribution in core specimens that better represent the moisture distribution in concrete before the concrete is wetted during drilling. The restriction concerning the commencement of core testing provides a minimum time for the moisture gradient to dissipate.

The investigator, or investigator's representative, should witness and document the core drilling. Samples should be numbered and their orientation in the structure indicated by permanent markings on the core itself. The investigator should record the extraction location of each core and any features that may affect strength, such as cracks or honeycombs. Similar features observed by careful inspection of surrounding concrete should also be documented. Given the likelihood of questionable low-strength values, any information that can later identify reasons for low values is useful.

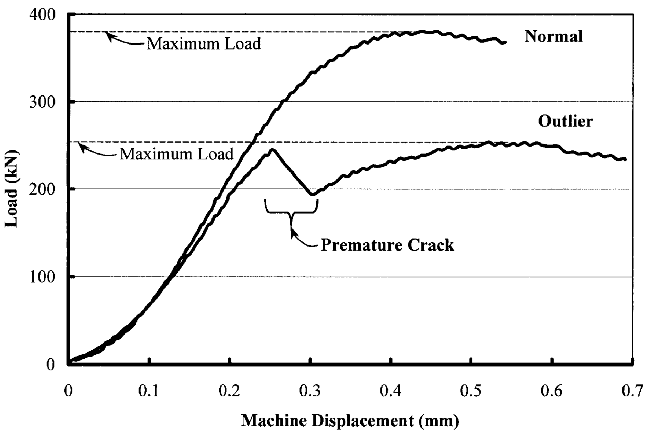

Fig. 6.1—Use of load-machine displacement curves to identify outlier due to flawed specimen (Bartlett and MacGregor 1994a).

CHAPTER 6—CORE TESTING

ASTM C42/C42M presents standard methods for specimen conditioning, preparing the ends before testing, and correcting the test result for core length-to-diameter ratio. Other standards for measuring the specimen length and performing the compression test are referenced, and information required in the test report is described. Care should be exercised during sawing, grinding, and capping to avoid core damage and to achieve the perpendicularity and planeness requirements of ASTM C39/C39M.

Core densities, which can indicate consolidation uniformity, are often useful to assess low core test results. Before capping, the core density can be computed by dividing its mass by its volume, calculated from its average diameter and length, or in accordance with ASTM C642.

When testing small-diameter cores, careful alignment of the specimen in the testing machine is necessary. If the diameter of the suspended spherically seated bearing block exceeds specimen diameter, the spherical seat may not rotate into proper alignment, causing nonuniform contact against the specimen. ASTM C39/C39M limits the upper bearing face diameter to avoid an excessively large upper spherical bearing block.

A load-machine displacement response graph can be a useful indicator of abnormal behavior resulting from testing a flawed specimen. For example, the two curves in Fig. 6.1 are for 4 x 4 in. (100 x 100 mm) cores, obtained from one beam, that were given identical moisture treatments. The lower curve is abnormal because the load drops markedly before reaching maximum value. This curve is consistent with a premature splitting failure and can be attributed to imperfect preparation of the specimen ends. Thus, the low result can be attributed to a credible physical cause and should be excluded from the data set.

Sullivan (1991) describes using nondestructive tests to check for core abnormalities before conducting compressive strength tests.

When the investigator cannot find a physical reason to explain an unusually low or high result, then statistical tests given in ASTM E178 can be used to determine whether the observation is an outlier. A minimum of six tests are needed to classify a test as an outlier (Bartlett and MacGregor 1995). An example calculation using ASTM E178 criteria to check whether a low value is an outlier is presented in the Appendix. If an outlier can be attributed to an error in preparing or testing the specimen, it should be excluded from the data set. If an observation is an outlier according to ASTM E178 criteria but the reason for the outlier cannot be determined, then the investigator should report suspect values and indicate whether they have been used in the subsequent analyses.

If a core test result is deemed an outlier and removed from the data set, the evaluator should decide whether it should be replaced with the test result for a new specimen. If evaluating whether suspect concrete in a new structure complies with strength-based acceptance criteria in accordance with ACI 318 and Chapter 8 of this guide, ACI 318 provides a procedure for addressing erratic core strengths. If determining an equivalent specified strength for evaluating the structural capacity of an existing structure, the provisions of Chapter 9 of this guide apply if three or more core test results are available. In this case it may not be necessary to obtain a new test result to replace the excluded value, but if the number of cores tested is small and the strength variability is high, the conservatism of the resulting equivalent specified strength may be mitigated by testing additional new specimens.

CHAPTER 7—ANALYZING STRENGTH TEST DATA

Analysis and interpretation of core strength data are complicated by the large scatter usually observed in test results. Chapter 7 describes the expected scatter of core tests from a sample of homogeneous material, discusses other reasons for strength variation that require consideration, and briefly reviews statistical techniques for identifying sources of variability in a specific data set. Detailed descriptions of these statistical techniques can be found in most statistical references, such as Larsen and Marx (2006).

7.1—ASTM C42/C42M precision statements

ASTM C42/C42M provides precision statements that quantify the inherent error associated with testing cores from a homogeneous material tested in accordance with standardized procedures. The single-operator coefficient of variation is 3.2% and the multilaboratory coefficient of variation is 4.7%. In the interlaboratory study used to derive these values, the measured values of single-operator coefficient of variation were 3.1 to 3.4% for cores from the three different slabs, and the measured values of multilaboratory coefficient of variation were 3.7 to 5.3% (Bollin 1993).

