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 are obtained either to assess whether suspect concrete in a new structure complies with strength-based acceptance criteria or to evaluate the structural capacity of an existing structure based on the actual in-place concrete strength. In either case, the process of obtaining core specimens and interpreting the strength test results is often confounded by various factors that affect either the in-place strength of the concrete or the measured strength of the test specimen. The scatter in strength test data, which is unavoidable given the inherent randomness of in-place concrete strengths and the additional uncertainty attributable to the preparation and testing of the specimen, may further complicate compliance and evaluation decisions.

This guide summarizes current practices for obtaining cores and interpreting core compressive strength test results. Factors that affect the in-place concrete strength are reviewed so locations for sampling can be selected that are consistent with the objectives of the investigation. 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 also provides guidance for checking strength compliance of concrete in a structure under construction and methods for determining an equivalent specified strength to assess the capacity of an existing structure.

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

CONTENTS

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CHAPTER 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:

a) Assess whether suspect concrete in a new structure complies with strength-based acceptance criteria; or

b) Determine in-place concrete strengths in an existing structure for the evaluation of structural capacity.

In new construction, cylinder strength tests that fail to meet strength-based acceptance criteria may be investigated using the provisions given in ACI 318. This guide presents procedures for obtaining and testing the cores and interpreting the results in accordance with ACI 318 criteria.

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

The analysis of core test data can be difficult, leading to uncertain interpretations and conclusions. Strength interpretations should always be made by, or with the assistance of, an investigator experienced in concrete technology. The factors that contribute to the scatter of core strength test results include:

a) Systematic variation of in-place strength along a member or throughout the structure;

b) Random variation of concrete strength, both within one batch and among batches;

c) Low test results attributable to flawed test specimens or improper test procedures;

d) Effects of the size, aspect ratio, and moisture condition of the test specimen on the measured strengths; and

e) Additional uncertainty attributable to the testing that is present even for tests carried out in strict accordance with standardized testing procedures.

This guide summarizes past and current research findings concerning some of these factors and provides guidance for the interpretation of core strength test results. The presentation of these topics follows the logical sequence of tasks in a core-testing program. Chapter 2 reviews factors that affect the in-place concrete strength so that sampling locations consistent with the objectives of the investigation can be identified. Chapters 3, 4, and 5 present guidelines for planning the test program, obtaining the cores, and conducting the tests. Chapter 6 discusses the causes and magnitudes of the scatter usually observed in core test strengths and provides statistical methods for data analysis. Chapter 7 summarizes criteria given in ACI 318 for investigating low-strength tests in new construction. Chapter 8 presents methods for determining an equivalent f_c' for use in evaluating the capacity of an existing structure. Various example calculations appear in the Appendix.

CHAPTER 2—VARIATION OF IN-PLACE CONCRETE STRENGTH IN STRUCTURES

This chapter discusses the variation of in-place concrete strength in structures so that the investigator can anticipate the relevant factors in the early stages of planning the testing program. Selecting locations from which cores will be extracted and analyzing and interpreting the data obtained are simplified and streamlined when the 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 the delivery of quality concrete does not ensure quality in-place concrete, some project specifications require minimum core compressive strength results for concrete acceptance (Ontario Ministry of Transportation and Communications 1985). If excess mixing water was added at the site, or poor placing, consolidation, or curing practices were followed, core test results may not represent the quality of concrete as delivered to the site.

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

2.1—Bleeding

Shallow voids under coarse aggregate caused by bleeding can reduce the compressive strength transverse to the direction of casting and consolidation (Johnson 1973). The strength of cores with axes parallel to the direction of casting can therefore be greater than that of cores with axes perpendicular to the direction of casting. The experimental findings, however, are contradictory because some investigators observed appreciable differences between the strengths of horizontally and vertically drilled cores (Sanga and Dhir 1976; Takahata, Iwashimizu, and Ishibashi 1991) while others did not (Bloem 1965). Although the extent of bleeding varies greatly with mixture proportions and constituent materials, the available core strength data do not demonstrate a relationship between bleeding and the top-to-bottom concrete strength differences.

For concrete cast against earth, such as slabs and pavements, the absorptive properties of the subgrade also affect core strength. Cores from concrete cast on subgrades that absorb water from the concrete will generally be stronger than cores from concrete cast against metal, wood, polyethylene, concrete, or wet, saturated clay.

2.2—Consolidation

Concrete is usually consolidated by vibration to expel entrapped air after placement. The strength is reduced by about 7% for each percent by volume of entrapped air remaining after insufficient consolidation (Popovics 1969; Concrete Society 1987; ACI 309.1R). The investigator may need to assess the extent to which poor consolidation exists in the concrete in question by using the nondestructive techniques reported in ACI 228.2R.

