

ACI MANUAL OF CONCRETE INSPECTION

Reported by ACI Committee 311

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American Concrete Institute®

ACKNOWLEDGMENTS

This manual is based on information from many sources, organizations, and individuals whose contributions are gratefully acknowledged. Published references are listed at the end of the text. References to standard specifications and methods of testing are listed separately.

The original manuscript was prepared by Joe W. Kelly, Chair of ACI Committee 311, and revised over a period of years to achieve a first edition in 1941. The second edition, also in 1941, included a number of corrections and minor revisions. The third edition, in 1955, incorporated many constructive suggestions from users. The fourth edition, in 1957, brought several sections up to date and contained editorial corrections.

The fifth edition provided new information on settlement of concrete, shoring and forming, strength requirements, cold-weather concreting, and shotcrete. The sixth edition primarily provided updated information in all chapters, and included editorial and substantive changes throughout.

The seventh edition presented a complete revision of the manual by eliminating sections of the previous edition covering concrete methods no longer in use. Chapters 2, 11, 12, 13, 14, 15 (partial), 16, 17, and 18 covered material that was included in the manual for the first time. The eighth and ninth editions were revised to reflect changes in technology and construction practices.

This tenth edition presents an extensive revision and update to the text along with new photos, charts, and forms.

The committee wishes to thank Anne Balogh for her extensive work in redrafting and unifying the previous text in preparation for this edition. Additionally, the committee wishes to express its thanks to Portland Cement Association (PCA) for generously providing many of the new photos contained in the manual and to committee member Michelle L. Wilson who organized and coordinated the selection of photos and graphics throughout the document.

Finally, the committee wishes to thank Chair George R. Wargo for his efforts in coordinating the work of all contributors.

ACI Manual of Concrete Inspection

Reported by ACI Committee 311

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PREFACE

This manual is intended to guide, assist, and instruct concrete inspectors and others engaged in concrete construction and testing, including field engineers, construction superintendents, supervisors, laboratory and field technicians, and workers. Designers may also find the manual to be a valuable reference by using the information to better adapt their designs to the realities of field construction. Because of the diverse possible uses of the manual and the varied backgrounds of the readers, it includes the reasoning behind the technical instructions.

The field of concrete construction has expanded dramatically over the years to reflect the many advances that have taken place in the concrete industry. Although many of the fundamentals presented in previous editions of this manual remain relevant and technically correct, this tenth edition incorporates new material to address these advances in technology. A list of only a few of the recent developments in materials, equipment, and processes includes:

- Shrinkage-compensating cement;
- Increased use of supplementary cementitious materials (SCMs);
- Polymer-modified mixtures;
- Self-consolidating concretes;
- New and refined admixtures;
- Fiber-reinforced concrete;
- Epoxy resins;
- High-capacity and automated concrete production equipment;
- High-performance and high-strength concrete; and
- Epoxy-coated and stainless steel-clad reinforcement.

The need to cover new issues affecting inspection is the reason ACI Committee 311 continues to revise the ACI Manual of Concrete Inspection. In preparing this edition of the manual, as with previous editions, the committee's task was to interpret the policies set forth by other authorized bodies rather than to make policy on construction practices. The main

emphasis of the manual is on the technical aspects of inspection and construction. For further information about construction practices, readers are encouraged to refer to the ACI Manual of Concrete Practice.

Because the content of this manual is general and broad in nature, no part of the manual should be included by reference in contract documents. Applicable inspection requirements for each project should be determined and included in the specifications.

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ACI Committee Reports, Guides, Manuals, Standard Practices, and Commentaries are intended for guidance in planning, designing, executing, and inspecting construction. This document is intended for the use of individuals who are competent to evaluate the significance and limitations of its content and recommendations and who will accept responsibility for the application of the material it contains. The American Concrete Institute disclaims any and all responsibility for the stated principles. The Institute shall not be liable for any loss or damage arising therefrom.