These precision statements are a useful basis for preliminary checks of core strength data if the associated assumptions and limitations are fully appreciated. Observed strength differences can exceed the limits stated in ASTM C42/C42M due to one or more of the following:

- Limits stated in ASTM C42/C42M are "difference 2 sigma" (d2s) limits so the probability that they are exceeded is 5%. Therefore, there is a 1-in-20 chance that the strength of single cores from the same material tested by one operator will differ by more than 9% of

Table 7.1—Probable range of core strengths due to single-operator error

Number of cores	Expected range of core strength as percent of average core strength	Range with 5% chance of being exceeded as percent of average core strength
3	5.4	10.6
4	6.6	11.6
5	7.2	12.4
6	8.1	12.9
7	8.6	13.3
8	9.1	13.7
9	9.5	14.1
10	9.8	14.3

their average, and a 1-in-20 chance that the average strength of cores from the same material tested by different laboratories will differ by more than 13% of their average;

- Variability of the in-place concrete properties can exceed that in the slabs investigated for the multilaboratory study reported by Bollin (1993); and
- Testing accuracy can be less rigorous than that achieved by laboratories that participated in the study reported by Bollin (1993).

The single-operator coefficient of variation is a measure of the repeatability of the core test when performed in accordance with ASTM C42/C42M. A practical use of this measure is to check whether the difference between strength test results of two individual cores obtained from the same material sample does not differ by more than 9% of their average. The difference between consecutive tests (or any two randomly selected tests) is usually less than the overall range between largest and least values, which tends to increase as the sample size increases. The expected range and the range that has a 1-in-20 chance of being exceeded, expressed as a fraction of the average value, can be determined for different sample sizes using results originally obtained by Pearson (1941-42). Table 7.1 shows values corresponding to the ASTM C42/C42M single-operator coefficient of variation of 3.2%, which indicate, for example, in a set of five cores from the same material sample, the expected range is 7.2% of the average value and there is a 1-in-20 chance the range will exceed 12.4% of the average value. Table 1 of ASTM C670 provides multipliers that, when applied to the single-operator coefficient of variation, also estimate the range that has a 1-in-20 chance of being exceeded.

The multilaboratory coefficient of variation is a measure of the core test reproducibility as performed in accordance with ASTM C42/C42M. Although the reported values are derived for tests defined as the average strength of two specimens, they can be assumed to be identical to those from tests defined as the average strength of three specimens. Thus, this measure indicates that if two independent laboratories test cores from the same material sample in accordance with the criteria given in ACI 318, and each laboratory tests three specimens in conformance with ASTM C42/C42M, there remains a 1-in-20 chance that the reported average strengths will differ by more than 13% of their average.

Table 7.2—Strength correction factors for length-to-diameter ratio

ℓ/d	ASTM C42/C42M	BS EN 12504-1:2009
2.00	1.00	1.00
1.75	0.98	0.97
1.50	0.96	0.92*
1.25	0.93	—
1.00	0.87	—

*ℓ/d = 1.5 is just outside range of validity of equation given in BS EN 12504-1:2009.

7.2—Review of core strength correction factors

The measured core strength depends partly on the specimen's length-to-diameter ratio, diameter, moisture condition at the time of testing, presence of reinforcement or other inclusions, and coring direction. Considerable research has been conducted concerning these factors, and strength correction factors have been proposed to account for their effects. Research findings, however, are contradictory. Published strength correction factors are not exact and may not be universally applicable because they were derived empirically from specific data sets. To indicate the degree of uncertainty associated with these factors, this section summarizes relevant research findings. Chapter 9 presents specific strength correction factor values.

7.2.1 *Length-to-diameter ratio*—The length-to-diameter ratio ℓ/d was identified in the 1927 edition of ASTM C42/C42M as a factor that influences the measured compressive strength of a core. Minor variations of the original ℓ/d strength correction factors were made in subsequent editions. Specimens with small ℓ/d fail at greater loads because the steel loading platens of the testing machine restrain lateral expansion throughout the specimen length more effectively and therefore provide confinement (Newman and Lachance 1964; Ottosen 1984). The end effect is largely eliminated in standard concrete compression test specimens that have ℓ/d of 2.

Table 7.2 shows values of strength correction factors recommended in ASTM C42/C42M and British Standard BS EN 12504-1:2009 (British Standards Institution 2009) for cores with ℓ/d between 1 and 2. ASTM C42/C42M does not permit testing cores with ℓ/d less than 1, and the equation given in the BS EN Standard does not apply to cores with ℓ/d less than 1.6 for in-place cylinder strength determination. The recommended values diverge as ℓ/d reduces. The ASTM factors are average values that pertain to dry or soaked specimens with strengths between 2000 and 6000 psi (14 and 40 MPa). ASTM C42/C42M states that actual ℓ/d correction factors depend on specimen strength and elastic modulus.

Bartlett and MacGregor (1994b) report that the necessary strength correction is slightly less for high-strength concrete and soaked cores, but recommend strength correction factor values similar to those in ASTM C42/C42M. They also observed that strength correction factors are less accurate as the magnitude of the necessary correction increases for cores with smaller ℓ/d. Thus, corrected core strength values do not have the same degree of certainty as strength obtained from specimens having ℓ/d of 2.

7.2.2 *Diameter*—There is conflicting experimental evidence concerning the strength of cores with different diameters. Whereas there is a consensus that differences between 4 and 6 in. (100 and 150 mm) diameter specimens are negligible (Concrete Society 1987), there is less agreement concerning 2 in. (50 mm) diameter specimens. In one study involving cores from 12 different concrete mixtures, the ratio of average strength of five 2 in. (50 mm) diameter cores to the average strength of three 4 in. (100 mm) diameter cores ranged from 0.63 to 1.53 (Yip and Tam 1988). Analysis of strength data from 1080 cores tested by various investigators indicated that strength of a 2 in. (50 mm) diameter core was, on average, 6% less than the strength of a 4 in. (100 mm) diameter core (Bartlett and MacGregor 1994d).