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

2.3—Curing

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

High initial temperatures generated by hydration can significantly reduce the strength of the interior regions of massive elements. For example, the results shown in Fig 2.1 indicate that the strength of cores obtained from the middle of mock 760 x 760 mm (30 x 30 in.) columns is consistently

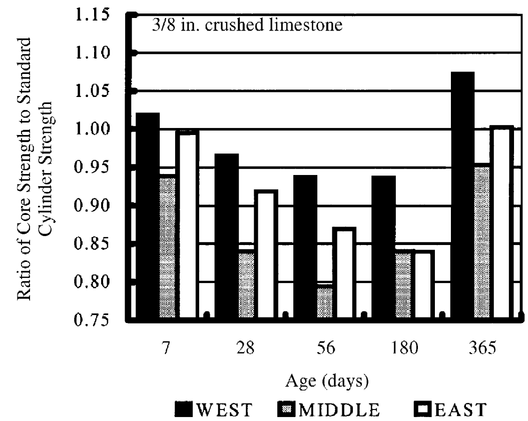

Fig. 2.1—Relationships between compressive strengths of column core samples and standard-cured specimens cast with high-strength concrete (Cook 1989).

less than the strength of cores obtained from the exterior faces (Cook 1989). The mock columns were cast using a high-strength concrete with an average 28-day standard cylinder strength in excess of 77 MPa (11,200 psi). Similarly, analysis of data 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 roughly 6% of the average strength in the specimen for every 10 °C (3% per 10 °F) increase of the average maximum temperature sustained during early hydration (Bartlett and MacGregor 1996a). The maximum temperatures recorded in these specimens varied between 45 and 95 °C (110 and 200 °F).

In massive concrete elements, hydration causes thermal gradients between the interior, which becomes hot, and the surfaces of the element, which remain relatively cool. In this case, the surfaces are restrained from contracting by the interior of the element, which can cause microcracking that reduces the 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).

The in-place strength of slabs or beams is more sensitive to the presence of adequate 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; Bloem 1968; Meynick and Samarin 1979; and Szypula and Grossman 1990) indicate that the strength of cores from poorly cured shallow elements averages 77% of the strength of companion 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 the strength loss at 91 days attributable to poor curing averages approximately 10% (Bartlett and MacGregor 1996b).

2.4—Microcracking

Microcracks in a core reduce the strength (Szypula and Grossman 1990), and their presence has been used to explain why the average strengths of cores from two ends of a beam cast from a single batch of concrete with a cylinder strength

Table 2.1—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%

of 54.1 MPa (7850 psi) differed by 11% of their average (Bartlett and MacGregor 1994a). Microcracks can be present if the core is drilled from a region of the structure that has been subjected to stress resulting from either applied loads or restraint of imposed deformations. Rough handling of the core specimen can also cause microcracking.

2.5—Overall variability of in-place strengths

Estimates of the overall variability of in-place concrete strengths reported by Bartlett and MacGregor (1995) are presented in Table 2.1. The variability is expressed in terms of the coefficient of variation V_{WS}, which is the ratio of the standard deviation of the in-place strength to the average in-place strength. The overall variability depends on the number of members in the structure, the number of concrete batches present, and whether the construction is precast or cast-in-place. The values shown are for concrete produced, placed, and protected in accordance with normal industry practice and may not pertain to concrete produced to either high or low standards of quality control.

CHAPTER 3—PLANNING THE TESTING PROGRAM

The procedure for planning a core-testing program depends on the objective of the investigation. Section 3.1 presents procedures for checking whether concrete in a new structure complies with strength-based acceptance criteria, while Section 3.2 presents those procedures for evaluating the strength capacity of an existing structure using in-place strengths.

As noted in Chapter 2, the strength of concrete in a placement usually increases with depth. In single-story columns, cores should be obtained from the central portion, where the strength is relatively constant, and not in the top 450 to 600 mm (18 to 24 in.), where it may decrease by 15%, or in the bottom 300 mm (12 in.), where it may increase by 10% (Bloem 1965).

3.1—Checking 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 the concrete represented by the low-strength test result. The investigator should only sample those areas where the suspect concrete was placed.

In some situations, such as a thin composite deck or a heavily reinforced section, it is difficult or impossible to obtain cores that meet all of the length and diameter requirements of ASTM C 42/C 42M. 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 same concrete mixture proportion: one that is acceptable based on standard strength tests and one that is questionable because of low strength test results. Nondestructive testing methods (ACI 228.1R) may indicate that the quality of concrete in the suspect columns exceeds that in the acceptable columns. Alternatively, it is appropriate to take 50 mm (2 in.) diameter cores from the columns where 25 mm (1 in.) maximum size aggregate was used. After trimming the cores, however, the ℓ/d will be less than 1.0 if the cover is only 50 mm (2 in.) and reinforcing bars cannot be cut. Acknowledging that strength tests of the "short" cores may not produce strength test results that accurately reflect the strength of the concrete in the columns, a relative comparison of the two concrete placements may be sufficient to determine if the strength of the concrete in question is comparable to the other placement or if more investigation is warranted.

3.2—Evaluating the capacity of an existing structure using in-place strengths

To establish in-place strength values for existing structures, the sample size and locations from which the cores will be extracted need to be carefully selected using procedures such as those described in ASTM E 122 and ASTM C 823.

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 E 122 recommends sample sizes be computed using Eq. (3-1) to achieve a 1-in-20 chance that the difference between the measured average of the sample and the average of the population, expressed as a percentage of the average of the population, will be less than some predetermined error.

$$n = \left(\frac{2V}{e}\right)^2 \tag{3-1}$$

where

n = the recommended sample size;

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

V = the estimated coefficient of variation of the population, in percent, and may be estimated from the values shown in Table 2.1 or from other available information.