Reference to this document shall not be made in contract documents. If items found in this document are desired by the Architect/Engineer to be a part of the contract documents, they shall be restated in mandatory language for incorporation by the Architect/Engineer.

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CHAPTER 1—INSPECTION AND THE INSPECTOR

1.1—Inspection processes

1.1.1 Why inspection is needed—The purpose of inspection is to verify that the requirements and intent of the contract design documents are faithfully accomplished. In concrete construction, inspection includes not only visual observations and field measurements, but also field and laboratory testing and the collection and evaluation of test data. In many instances, inspectors also act as or are assisted by the field technicians assigned to perform the testing.

One important responsibility of the concrete inspector is to assess the quality of the materials used in the concrete. It is difficult, and usually impossible, to produce specified concrete from nonconforming materials. Thus, the final materials entering the concrete mixture should be of specified quality.

An important factor in quality construction is good workmanship in all operations and processes (Fig. 1.1). Observing this aspect becomes an important responsibility of the inspector. Even when concrete is made using high-quality materials, proportioned correctly, and batched correctly, the resulting concrete structure can be unsatisfactory if construction workmanship is of poor quality.

Manual skills, technical knowledge, motivation, and pride all contribute to good workmanship. Most individuals involved with concrete design and construction take pride in their efforts and strive to attain superior quality (Fig. 1.2). Not all personnel, however, will receive the necessary training to do their jobs properly. The need to meet fast-track

construction schedules and stay within cost limits often places too much emphasis on production rates. If speed becomes a top priority, construction quality may not receive adequate attention if not properly executed. Ironically, cost may also suffer. Techniques that speed concrete placement can actually add to material costs or result in the need for expensive repairs.

Jacob Feld, a noted investigator of structural failures, listed examples in his book, *Lessons from Failures of Concrete Structures*, showing that a high percentage of failures of concrete structures that he had investigated were caused in significant part by poor construction—in other words, poor workmanship. He stated: “The one thing which these failures conclusively point to is that all good concrete construction should be subjected to rigid inspection... It is believed that only by this kind of inspection is it possible to guard against the failure of concrete structures” (Feld 1964).

For every monumental structural collapse, innumerable instances of small failures occur. This is particularly true when important concrete properties, such as durability and watertightness, do not conform to design requirements.

Superior concrete structures can be built at a reasonable cost if concrete producers and contractors are vigilant. As the late F. R. McMillan said in the foreword to his famous *Concrete Primer* (McMillan and Tuthill 1987): “Many who have been interested in the cause of better concrete have noted the difficulty of making any real progress until someone in authority has been convinced that good concrete can be had, that it should be had, and, having been so convinced, has sent out the word that it must be had.”

1.1.2 Purposes of inspection—The desire for quality has led to the use of inspection personnel to monitor and document quality of concrete construction. The responsibilities and duties of inspectors have broadened over the years. Today, several inspection teams may be used on one project to represent the interests of the various parties involved. Inspectors may be employed:

- By project owners to provide quality assurance for the work;
- By government agencies and large industries to assure the quality;
- By architects and engineers to verify and document compliance with project specifications and drawings;
- By contractors to provide quality-control inspection for projects under construction. This helps provide assurance to the contractor that the finished construction will meet all requirements of the contract documents and thus will be accepted by the owner;
- By producers of concrete materials and products who need assurance that finished products will meet the requirements of the contract documents. Examples include producers of cement, aggregate, ready mixed concrete, and precast products;
- By licensing and building-permit jurisdictions charged with enforcing building codes and other regulations. In this case, the inspector will be responsible only for assuring that the finished structure conforms to requirements of the codes or regulations; or