The scatter in strengths of 2 in. (50 mm) diameter cores often exceeds that observed for 4 or 6 in. (100 or 150 mm) diameter cores. The in-place strength variability within the element being cored inflates the strength variability of small-volume specimens. Cores drilled vertically through the thickness of a slab can be particularly susceptible to this effect (Lewis 1976).

It is often difficult to obtain a 2 in. (50 mm) diameter specimen that is not affected by the drilling process or does not contain a small defect that will affect the result. If correction factors are required to convert the strength of 2 in. (50 mm) diameter cores to the strength of equivalent 4 or 6 in. (100 or 150 mm) diameter cores, the investigator should derive them directly using cores of each diameter obtained from the structure in question.

7.2.3 *Moisture condition*—Different moisture-conditioning treatments have a considerable effect on measured strengths. Air-dried cores are, on average, 10 to 14% (Neville 1981; Bartlett and MacGregor 1994a) stronger than soaked cores, although the actual ratio for cores from a specific concrete can differ considerably from these values. Soaking causes concrete at the surface of the specimen to swell, and restraint of this swelling by the interior region causes self-equilibrated stresses that reduce the measured compressive strength (Popovics 1986). Conversely, drying the surface causes shrinkage that, when restrained, creates a favorable residual stress distribution that increases the measured strengths. In both cases, changes in moisture condition are initially very rapid (Bartlett and MacGregor [1994c], based on data reported by Bloem [1965]). If cores are not given standardized moisture conditioning before testing, or if the period between the end of moisture treatment and testing varies significantly, then additional variability of measured strengths can be introduced.

The percentage of strength loss caused by soaking the core depends on several factors. Less-permeable concrete exhibits a smaller strength loss. Bartlett and MacGregor (1994a) observed a more severe strength loss in 2 in. (50 mm) diameter cores compared with 4 in. (100 mm) diameter cores from the same element. Extending the soaking period beyond 40 hours can cause further core strength reduction. The difference between strengths of soaked and air-dried cores may be smaller for structural lightweight-aggregate concrete (Bloem 1965).

7.2.4 *Presence of reinforcement bars or other inclusions*—The investigator should avoid specimens containing embedded reinforcement because it may influence the measured compressive strength. Previous editions of ASTM C42/C42M have recommended trimming the core to eliminate reinforcement, provided an ℓ/d of at least 1.0 can be maintained.

7.2.5 *Coring location*—Cores should be drilled perpendicular to the surface according to ASTM C42/C42M. For slabs, the core is often drilled vertically or in the direction of placement and compaction. The core, in this case, can be stronger than a core drilled horizontally, due to bleed water collecting beneath the coarse aggregate as described in Chapter 3. For columns, walls, and beams, the core is often drilled horizontally or perpendicular to the direction of placement and compaction. The influence of coring location, in this case, can be more pronounced near the upper surface of members where bleed water is concentrated. To determine whether the in-place strength is affected by drilling location, the investigator should assess this using specimens drilled in different locations from the structure in question, if practicable.

7.3—Statistical analysis techniques

Statistical analysis techniques can determine if data are random or can be grouped into unique sets. For example, statistical tests can verify that strengths in the uppermost parts of columns are significantly less than strengths elsewhere, and so focus the investigation accordingly.

Statistical tests are useful for analyzing preliminary hypotheses developed during an initial review of data that are logically consistent with the circumstances of the investigation and are credible in light of past experience. Although it is possible to use statistical techniques to look for correlations and trends in data in an exploratory manner, it is rarely efficient to do so. Flawed conclusions are undetectable if statistical analyses are conducted without an understanding of essential physical and behavioral characteristics represented in the data. Instead, it is preferable to identify factors that affect strength in a particular instance and then use statistical analyses to verify whether these factors are significant.

Perhaps the most useful analysis method is the Student's t test, which is used to decide whether the difference between two average values is sufficiently large to imply that the true mean values of underlying populations, from which samples are drawn, are different. ASTM C823/C823M recommends using the Student's t test to investigate whether the average strength of cores from concrete of questionable quality differs from the average strength of cores from concrete of high quality. Details of the Student's t test can be found in most statistical references (for example, Larsen and Marx [2006]), and a numerical example illustrating its use is presented in the Appendix.

Two types of errors are associated with any statistical test. A Type I error occurs when a hypothesis such as "the true mean values of two groups are equal" is rejected when, in fact, it is true. A Type II error occurs when a hypothesis is accepted when, in fact, it is false. In quality control, these are referred to as the producer's and the consumer's risk, respectively, because the producer's concern is that a satisfactory product will be rejected, and the consumer's concern is that an unsatisfactory product will be accepted. It is not possible to reduce the likelihood of a Type I error without increasing the likelihood of a Type II error, or vice versa, unless the sample size is increased. Decisions made on the basis of a small number of tests increase the likelihood of large error. The investigator should recognize that most statistical tests, including the Student's *t* test, are designed to limit the likelihood of a Type I error. If an observed difference obtained from a small sample seems large but is not statistically significant, then a true difference may exist and can be substantiated if additional cores are obtained to increase the sample size.