For example, if the estimated coefficient of variation of the 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. (3-1) indicates that (for V = 0.15 and e = 0.10) a total of nine cores should be obtained. If a higher confidence level is desired, or if a 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 E 178, become more effective as the sample size increases. As indicated by the relationships between the percentage error and the recommended number of specimens shown in Fig. 3.1, however, the benefits of larger sample sizes tend to diminish. ASTM C 823 recommends that a

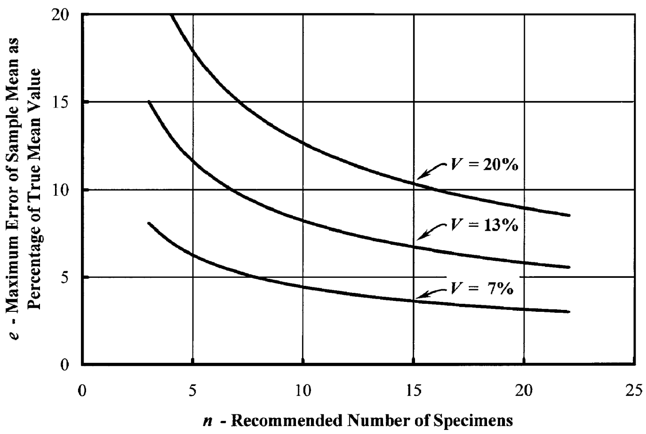

Fig. 3.1—Maximum error of sample mean for various recommended number of specimens.

minimum of five core test specimens be obtained for each category of concrete with a unique condition or specified quality, specified mixture proportion, or specified material property. ASTM C 823 also provides guidance for repeating the sampling sequence for large structures.

The investigator should select locations from which the cores will be extracted based on the overall objective of the investigation, 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. If the in-place strength for a specific component or group of components is sought, the investigator should extract the cores at randomly selected locations from those specific components.

When determining sample locations, the investigator should consider whether different strength categories of concrete may be present in the structure. For example, the 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 categories of concrete, the investigator should objectively select sample locations so as not to unfairly bias the outcome. Alternatively, he or she should randomly select a sufficient number of sampling locations for each category of concrete 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 C 823 contain further guidance concerning sampling techniques.

CHAPTER 4—OBTAINING SPECIMENS FOR TESTING

Coring techniques should result in high-quality, undamaged, representative test specimens. The investigator should delay coring until the concrete being cored has sufficient strength and hardness so that the bond between the mortar and aggregate will not be disturbed. ASTM C 42/C 42M suggests that the concrete should not be cored before it is 14 days old, unless other information indicates that the concrete can withstand the coring process without damage. ASTM C 42/C 42M further suggests that in-place nondestructive tests (ACI 228.1R) may be performed to estimate the level of strength development of the concrete before coring is attempted.

Core specimens for compression tests should preferably not contain reinforcing bars. These can be located before drilling the core using a pachometer or cover meter. Also, avoid cutting sections containing conduit, ductwork, or prestressing tendons.

As described in Chapter 6, the strength of the specimen 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 diameters of 100 to 150 mm (4 to 6 in.) and ℓ/d ratios between 1.5 and 2 to minimize error introduced by the strength correction factors (Neville 2001).

The drilling of the core should be carried out 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 the introduction of large strains in the core. The drill bit should be lubricated with water and should be resurfaced or replaced when it becomes worn. The operator should be informed beforehand that the cores are for strength testing and, therefore, require proper handling and storage.

Core specimens in transit require protection from freezing and damage because a damaged specimen will not accurately represent the in-place concrete strength.

A core drilled with a water-cooled bit results in a moisture gradient between the exterior and interior of the core that adversely affects its compressive strength (Fiorato, Burg, Gaynor 2000; Bartlett and MacGregor 1994c). ASTM C 42/C 42M presents moisture protection and scheduling requirements that are intended to achieve a moisture distribution in core specimens that better represent the moisture distribution in the concrete before the concrete was wetted during drilling. The restriction concerning the commencement of core testing provides a minimum time for the moisture gradient to dissipate.

The investigator, or a representative of the investigator, 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 location in the structure from which each core is extracted and any features that may affect the strength, such as cracks or honeycombs. Similar features observed by careful inspection of the surrounding concrete should also be documented. Given the likelihood of questionable low-strength values, any information that may later identify reasons for the low values will be valuable.

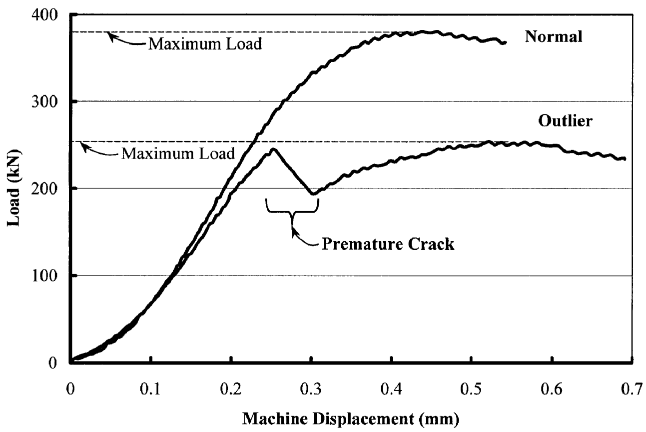

Fig. 5.1—Use of load-machine displacement curves to identify outlier due to flawed specimen (Bartlett and MacGregor 1994a).

CHAPTER 5—TESTING THE CORES

ASTM C 42/C 42M presents standard methods for conditioning the specimen, preparing the ends before testing, and correcting the test result for the core length-to-diameter ratio. Other standards for measuring the length of the specimen and performing the compression test are referenced and information required in the test report is described.