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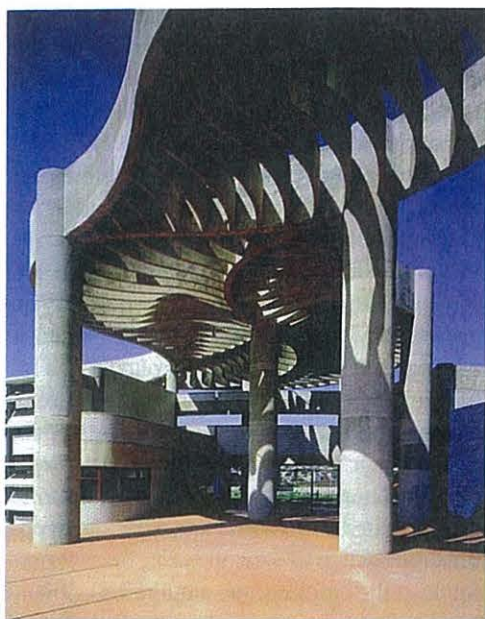


Fig. 1.1—Increasingly sophisticated building designs call for higher standards of materials and workmanship.

- By commercial laboratories designated to provide testing and inspection services.

Regardless of the function, inspection, including laboratory and field testing, may be performed by a team or, for very small projects, by just one person (Fig. 1.3).

1.1.3 Owner and contractor inspections

1.1.3.1 Owner inspection—Owner inspection provides assurance to owners that the requirements of the contract documents (drawings and specifications) are fulfilled. ACI 311.4R, Section 2.3, was prepared to guide architects, engineers, and owners in the development of effective inspection programs. It states:

For the protection of the public and the owner, the responsibility for planning and detailing owner inspection should be vested in the A/E as a continuing function of the design responsibility. The A/E should ensure that the program for owner inspection meets all requirements of design specifications and the local building code. The inspection responsibility may be discharged directly, may be conducted by owner personnel, or may be delegated to an independent inspection organization reporting to the A/E.

If the A/E is also responsible for construction, an independent inspection organization should be retained directly by the owner. When the owner provides the A/E service, the owner should also provide inspection or retain an independent inspection organization. Inspection requirements on projects supervised by a construction manager should also be detailed by the A/E and should be carried out by inspection personnel representing the owner.

The fee for owner inspection and testing should be a separate and distinct item and should be paid by the owner, or by the A/E acting on behalf of the owner, directly to the inspection organization. The owner or

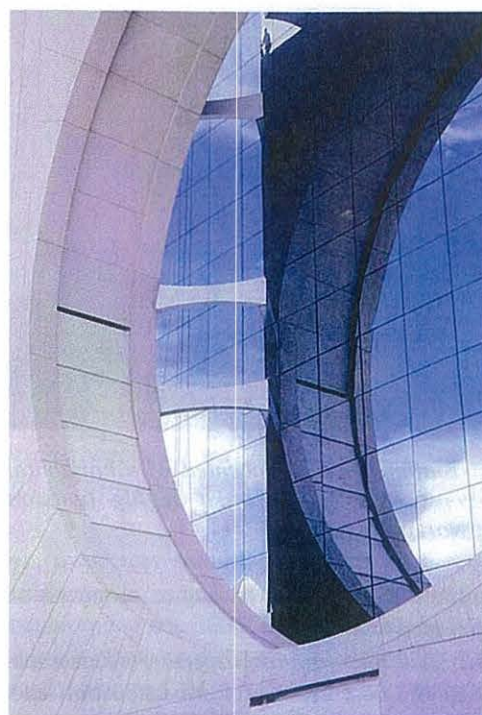


Fig. 1.2—Concrete's ability to be formed into any shape lets artistry and function go hand in hand.



Fig. 1.3—ACI-certified field technician performing a slump test.

A/E should avoid the undesirable practice of arranging payment through the contractor for inspection services intended for use by the owner as a basis of acceptance. Such a practice is not in the owner's interest, and may result in a conflict of interests. Impartial service is difficult under such circumstances, and the fees for inspection are eventually paid by the owner in any case.

Under a typical construction contract, inspectors representing the owner have no responsibility or authority to manage the contractor's workforce. The owner's inspection team is responsible for determining that materials, procedures, and end products conform to the requirements of the contract documents. Because inspectors are only responsible for evaluating the contractor's work for conformance, they may

Architect
Engineer



Fig. 1.4—Building design accented with horizontal joints; quality construction provides a highly functional and aesthetic workplace.

not burden the contractor by adding requirements not given in the contract documents.