CHAPTER 8—INVESTIGATION OF LOW-STRENGTH TEST RESULTS IN NEW CONSTRUCTION USING ACI 318

In new construction, low cylinder strength tests are investigated in accordance with provisions of ACI 318. Suspect concrete is considered structurally adequate if the average strength of three cores, corrected for ℓ/d in accordance with ASTM C42/C42M, exceeds $0.85f_c'$, and no individual strength is less than $0.75f_c'$. ACI 318 recognizes that core strengths are potentially lower than strengths of cast specimens representing the quality of concrete delivered to the project. This relationship is corroborated by observations that strengths of 56-day-old soaked cores averaged 93% of the strength of standard cured 28-day cylinders and 86% of the strength of standard-cured 56-day cylinders (Bollin 1993).

ACI 318 permits additional testing of cores extracted from locations represented by erratic strength results. ACI 318 does not define "erratic," but this might reasonably be interpreted as a result that clearly differs from the rest and can be substantiated by a valid physical reason that has no bearing on the structural adequacy of the concrete in question.

For structural adequacy, the ACI 318 strength requirements for cores need only be met at the age when the structure will be subject to design loads.

CHAPTER 9—DETERMINING AN EQUIVALENT f_c' VALUE FOR EVALUATING STRUCTURAL CAPACITY OF AN EXISTING STRUCTURE

Chapter 9 presents procedures to determine an equivalent design strength for structural evaluation for direct substitution into conventional strength equations that include customary strength reduction factors. This equivalent design strength is the lower tenth percentile of the in-place strength and is consistent with the statistical description of the specified compressive strength of concrete, f_c'. Chapter 9 presents two methods for estimating the lower tenth-percentile value from core test data.

Procedures described in this chapter are only appropriate where the determination of an equivalent f_c' is necessary for strength evaluation of an existing structure, and should not be used to investigate low cylinder strength test results.

9.1—Conversion of core strengths to equivalent in-place strengths

The in-place strength of the concrete at the location from which a core test specimen was extracted can be computed using the equation

$$f_c = F_{\ell/d}F_{dia}F_{mc}F_df_{core} \quad (9\text{-}1)$$

Table 9.1—Magnitude and accuracy of strength correction factors for converting core strengths into equivalent in-place strengths*

	Factor	Mean value	Coefficient of variation V, %
$F_{\ell/d}$: ℓ/d ratio†	Standard treatment‡:	$1 - \{0.130 - \alpha f_{core}\}\left(2 - \frac{\ell}{d}\right)^2$	$2.5\left(2 - \frac{\ell}{d}\right)^2$
	Soaked 48 hours in water:	$1 - \{0.117 - \alpha f_{core}\}\left(2 - \frac{\ell}{d}\right)^2$	$2.5\left(2 - \frac{\ell}{d}\right)^2$
	Dried§:	$1 - \{0.144 - \alpha f_{core}\}\left(2 - \frac{\ell}{d}\right)^2$	$2.5\left(2 - \frac{\ell}{d}\right)^2$
F_{dia}: core diameter	2 in. (50 mm)	1.06	11.8
	4 in. (100 mm)	1.00	0.0
	6 in. (150 mm)	0.98	1.8
F_{mc}: core moisture content	Standard treatment‡:	1.00	2.5
	Soaked 48 hours in water:	1.09	2.5
	Dried§:	0.96	2.5
F_d: damage due to drilling		1.06	2.5

*To obtain equivalent in-place concrete strength, multiply the measured core strength by appropriate factor(s) in accordance with Eq. (9-1).
†Constant α equals $3(10^{-6})$ 1/psi for f_{core} in psi, or $4.3(10^{-4})$ 1/MPa for f_{core} in MPa.
‡Standard treatment specified in ASTM C42/C42M.
§Dried in air at 60 to 70°F (16 to 21°C) and relative humidity less than 60% for 7 days.

where f_c is the equivalent in-place strength; f_{core} is the core strength; and strength correction factors $F_{\ell/d}$, F_{dia}, and F_{mc} account for effects of ℓ/d ratio, diameter, and moisture condition of the core, respectively. Factor F_d accounts for the effect of damage sustained during drilling, including microcracking and undulations at the drilled surface and cutting through coarse-aggregate particles that may subsequently pop out during testing (Bartlett and MacGregor 1994d). Table 9.1 shows mean values of strength correction factors reported by Bartlett and MacGregor (1995) based on data for normal-weight concrete with strengths between 2000 and 13,400 psi (14 and 92 MPa). The right-hand column shows coefficients of variation V that indicate uncertainty of the mean value. It follows that a 4 in. (100 mm) diameter core with $\ell/d = 2$ that has been soaked 48 hours before testing has $f_c = 1.0 \times 1.0 \times 1.09 \times 1.06f_{core} = 1.16f_{core}$.

9.2—Uncertainty of estimated in-place strengths

After core strengths have been converted to equivalent in-place strengths, sample statistics can be calculated. The sample mean in-place strength $\bar{f_c}$ is obtained from the following equation

$$\bar{f_c} = \frac{1}{n}\sum_{i=1}^{n} f_{ci} \quad (9\text{-}2)$$

where n is the number of cores, and f_{ci} is the equivalent in-place strength of an individual core specimen, calculated using Eq. (9-1). The sample standard deviation of the in-place strength, s_c, is obtained from the following equation

$$s_c = \sqrt{\sum_{i=1}^{n} \frac{(f_{ci} - \bar{f}_c)^2}{(n-1)}} \quad (9\text{-}3)$$

The sample mean and sample standard deviation are estimates of the true mean and true standard deviation, respectively, of the entire population. The accuracy of these estimates, which improves as the sample size increases, can be investigated using the classical statistical approach to parameter estimation (for example, Larsen and Marx [2006]).