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

When testing cores with small diameters, careful alignment of the specimen in the testing machine is necessary. If the diameter of the suspended spherically seated bearing block exceeds the diameter of the specimen, the spherical seat may not rotate into proper alignment, causing nonuniform contact against the specimen. ASTM C 39 limits the diameter of the upper bearing face 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. 5.1 are for 100 x 100 mm (4 x 4 in.) cores, obtained from one beam, that were given identical moisture treatments. The lower curve is abnormal because the load drops markedly before reaching its maximum value. This curve is consistent with a premature splitting failure and may be attributed to imperfect preparation of the ends of the specimen. Thus, the low result can be attributed to a credible physical cause and should be excluded from the data set.

Sullivan (1991) describes the use of nondestructive tests to check for abnormalities in cores before the compressive strength tests are conducted.

If the investigator cannot find a physical reason to explain why a particular result is unusually low or unusually high, then statistical tests given in ASTM E 178 can be used to determine whether the observation is an "outlier." When the sample size is less than six, however, these tests do not consistently classify values as outliers that should be so classified (Bartlett and MacGregor 1995). An example calculation using ASTM E 178 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 E 178 criteria but the reason for the outlier cannot be determined, then the investigator should report the suspect values and indicate whether they have been used in subsequent analyses.

CHAPTER 6—ANALYZING STRENGTH TEST DATA

The analysis and interpretation of core strength data are complicated by the large scatter usually observed in the test results. This chapter describes the expected scatter of properly conducted tests of cores from a sample of homogeneous material, discusses other possible 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 Ang and Tang (1975) or Benjamin and Cornell (1970).

6.1—ASTM C 42/C 42M precision statements

ASTM C 42/C 42M provides precision statements that quantify the inherent error associated with testing cores from a homogeneous material tested in accordance with the 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 the single operator coefficient of variation varied from 3.1 to 3.4% for cores from the three different slabs, and measured values of the multilaboratory coefficient of variation varied between 3.7 and 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 C 42/C 42M due to one or more of the following reasons:

a) The limits stated in ASTM C 42/C 42M 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 their average, and also 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;

b) The variability of the in-place concrete properties can exceed that in the slabs investigated for the multilaboratory study reported by Bollin (1993); and

c) The testing accuracy can be less rigorous than that achieved by the 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 C 42/C 42M. A practical use of this measure is to check whether the difference between strength test results of two individual cores obtained from the same sample of material does not differ by more than 9% of their average. The difference between consecutive tests (or any two randomly selected tests) is usually much less than the overall range between the 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 6.1 shows values corresponding to the ASTM C 42/C 42M single-operator coefficient of variation of 3.2%, which indicate, for example, in a set of five cores from the same sample of material, 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 C 670 gives 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 reproducibility of the core test, as performed in accordance with ASTM C 42/C 42M. 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, for example, if two independent laboratories test cores from the same sample of material in accordance with criteria given in ACI 318, and each laboratory tests three specimens in conformance with ASTM C 42/C 42M, there remains a 1-in-20 chance that the reported average strengths will differ by more than 13% of their average.

Table 6.1—Probable range of core strengths due to single-operator error

Number of cores	Expected range of core strength as % of average core strength	Range with 5% chance of being exceeded as % 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

Table 6.2—Strength correction factors for length-to-diameter ratio

ℓ/d	ASTM C 42/C 42M	BS 1881
2.00	1.00	1.00
1.75	0.98	0.97
1.50	0.96	0.92
1.25	0.93	0.87
1.00	0.87	0.80

6.2—Review of core strength correction factors

The measured strength of a core depends partly on factors that include the ratio of length to diameter of the specimen, the diameter, the moisture condition at the time of testing, the presence of reinforcement or other inclusions, and the direction of coring. Considerable research has been carried out concerning these factors, and strength correction factors have been proposed to account for their effects. The research findings, however, have often been contradictory. Also, published strength correction factors are not necessarily exact and may not be universally applicable because they have been derived empirically from specific sets of data. To indicate the degree of uncertainty associated with these factors, this section summarizes some of the relevant research findings. Chapter 8 presents specific strength correction factor values.

6.2.1 *Length-to-diameter ratio*—The length-to-diameter ratio ℓ/d was identified in the 1927 edition of ASTM C 42/C 42M as a factor that influences the measured compressive strength of a core, and minor variations of the original ℓ/d strength correction factors have been recommended 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 length of the specimen more effectively and so provide confinement (Newman and Lachance 1964; Ottosen 1984). The end effect is largely eliminated in standard concrete compression test specimens, which have a length to diameter ratio of two.

Table 6.2 shows values of strength correction factors recommended in ASTM C 42/C 42M and British Standard BS 1881 (1983) for cores with ℓ/d between 1 and 2. Neither standard permits testing cores with ℓ/d less than 1. The recommended values diverge as ℓ/d approaches 1. The ASTM factors are average values that pertain to dry or soaked specimens with strengths between 14 and 40 MPa (2000 and 6000 psi). ASTM C 42/C 42M states that actual ℓ/d correction factors depend on the strength and elastic modulus of the specimen.

Bartlett and MacGregor (1994b) report that the necessary strength correction is slightly less for high-strength concrete and soaked cores, but they recommend strength correction factor values that are similar to those in ASTM C 42/C 42M. They also observed that the 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.

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

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

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

6.2.3 *Moisture condition*—Different moisture-conditioning treatments have a considerable effect on the 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 average values. Soaking causes the 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 the 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 duration of the period between the end of the moisture treatment and the performance of the test varies significantly, then additional variability of the measured strengths can be introduced.