The accuracy of the estimated in-place strengths also depends on the accuracy of the various strength correction factors used in Eq. (9-1). The standard deviation of in-place strength due to the empirical nature of strength correction factors, s_a, can be obtained from the following equation

$$s_a = \bar{f}_c \sqrt{V_{\ell/d}^2 + V_{dia}^2 + V_{mc}^2 + V_d^2} \quad (9\text{-}4)$$

The right column of Table 9.1 shows values of $V_{\ell/d}$, V_{dia}, V_{mc}, and V_d, the coefficients of variation associated with strength correction factors $F_{\ell/d}$, F_{dia}, F_{mc}, and F_d, respectively. The coefficient of variation for a particular strength correction factor need only be included in Eq. (9-4) if the corresponding factor used in Eq. (9-1) to obtain the in-place strength differs from 1.0. If test specimens have different ℓ/d, it is appropriate and slightly conservative to use the $V_{\ell/d}$ value for the core with the smallest ℓ/d. For cores from concrete produced with similar proportions of similar aggregates, cement, and admixtures, errors due to the strength correction factors remain constant irrespective of the number of specimens obtained.

The overall uncertainty of estimated in-place strengths is a combination of sampling uncertainty and uncertainty caused by strength correction factors. These two sources of uncertainty are statistically independent, and so the overall standard deviation s_o is determined using the following equation

$$s_o = \sqrt{s_c^2 + s_a^2} \quad (9\text{-}5)$$

9.3—Percentage of in-place strengths less than f_c'

Criteria in ACI 318 for proportioning concrete mixtures require the target strength exceeds f_c' to achieve an approximately 1-in-100 chance that the average of three consecutive tests will fall below f_c', and approximately a 1-in-100 chance that no individual test will fall more than 500 psi (3.5 MPa) below f_c' if the specified strength is less than 5000 psi (35 MPa), or below $0.90f_c'$ if the specified strength exceeds 5000 psi (35 MPa). These criteria imply that f_c' represents approximately the 10% fractile, or the lower tenth-percentile value, of strength obtained from a standard test of 28-day cylinders. In other words, one standard strength test in 10 will be less than f_c' if the target strength criteria required by ACI 318 are followed. Various methods for converting in-place strengths obtained by nondestructive testing into an equivalent f_c' are therefore based on estimating the 10% fractile of the in-place strength (Bickley 1982; Hindo and Bergstrom 1985; Stone et al. 1986).

This practice was corroborated by a study that showed f_c' represents approximately the 13% fractile of the 28-day in-place strength in walls and columns and approximately the 23% fractile of the 28-day in-place strength in beams and slabs (Bartlett and MacGregor 1996b). The value for columns is more appropriate for defining the equivalent specified strength because the column nominal strength is more sensitive to the concrete compressive strength than a beam or slab. Therefore, a procedure assuming the specified strength is equal to the 13% fractile of the in-place strength is appropriate, and one assuming f_c' is equivalent to the 10% fractile of the in-place strength is conservative.

9.4—Methods to estimate the equivalent specified strength

There is no universally accepted method for determining the 10% fractile of the in-place strength, which as described in Section 9.3, is approximately equivalent to f_c'. In general, the following considerations should be addressed:

- Factors that bias core test results, which can be accounted for using strength correction factors discussed in Chapter 7;
- Uncertainty of each strength correction factor used to estimate in-place strength;
- Errors of the measured average value and measured standard deviation attributable to sampling and therefore the errors decrease as the sample size increases;
- Variability attributable to acceptable deviations from standardized testing procedures that can cause the measured standard deviation of strength tests to exceed true in-place strength variation; and
- Desired confidence level, which represents the likelihood that the fractile value calculated using sample data, will be less than the true fractile value of the underlying population from which the sample is drawn.

This section presents two methods for estimating the 10% fractile of the in-place strength. To use either method, it is necessary to assume a type of probability distribution for the in-place strengths and to determine the desired confidence level.

Concrete strengths are normally distributed if control is excellent or follow a lognormal distribution if control is poor (Mirza et al. 1979). The assumption of a normal distribution always gives a lower estimate of the 10% fractile, but when the coefficient of variation of in-place strength is less than 20%, any difference is insignificant. Adopting a normal distribution permits the use of many other statistical tools and techniques that have been derived on the basis of normality. Where a lognormal distribution is adopted, these tools can be used by working with the natural logarithms of the estimated in-place strengths.

There is less available guidance concerning the appropriate confidence level. Hindo and Bergstrom (1985) suggest that the 75% confidence level should be adopted for ordinary structures, 90% for important buildings, and 95% for crucial

Table 9.2—*K*-factors for one-sided tolerance limits on the 10% fractile (Natrella 1963)

n	Confidence level 75%	90%	95%
3	2.50	4.26	6.16
4	2.13	3.19	4.16
5	1.96	2.74	3.41
6	1.86	2.49	3.01
8	1.74	2.22	2.58
10	1.67	2.06	2.36
12	1.62	1.97	2.21
15	1.58	1.87	2.07
18	1.54	1.80	1.97
21	1.52	1.75	1.90
24	1.50	1.71	1.85
27	1.49	1.68	1.81
30	1.48	1.66	1.78
35	1.46	1.62	1.73
40	1.44	1.60	1.70

Note: n = number of specimens tested.

Table 9.3—*Z*-factors for use in Eq. (9-7) and (9-8) (Natrella 1963)

Confidence level, %	Z
75	0.67
90	1.28
95	1.64

components in nuclear power plants. ACI 228.1R reports that a 75% confidence level is widely used when assessing the in-place strength of concrete during construction. Tables 9.2, 9.3, and 9.4 give parameters, based on the normal strength distribution, to facilitate use of one of these three confidence levels in calculating the equivalent specified strength.