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

6.2.4 *Presence of reinforcing bars or other inclusions*—The investigator should avoid specimens containing embedded reinforcement because it may influence the measured compressive strength. Previous editions of ASTM C 42 have recommended trimming the core to eliminate the reinforcement provided, ℓ/d, of at least 1.0 can be maintained.

6.2.5 *Coring direction*—Cores drilled in the direction of placement and compaction (which would be loaded in a direction perpendicular to the horizontal plane of concrete as placed, according to ASTM C 42/C 42M) can be stronger than cores drilled normal to this direction because bleed water can collect underneath coarse aggregate, as described in Chapter 2. In practice, it is often easier to drill horizontally into a column, wall, or beam in a direction perpendicular to the direction of placement and compaction. The influence of coring direction 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 the direction of drilling, the investigator should assess this directly using specimens drilled in different directions from the structure in question, if possible.

6.3—Statistical analysis techniques

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

Statistical tests are particularly useful for analyzing preliminary hypotheses developed during an initial review of the data, which are logically consistent with the circumstances of the investigation and are credible in light of past experience. While it is possible to conduct “fishing expeditions” using 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 a clear understanding of the essential physical and behavioral characteristics represented in the data. Instead, it is preferable to first identify the possible factors that affect the strength in a particular instance and then use statistical analyses to verify whether these factors are in fact 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 the underlying populations, from which the samples are drawn, are different. ASTM C 823 recommends the use of the Student’s *t* test to investigate whether the average strength of cores obtained from concrete of questionable quality differs from the average strength of cores obtained from concrete of good quality. Details of the Student’s *t* test can be found in most statistical references (Benjamin and Cornell 1970; Ang and Tang 1975), and a numerical example illustrating its use is presented in the Appendix.

There are two types of error 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, and a Type II error occurs when a hypothesis is accepted when, in fact, it is false. In the practice of 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. When decisions are made on the basis of a small number of tests (and so the likelihood of an error is large), 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

Table 8.1—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†		
As-received‡	$1-\{0.130-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
Soaked 48 h	$1-\{0.117-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
Air 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		
50 mm (2 in.)	1.06	11.8
100 mm (4 in.)	1.00	0.0
150 mm (6 in.)	0.98	1.8
F_{mc}: core moisture content		
As-received‡	1.00	2.5
Soaked 48 h	1.09	2.5
Air 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. (8-1).

†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 C 42/C 42M.

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

In new construction, low cylinder strength tests are investigated in accordance with the provisions of ACI 318. The suspect concrete is considered structurally adequate if the average strength of the three cores, corrected for ℓ/d in accordance with ASTM C 42/C 42M, exceeds $0.85f_c'$, and no individual strength is less than $0.75f_c'$. Generally, these criteria have served producers and consumers of concrete well. ACI 318 recognizes that the strengths of cores are potentially lower than the strengths of cast specimens representing the quality of concrete delivered to the project. This relationship is corroborated by observations that the strengths of 56-day-old soaked cores averaged 93% of the strength of standard 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 that 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 8—DETERMINING AN EQUIVALENT f_c' VALUE FOR EVALUATING THE STRUCTURAL CAPACITY OF AN EXISTING STRUCTURE

This chapter 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 strength of concrete f_c'. This chapter presents two methods for estimating the lower tenth-percentile value from core test data.

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

8.1—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

$$f_c = F_{\ell/d}F_{dia}F_{mc}F_d f_{core} \tag{8-1}$$

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 the effects of the length-to-diameter 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 8.1 shows the mean values of the strength correction factors reported by Bartlett and MacGregor (1995) based on data for normalweight concrete with strengths between 14 and 92 MPa (2000 and 13,400 psi). The right-hand column shows coefficients of variation V that indicate the uncertainty of the mean value. It follows that a 100 mm (4 in.) diameter core with ℓ/d = 2 that has been soaked 48 h before testing has $f_c = 1.0 \times 1.0 \times 1.09 \times 1.06 f_{core}$ = 1.16 f_{core}.

8.2—Uncertainty of estimated in-place strengths

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

$$\bar{f}_c = \frac{1}{n}\sum_{i=1}^{n} f_{ci} \quad (8\text{-}2)$$

where n is the number of cores, and f_{ci} is the equivalent in-place strength of an individual core specimen, calculated using Eq. (8-1). The sample standard deviation of the in-place strength s_c is obtained from the following equation

$$s_c = \sqrt{\sum_{i=1}^{n} \frac{(f_{ci} - \bar{f}_c)^2}{(n-1)}} \quad (8\text{-}3)$$

The sample mean and the 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 (Ang and Tang 1975).

The accuracy of the estimated in-place strengths also depends on the accuracy of the various strength correction factors used in Eq. (8-1). The standard deviation of the in-place strength due to the empirical nature of the strength correction factors s_a can be obtained from the following equation

$$s_a = \bar{f}_c \sqrt{V_{\ell/d}^{\,2} + V_{dia}^{\,2} + V_{mc}^{\,2} + V_d^{\,2}} \quad (8\text{-}4)$$

The right column of Table 8.1 shows the 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 due to a particular strength correction factor need only be included in Eq. (8-4) if the corresponding factor used in Eq. (8-1) to obtain the in-place strength differs from 1.0. If the 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, the errors due to the strength correction factors remain constant irrespective of the number of specimens obtained.

The overall uncertainty of the estimated in-place strengths is a combination of the sampling uncertainty and the uncertainty caused by the 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

$$s_o = \sqrt{s_c^{\,2} + s_a^{\,2}} \quad (8\text{-}5)$$

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

The criteria in ACI 318 for proportioning concrete mixtures require that the target strength exceeds f_c' to achieve approximately a 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 3.5 MPa (500 psi) below f_c' if the specified strength is less than 35 MPa (5000 psi), or below $0.90f_c'$ if the specified strength exceeds 35 MPa (5000 psi). These criteria imply that f_c' represents approximately the 10% fractile, or the lower tenth-percentile value, of the 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, Carino, and Reeve 1986).

This practice was corroborated by a study that showed f_c' represents roughly the 13% fractile of the 28-day in-place strength in walls and columns and roughly 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 an equivalent specified strength because the nominal strength of a column is more sensitive to the concrete compressive strength than a beam or slab. Therefore, a procedure that assumes that the specified strength is equal to the 13% fractile of the in-place strength is appropriate, and one that assumes that f_c' is equivalent to the 10% fractile of the in-place strength is slightly conservative.

8.4—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 8.3, is roughly equivalent to f_c'. In general, the following considerations should be addressed:

a) Factors that bias the core test result, which can be accounted for using the strength correction factors discussed in Chapter 6;

b) Uncertainty of each strength correction factor used to estimate the in-place strength;

c) Errors of the measured average value and measured standard deviation that are attributable to sampling and therefore decrease as the sample size increases;

d) Variability attributable to acceptable deviations from standardized testing procedures that can cause the

Table 8.2—*K*-factors for one-sided tolerance limits on 10% fractile (Natrella 1963)

	Confidence level		
n	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.

measured standard deviation of strength tests to exceed the true in-place strength variation; and

e) Desired confidence level, which represents the likelihood that the fractile value calculated using the 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.

There is a general consensus that concrete strengths are normally distributed if control is excellent or follow a lognormal distribution if control is poor (Mirza, Hatzinikolas, and MacGregor 1979). The assumption of a normal distribution always gives a lower estimate of the 10% fractile; although, if the coefficient of variation of the in-place strength is less than 20%, any difference is of little practical significance. It is convenient to adopt the normal distribution because this permits the use of many other statistical tools and techniques that have been derived on the basis of normality. If a lognormal distribution is adopted, however, 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 very important buildings, and 95% for crucial components in nuclear power plants. ACI 228.1R reports that a confidence level of 75% is widely used in practice when assessing the in-place strength of concrete during construction. Tables 8.2, 8.3, and 8.4 give parameters, based on a normal distribution of strengths, to facilitate the use of one of these three confidence levels in calculating the equivalent specified strength.

8.4.1 *Tolerance factor approach*—The conventional approach to estimate a fractile value is to use a tolerance factor K that accommodates the uncertainties of both the sample mean and the sample standard deviation caused by smaller sample sizes (Philleo 1981). If the samples are drawn from a normal population, values of K are based on a noncentral t distribution (Madsen, Krenk, and Lind 1986) and are tabulated for various sample sizes, confidence levels, and fractile values in Natrella (1963). The tolerance factor approach 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

$$f_{0.10} = \bar{f}_c - Ks_c \qquad (8\text{-}6)$$

where $\bar{f}_c$ and s_c are obtained from Eq. (8-2) and (8-3), respectively. The value of K for one-sided tolerance limits on the 10% fractile value, shown in Table 8.2, decreases markedly as the sample size n increases.

The estimate of the lower tenth-percentile of the in-place strength obtained from Eq. (8-6) does not account for the uncertainty introduced by the use of the 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 8.3, which is derived from the standard normal distribution. Thus, the equivalent design strength $f'_{c,eq}$, following the tolerance factor approach, is obtained from the equation

Table 8.3—*Z*-factors for use in Eq. (8-7) and (8-8) (Natrella 1963)

Confidence level, %	*Z*
75	0.67
90	1.28
95	1.64

Table 8.4—One-sided *T*-factors for use in Eq. (8-8) (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.

$$f'_{c,eq} = \bar{f}_c - \sqrt{(Ks_c)^2 + (Zs_a)^2} \qquad (8\text{-}7)$$

An example calculation using the tolerance factor approach is given in the Appendix.

8.4.2 *Alternate approach*—Bartlett and MacGregor (1995) suggest that the tolerance factor approach may be unduly conservative in practice because core tests tend to 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. (8-7) is too high. Also, the precision inherent in the tolerance factor approach is significantly higher than that associated with current design, specification, and acceptance practices.

A study of a large number of cores from members from different structures indicated that the variability of the average in-place strength between structures dominates the overall variability of the 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 that the actual within-structure strength variation is accurately represented by the generic values shown in Table 2.1, the approximate 10% fractile of the in-place strength can then be obtained. Thus, the variability of the measured core strengths, which can exceed the true in-place strength variability due to testing factors that are hard to quantify, affects only the estimate of the 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 the core data. Then the 10% fractile of the in-place strength, which is equivalent to the specified strength, is obtained.

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

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

The first term under the square root represents the effect of the sample size 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 (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 level. Tables 8.3 and 8.4 show values of Z and T for the 75, 90, and 95% one-sided confidence levels, respectively. Bartlett and MacGregor (1995) suggest that a 90% confidence level is probably conservative for general use, but a greater confidence level may be appropriate if the reliability is particularly sensitive to the in-place concrete strength.