9.4.1 *Tolerance Factor Method (Hindo and Bergstrom 1988)*—One conventional approach to estimate a fractile value is to use a tolerance factor K that accommodates the uncertainties of the sample mean and the sample standard deviation caused by smaller sample sizes (Philleo 1981). If samples are drawn from a normal population, K values are based on a noncentral t distribution (Madsen et al. 1986) and tabulated for various sample sizes, confidence levels, and fractile values (Natrella 1963). The Tolerance Factor Method is presented in detail in ACI 228.1R as a relatively simple statistically based method for estimating the tenth percentile of the strength. Neglecting errors due to the use of empirically derived strength correction factors, the lower tolerance limit on the 10% fractile of the in-place strength data, $f_{0.10}$, is obtained from the following equation

$$f_{0.10} = \bar{f}_c - Ks_c \qquad (9\text{-}6)$$

where $\bar{f}_c$ and s_c are obtained from Eq. (9-2) and (9-3), respectively. The K value for one-sided tolerance limits on the 10% fractile value, shown in Table 9.2, decreases markedly as sample size n increases.

The estimate of the lower tenth percentile of the in-place strength obtained from Eq. (9-6) does not account for uncertainty introduced by the use of strength correction factors. This uncertainty, which does not diminish as the number of specimens increases, can be accounted for using a factor Z shown in Table 9.3, which is derived from the standard normal distribution. Thus, the equivalent design strength $f'_{c,eq}$, following the Tolerance Factor Method, is obtained from the equation

$$f'_{c,eq} = \bar{f}_c - \sqrt{(Ks_c)^2 + (Zs_a)^2} \qquad (9\text{-}7)$$

An example calculation using the Tolerance Factor Method is given in the Appendix.

9.4.2 *Alternate Method*—Bartlett and MacGregor (1995) suggest the tolerance factor approach may be unduly conservative in practice because core tests overestimate the true variability of the in-place strengths. Therefore, the resulting value of $f'_{c,eq}$ is too low because the value of s_c used in Eq. (9-7) is too high. The Tolerance Factor Method is not robust for data sets where undetected outliers may be present (Bartlett 2008). The precision inherent in the Tolerance Factor Method is significantly higher than that associated with current design, specification, and acceptance practices.

A study of many cores from members from different structures indicated the variability of average in-place strength between structures dominates the overall variability of in-place strength (Bartlett and MacGregor 1996b). Thus, core data can be used to estimate the average in-place strength and a lower bound on this average strength for a particular structure. Assuming the within-structure strength variation is accurately represented by the generic values shown in Table 3.1, the approximate 10% fractile of the in-place strength can then be obtained. Thus, the variability of measured core strengths, which can exceed true in-place strength variability due to hard-to-quantify testing factors, affects only the estimate of lower bound on the mean strength.

In this approach, the equivalent specified strength is estimated using a two-step calculation. First, a lower bound estimate on the average in-place strength is determined from core data. Then, the 10% fractile of the in-place strength, which is equivalent to the specified strength, is obtained.

The lower-bound estimate on the mean in-place strength $(\bar{f}_c)_{CL}$ can be determined for some desired confidence level CL using the following equation

$$(\bar{f}_c)_{CL} = \bar{f}_c - \sqrt{\frac{(Ts_c)^2}{n} + (Zs_a)^2} \qquad (9\text{-}8)$$

The first term under the square root represents the sample size effect on the uncertainty of the mean in-place strength. The factor T is obtained from a Student's t distribution with (n – 1) degrees of freedom (Larsen and Marx 2006; Natrella 1963), which depends on the desired confidence level. The second term under the square root reflects the uncertainty attributable to the strength correction factors. As in the tolerance factor approach, it depends on a factor Z obtained from the standard normal distribution for the desired confidence

Table 9.4—One-sided *T*-factors for use in Eq. (9-8) (Natrella 1963)

	Confidence level		
n	75%	90%	95%
3	0.82	1.89	2.92
4	0.76	1.64	2.35
5	0.74	1.53	2.13
6	0.73	1.48	2.02
8	0.71	1.41	1.90
10	0.70	1.38	1.83
12	0.70	1.36	1.80
15	0.69	1.34	1.76
18	0.69	1.33	1.74
21	0.69	1.33	1.72
24	0.69	1.32	1.71
30	0.68	1.32	1.70

Note: n = number of specimens tested.

Table 9.5—*C*-factors for use in Eq. (9-9)

Structure composed of:		One member	Many members
One batch of concrete		0.91	0.89
Many batches of concrete	Cast-in-place	0.85	0.83
	Precast	0.88	0.87

level. Tables 9.3 and 9.4 show values of Z and T, respectively, for the 75, 90, and 95% one-sided confidence levels. Bartlett and MacGregor (1995) suggest that a 90% confidence level is probably conservative for general use, but a greater confidence level can be appropriate if reliability is particularly sensitive to the in-place concrete strength.

The equivalent specified strength is defined using $(\bar{f}_c)_{CL}$ from the following expression

$$f'_{c,eq} = C(\bar{f}_c)_{CL} \tag{9-9}$$

Assuming normally distributed in-place strengths, the desired 10% strength fractile is obtained using the constant C equal to $(1 - 1.28V_{WS})$, where V_{WS} is the within-structure coefficient of variation of the strengths shown in Table 3.1. Therefore, C values depend on the number of batches, number of members, and type of construction, as shown in Table 9.5. To estimate the 13% fractile of in-place concrete strength, Bartlett and MacGregor (1995) recommend C values equal to 0.85 for cast-in-place construction consisting of many batches of concrete, or 0.90 for precast construction or cast-in-place members cast using a single batch of concrete. An example illustrating this approach is presented in the Appendix.