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

Table 8.5—*C*-factors for use in Eq. (8-9)

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

$$f'_{c,eq} = C(\bar{f}_c)_{CL} \qquad (8\text{-}9)$$

Assuming the in-place strengths to be normally distributed, the desired 10% strength fractile is obtained using the constant C equal to $(1\text{-}1.28V_{WS})$, where V_{WS} is the within-structure coefficient of variation of the strengths shown in Table 2.1. Therefore, values of C depend on the number of batches, number of members, and type of construction, as shown in Table 8.5. To estimate the 13% fractile of the in-place concrete strength, Bartlett and MacGregor (1995) recommend values of C 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 9—SUMMARY

This guide summarizes current practices for obtaining cores and interpreting core compressive strength test results in light of past and current research findings. Parallel procedures are presented for the 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 that is equivalent to the specified compressive strength f'_c and so can be directly substituted into conventional strength equations with customary strength reduction factors for the strength evaluation of an existing structure. It is inappropriate to use the procedures for determining an 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 2 describes how bleeding, consolidation, curing, and microcracking affect the in-place concrete strength in structures so that the investigator can account for this strength variation when planning the testing program. Chapter 3 identifies preferred sample locations and provides guidance on the number of specimens that should be obtained. Chapter 4 summarizes coring techniques that should result in high-quality, undamaged, representative test specimens. It is recommended that specimens with diameters of 100 to 150 mm (4 to 6 in.) and length-to-diameter ratios between 1.5 and 2 be obtained wherever possible to minimize any errors introduced by the strength correction factors for nonstandard specimens.

Chapter 5 describes procedures for testing the cores and detecting "outliers" by inspection of load-machine displacement curves or using statistical tests from ASTM E 178. Chapter 6 summarizes the subsequent analysis of strength test data including the use of ASTM C 42/42 M precision statements that quantify the expected variability of properly conducted tests for a sample of homogeneous material, research findings concerning the 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 7 briefly elaborates on criteria presented in ACI 318 for using core test results to investigate low-strength cylinder test results in new construction.

Chapter 8 presents two methods for estimating the lower tenth-percentile value of the in-place concrete strength using core test data to quantify the in-place strength. This value is equivalent to the specified concrete strength f_c' and so can be directly substituted into conventional strength equations with customary strength reduction factors for the strength evaluation of an existing structure.

Example calculations are presented in an appendix for: outlier identification in accordance with ASTM E 178 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 8.

CHAPTER 10—REFERENCES

10.1—Referenced standards and reports

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

American Concrete Institute

228.1R In-Place Methods for Determination of Strength of Concrete

228.2R Nondestructive Test Methods for the Evaluation of Concrete in Structures

309.1R Behavior of Fresh Concrete During Vibration

311.1R ACI Manual of Concrete Inspection, SP-2

318 Building Code Requirements for Reinforced Concrete and Commentary

ASTM International

C 39 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

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

C 670 Standard Practice for Preparing Precision and Bias Statements for Test Methods for Construction Materials

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

E 122 Standard Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process

E 178 Standard Practice for Dealing with Outlying Observations

10.2—Cited references

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APPENDIX—EXAMPLE CALCULATIONS

A1—Outlier identification in accordance with ASTM E 178 criteria

Six cores are obtained from a single element. All have the same diameter ℓ/d and are given identical conditioning treatments in accordance with ASTM C 42/C 42M before testing. The measured strengths are 22.1, 29.4, 30.2, 30.8, 31.0, and 31.7 MPa (3200, 4270, 4380, 4470, 4500, and 4600 psi). The average strength is 29.2 MPa (4240 psi), and the standard deviation is 3.56 MPa (520 psi). If the smallest strength value is an outlier and so can be removed from the data set, the average strength will increase by almost 5% and the standard deviation will be markedly reduced.

The test statistic for checking if the smallest measured strength is an outlier according to ASTM E 178 criteria is the difference between the average and minimum values divided by the sample standard deviation. In this case it equals SI: (29.2 MPa – 22.1 MPa)/3.56 MPa = 1.99 [(4240 psi – 3200 psi)/520 psi = 2.00]. From Table 1 of ASTM E 178-80, the critical value for the two-sided test is 1.973 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 22.1 MPa (3200 psi) is an outlier and can be removed from the data set. This decision conforms to the ASTM E 178 recommendation that a low significance level, such as 1%, be used as the critical value to test outlying observations.

If, in this example, the smallest core strength was 26.9 MPa (3900 psi) instead of 22.1 MPa (3200 psi), the average of the six strengths would be 30.0 MPa (4350 psi) with a standard deviation of 1.71 MPa (250 psi). The low value is (30.0 MPa – 26.9 MPa)/1.71 MPa = 1.81 standard deviations below the mean value [(4350 psi – 3900 psi)/250 psi = 1.80], which is less than the critical value of 1.822 given in Table 1 of ASTM E 178-80 for the two-sided test at the 10% significance level. The low test result would be expected to occur by chance at least once every 10 times, and because this is likely the 26.9 MPa (3900 psi) value is not an outlier according to ASTM E 178 and should not be removed from the data set.

A2—Student's *t* test for significance of difference between observed average values

It is not always obvious that any difference between average concrete strengths observed for cores from different structural components indicate a true difference of concrete quality between the components. For example, assume four cores obtained from four beams have measured strengths of 27.3, 29.0, 29.6, and 29.4 MPa (3960, 4210, 4300, and 4270 psi), which average 28.8 MPa (4180 psi) with a standard deviation of 1.05 MPa (155 psi). Five cores obtained from five columns have measured strengths of 31.2, 31.8, 30.9, 31.4, and 31.9 MPa (4520, 4610, 4480, 4560, and 4630 psi), which average 31.4 MPa (4560 psi) and have a standard deviation of 0.42 MPa (62 psi). Clearly 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?

To check whether the observed 2.6 MPa (380 psi) difference between the average strengths is statistically significant and not simply a value that might often be exceeded by chance given the scatter of the data, a test based on the Student's *t* distribution (Benjamin and Cornell 1970; Ang and Tang 1975) can be performed. The test statistic *t* for testing the hypothesis that the mean values of the underlying populations are equal is

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

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 (Benjamin and Cornell 1970; Ang and Tang 1975).

The rejection region is defined at a significance level α with degrees of freedom $df = \nu_1 + \nu_2 - 2$. Should the observed t value exceed the critical value, $t_{1-\alpha/2}$, which is tabulated in most statistical references (Benjamin and Cornell 1970; Ang and Tang 1975), 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, more stringent significance levels are recommended for outlier detection.

Thus, for the example data:

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

$$= 0.76 \text{ MPa}$$

$$\left(S_p = \sqrt{\frac{(4-1)(155\text{ psi})^2 + (5-1)(62\text{ psi})^2}{(4+5-2)}} = 112\text{ psi}\right)$$

so

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

$$\left(t = \frac{|4560\text{psi} - 4185\text{psi}|}{112\text{ psi}\sqrt{\left(\frac{1}{4}+\frac{1}{5}\right)}} = 5.0\right)$$

For this case with seven degrees of freedom, the critical values for the two-sided test are 2.37 at the 95% significance level, 3.50 at the 99% significance level, and 4.78 at the 99.9% significance level (Ang and Tang 1975). Because the observed t statistic is slightly larger than the critical value at the 99.9% significance level, the value $1 - \alpha/2$ exceeds 99.9%, and so α is less than 0.2%. 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 the cores from the beams and the columns are significantly different. The data sets should not be combined, and distinct strength values should be computed separately for the columns and for the beams.

A3—Equivalent specified strength by tolerance factor approach

An equivalent specified strength is to be computed using the tolerance factor approach for five 100 x 200 mm (4 x 8 in.) cores that have been air-dried in accordance with ASTM C 42/C 42M before testing. The test strengths are 27.1, 29.8, 32.7, 34.8, and 39.6 MPa (3930, 4320, 4740, 5040, and 5740 psi). Only strength corrections for the effects of the moisture condition and the damage due to drilling are necessary to obtain the equivalent in-place strengths. Thus, using Eq. (8-1) and the factors from Table 8.4, $f_c = 1.02\ f_{core}$, and the corresponding in-place strengths, rounded to the nearest 0.1 MPa (10 psi) in accordance with ASTM practice, are 27.6, 30.4, 33.3, 35.5, and 40.4 MPa (4010, 4410, 4830, 5140, and 5850 psi). The mean in-place strength $\bar{f_c}$ is 33.4 MPa (4850 psi), and the sample standard deviation of the in-place strength values s_c is 4.9 MPa (700 psi). If the uncertainty associated with the use of the strength correction factors is neglected, then the 75% confidence limit on the 10% fractile of the in-place strength is obtained using Eq. (8-6) with, from Table 8.2, $K = 1.96$

$$f_{0.10} = 33.4\text{MPa} - 1.96 \times 4.90\text{ MPa} = 23.8\text{ MPa} \quad \text{(A-5)}$$

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

The uncertainty introduced by strength correction factors F_d and F_{mc} is determined using Eq. (8-4)

$$s_a = 33.4\text{ MPa}\sqrt{0^2 + 0^2 + 0.025^2 + 0.025^2} = 1.18\text{MPa} \quad \text{(A-6)}$$

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

Thus, from Eq. (8-7), the 75% confidence limit on the 10% fractile of the in-place strength, determined using $Z = 0.67$ from Table 8.3 is

$$f'_{c,eq} = 33.4\text{ MPa} - \sqrt{(1.96 \times 4.9\text{MPa})^2 + (0.67 \times 1.18\text{MPa})^2} \quad \text{(A-7)}$$

$$= 23.8\text{ MPa}$$

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

$$= 3470\text{ psi})$$

In this example, the uncertainty due to the strength correction factors does not greatly influence the result because the 10% fractile of the in-place strength, Eq. (A-5), is essentially identical to the equivalent specified strength, Eq. (A-7). The equivalent specified strength is 23.8 MPa (3470 psi).

A4—Equivalent specified strength by alternate approach

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. (8-8) with Z = 1.28 from Table 8.3 and T = 1.53 from Table 8.4,

$$(\bar{f_c})_{90} = 33.4\text{ MPa} - \sqrt{\frac{(1.53 \times 4.9\text{ MPa})^2}{5} + (1.28 \times 1.18\text{ MPa})^2} \quad \text{(A-8)}$$

$$= 29.7\text{ MPa}$$

$$\left((\bar{f_c})_{90} = 4850\text{ psi} - \sqrt{\frac{(1.53 \times 700\text{ psi})^2}{5} + (1.28 \times 171\text{ psi})^2}\right.$$

$$= 4320\text{ psi})$$

Hence, from Eq. (8-9) with C = 0.83 for a cast-in-place structure composed of many members cast from many batches

$$f'_{c,eq} = 0.83 \times 29.7\text{ MPa} = 24.7\text{ MPa} \quad \text{(A-9)}$$

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

The equivalent specified strength is therefore 24.7 MPa (3580 psi) using the alternate approach. It is slightly greater than that computed using the tolerance factor method because, as described in Section 8.4.2, core test data tend to overestimate the true variability of the in-place strengths.