CHAPTER 10—REFERENCES

10.1—Referenced standards and reports

The standards and reports listed were the latest editions at the time this guide 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

228.1R In-Place Methods to Estimate Concrete Strength

228.2R Nondestructive Test Methods for Evaluation of Concrete in Structures

304R Guide for Measuring, Mixing, Transporting, and Placing Concrete

305R Hot Weather Concreting

308R Guide to Curing Concrete

309.1R Report on Behavior of Fresh Concrete During Vibration

311.1R ACI Manual of Concrete Inspection (SP-2) (Synopsis)

318/318M Building Code Requirements for Structural Concrete and Commentary

ASTM International

C39/39M Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

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

C94/C94M Standard Specification for Ready-Mixed Concrete

C642 Standard Test Method for Density, Absorption, and Voids in Hardened Concrete

C670 Standard Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials

C823/C823M Standard Practice for Examination and Sampling of Hardened Concrete in Constructions

E122 Standard Practice for Calculating Sample Size to Estimate, with Specified Precision, the Average for a Characteristic of a Lot or Process

E178 Standard Practice for Dealing with Outlying Observations

10.2—Cited references

Bartlett, F. M., 2008, Discussion of "Survey of Practice to Determine Strength of In Situ Concrete from Core Tests," *Journal of Performance of Constructed Facilities*, V. 22, No. 5, Oct., p. 348.

Bartlett, F. M., and MacGregor, J. G., 1994a, "Cores from High Performance Concrete Beams," *ACI Materials Journal*, V. 91, No. 6, Nov.-Dec., pp. 567-576.

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APPENDIX—EXAMPLE CALCULATIONS

A.1—Outlier identification in accordance with ASTM E178 criteria

An outlier, as determined using the method given in ASTM E178, is an observation that appears to significantly deviate from other observations of a sample. An illustration of this method is shown in Table A.1, for two cases where six cores are obtained from a single element. In each case, all six cores have the same diameter, ℓ/d, and are given identical conditioning treatments in accordance with ASTM C42/C42M before testing.

Table A.1—Identification of outliers according to ASTM E178

	Case 1	Case 1—w/o outlier	Case 2
Concrete compressive strength, psi	3200, 4270, 4380, 4470, 4500, and 4600	4270, 4380, 4470, 4500, and 4600	3900, 4270, 4380, 4470, 4500, and 4600
Average concrete compressive strength, psi	4240	4440	4350
Standard deviation, psi	520	125	250
(Average – minimum)/standard deviation	(4240 – 3200)/520 = 2.00	—	(4240 – 3900)/250 = 1.80
Critical value at 1% significance level	1.944	—	1.944
Outlier	Yes	—	No

The test statistic for checking if the smallest measured strength is an outlier according to ASTM E178 criteria is the difference between the average and minimum values divided by the sample standard deviation. For Case 1, this value is [(4240 psi – 3200 psi)/520 psi =] 2.00 as shown. From Table 1 of ASTM E178-08, the critical value for the one-sided test is 1.944 at the 1.0% significance level for a set of six observations. Thus, an observation this different from the mean value would be expected to occur by chance less than once every 100 times, and because this is unlikely, the low value of 3200 psi is an outlier and can be removed from the data set. This decision conforms to the ASTM E178 recommendation that a low significance level, such as 1%, be used as the critical value to test outlying observations. Removing the outlier increases the average concrete compressive strength by 5% and also markedly reduces the standard deviation from 520 psi to 125 psi as shown.

Case 2 is identical to Case 1, except that the smallest core strength is 3900 psi instead of 3200 psi. The low value is only [(4350 psi – 3900 psi)/250 psi =] 1.80 standard deviations below the mean value, which is less than the critical value of 1.729 given in Table 1 of ASTM E178 for the one-sided test at the 10% significance level, and much less than the critical value of 1.944 at the 1% significance level as shown. Thus, the low test result would be expected to occur by chance at least once every 10 times, and because this is likely the 3900 psi value is not an outlier according to ASTM E178 and should not be removed from the data set.

A.2—Student's *t* test for significance of difference between observed average values (for example, Larsen and Marx [2006])

The Student's *t* test is useful in evaluating the difference between average concrete strengths observed for cores from different structural components. For example, in Table A.2, concrete compressive strengths from four cores obtained from four beams are presented along with five cores obtained from five columns. The column cores are stronger, but is the difference large enough, given the small sample sizes, to consider the two data sets separately instead of combining them into a single set of nine observations for subsequent analysis?

Table A.2—Core compressive strength test results from different structural components

	Beam	Column
Concrete compressive strength, MPa	27.3, 29.0, 29.4, and 29.6	30.9, 31.2, 31.4, 31.8, and 31.9
Average concrete compressive strength, MPa	28.8	31.4
Standard deviation, MPa	1.05	0.42

Table A.3—Student's *t* test probability values for seven degrees of freedom

Critical value	Significance level	Probability
2.37	95%	1-in-10
3.50	99%	1-in-50
4.78	99.9%	1-in-500

To check whether the observed 2.6 MPa difference between average strengths is statistically significant and not simply a value that might often be exceeded by chance given the data scatter, a test based on the Student's t distribution can be performed. The test statistic t for testing the hypothesis that the mean values of the underlying populations are equal is

$$t = \frac{|\bar{x}_2 - \bar{x}_1|}{S_p\sqrt{\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}} \quad \text{(A-1)}$$

where the standard deviation of the pooled sample, S_p, is

$$S_p = \sqrt{\frac{(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2}{(n_1 + n_2 - 2)}} \quad \text{(A-2)}$$

In these equations, $\bar{x}$ is the sample mean, s is the sample standard deviation, n is the number of observations, and subscripts 1 and 2 are used to distinguish between the two populations. The test is only valid when the true variances of the two populations' σ^2 are equal, which can be verified using an F test (for example, Larsen and Marx [2006]), or when the sample sizes are nearly equal or large.

The rejection region is defined at a significance level α with degrees of freedom $df = n_1 + n_2 - 2$. Should the observed t value exceed the critical value, $t_{1-\alpha/2}$, which is tabulated in most statistical references (for example, Larsen and Marx [2006]), then the probability that a difference at least as large as that observed will occur by chance is α. Most engineers and statisticians would not consider a difference to be statistically significant if the associated significance level is greater than 5%. As noted in the first example, significance levels that are more stringent are recommended for outlier detection.

Thus, for the example data:

$$S_p = \sqrt{\frac{(4-1)(1.05\text{ MPa})^2 + (5-1)(0.42\text{ MPa})^2}{(4+5-2)}} = 0.76\text{ MPa} \quad \text{(A-3)}$$

so

Table A.4—4 x 8 core compressive strength data

	Original core test data	Equivalent in-place strength
Concrete compressive strength, psi	3930, 4320, 4740, 5040, and 5740	4010, 4410, 4830, 5140, and 5850
Mean concrete compressive strength $\bar{f}_c$, psi	—	4850
Standard deviation s_c, psi	—	700

$$t = \frac{|31.4\text{ MPa} - 28.8\text{ MPa}|}{0.76\text{ MPa}\sqrt{\left(\frac{1}{4} + \frac{1}{5}\right)}} = 5.1 \quad \text{(A-4)}$$

There are seven degrees of freedom for this data. The critical values are provided in Table A.3 for various significance levels.

The observed t statistic is slightly larger than the critical value at the 99.9% significance level. Thus, the probability of a difference of this magnitude occurring by chance is less than 1-in-500, and it can be concluded that the average strengths of cores from the beams and columns are significantly different. The data sets should not be combined, and distinct strength values should be computed separately for the columns and beams.

Most electronic spreadsheets provide formulas that conduct F tests and t tests on user-defined arrays of data. Typically, these are listed in the "Statistical" category.

A.3—Equivalent specified strength by tolerance factor approach (Hindo and Bergstrom 1985)

An equivalent specified strength is to be computed using the tolerance factor approach for five 4 x 8 in. (100 x 200 mm) cores that were dried in air at 60 to 70°F (16 to 21°C) and relative humidity less than 60% for 7 days before testing. The core test data are presented in Table A.4. Only strength corrections for moisture condition effects and drilling damage are necessary to obtain the equivalent in-place strengths. Equation (9-1) and the factors from Table 9.4 are used to create the following equation to compute the equivalent in-place strengths:

$$f_c = 1.02 f_{core}$$

If the uncertainty associated with use of strength correction factors is neglected, then the 75% confidence limit on the 10% fractile of in-place strength is obtained using Eq. (9-6) with $K = 1.96$ from Table 9.2

$$f_{0.10} = 4850\text{ psi} - 1.96 \times 700\text{ psi} = 3480\text{ psi} \quad \text{(A-5)}$$

The uncertainty introduced by strength correction factors F_d and F_{mc} is determined using Eq. (9-4)

$$s_a = 4850\text{ psi}\sqrt{0^2 + 0^2 + 0.025^2 + 0.025^2} = 171\text{ psi} \quad \text{(A-6)}$$

Thus, from Eq. (9-7), the 75% confidence limit on the 10% fractile of in-place strength determined using $Z = 0.67$ from Table 9.3 is

$$f'_{c,eq} = 4850 \text{ psi} - \sqrt{(1.96 \times 700 \text{ psi})^2 + (0.67 \times 171 \text{ psi})^2} = 3470 \text{ psi} \quad \text{(A-7)}$$

In this example, uncertainty due to strength correction factors does not greatly influence the result because the 10% fractile of in-place strength (Eq. (A-5)) is essentially identical to the equivalent specified strength (Eq. (A-7)). The equivalent specified strength is 3470 psi (24 MPa).

A.4—Equivalent specified strength by alternate approach (Bartlett and MacGregor 1995)

For the core test results from the previous example, the equivalent specified strength is to be determined using the alternate approach. The 90% one-sided confidence interval on the mean in-place strength is, using Eq. (A-8) with Z = 1.28 from Table 9.3 and T = 1.53 from Table 9.4

$$(\bar{f}_c)_{90} = 4850 \text{ psi} - \sqrt{\frac{(1.53 \times 700 \text{ psi})^2}{5} + (1.28 \times 171 \text{ psi})^2} = 4320 \text{ psi} \quad \text{(A-8)}$$

Hence, from Eq. (A-9) with C = 0.83 for a cast-in-place structure composed of many members cast from many batches

$$f'_{c,eq} = 0.83 \times 4320 \text{ psi} = 3580 \text{ psi} \quad \text{(A-9)}$$

The equivalent specified strength is therefore 3580 psi (25 MPa) using the alternate approach. It is slightly greater than that computed using the Tolerance Factor Method because, as described in Section 9.4.2, core test data tend to overestimate the true variability of in-place strengths.

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Guide for Obtaining Cores and Interpreting Compressive Strength Results

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