1.1.3.2 Quality-control inspection—Personnel maintained or hired by the contractor perform inspection and testing services used to control quality during construction. In some construction contracts, particularly those with certain government agencies, the contractor is required to provide a specified amount of inspection and testing as part of a formal quality-control program. Even when not contractually required, many contractors maintain inspection and testing personnel, separate from the line of supervision, to monitor quality control. These inspectors report directly to the contractor's management. Sometimes inspection work is performed by the contractor's construction supervisors as an automatic part of construction operations.

By performing their own quality-control inspection, contractors can avoid later rejection of work that would be very costly to replace or correct. Inspection performed by or for the contractor, particularly when contractually required, should be more detailed than that performed for owner inspection, with greater attention being given to form alignment, positioning of embedments and reinforcing bars, and general placement practices. Even when contract documents require extensive quality-control inspection and testing by the contractor, however, owners should not reduce or eliminate their own inspection efforts. The owner should maintain formal oversight of the contractor to ensure that the quality-control program achieves its objectives (Fig. 1.4).

Manufacturers of concrete materials and products also use inspection and testing to maintain quality control and to ensure that contract requirements are met. These programs should parallel the contractor's quality-control efforts. Owners should independently audit and assess the effectiveness of these quality-control programs.

1.2—Inspector

1.2.1 Duties—Although the duties and emphasis of the different inspection teams involved on a project may vary and sometimes overlap, the basic approach is the same. The owner's inspectors generally emphasize inspection of

subgrade and contact surfaces, inspection of reinforcing steel, inspection of concrete materials as they are delivered, testing of the unhardened and hardened concrete, and inspection of the finished structure. The contractor's quality-control inspectors usually emphasize inspection of materials production, setting of formwork, and placement and curing of concrete. The duties that inspectors are most frequently asked to perform include:

- Identifying and examining materials before acceptance. This includes verifying quality based on certifications and test results provided by producers and suppliers as well as sampling and testing materials delivered to the job site;
- Monitoring batching and mixture proportioning (including adjustments) and testing concrete for consistency, air content, temperature, and density;
- Examining foundations, subgrade, forms, reinforcing steel, embedded items, and other work in preparation for concreting;
- Inspecting the mixing, conveying, placing, consolidating, finishing, curing, and protection of concrete;
- Preparing required concrete specimens for laboratory tests, including monitoring for adequate field curing and proper protection of the specimens;
- Observing the equipment, working conditions, weather, and other items affecting the concrete or other related parts of the structures;
- Observing methods used for curing and protection of concrete, especially during periods of hot or cold weather;
- Evaluating test results for conformance to the specifications;
- Reporting nonconforming conditions and materials in a prompt manner;
- Verifying that unacceptable items and procedures are corrected; and
- Preparing records and reports.

To be effective, inspectors need support from management. They need to also be observant and able to evaluate the relative importance of various work items to give priority to more important matters. Above all, inspectors should be completely familiar with all acceptance criteria in the contract documents, and they should promptly document and report any nonconformance to these criteria to contractors and to their own supervisors.

1.2.2 Education and certification—Inspectors and technicians can receive related technical training at colleges, trade schools, and similar educational institutions. ACI Certification Program Sponsoring Groups may also provide specific training before certification exam sessions. Certification of inspectors and technicians is becoming the standard and, in many cases, is mandatory. It provides third-party assurance that the inspector or technician has at least the basic skills and knowledge to perform the job. Some states and local jurisdictions require certification, but in most cases, the requirement is placed in industry standards and then referenced in the building codes. Industry standards containing a certification requirement or recommendation for certification include ACI 301, 311.4R, 311.5, 318, and 349 and ASTM C31/C31M, C39/C39M, C42/C42M, C78, C94/C94M, C192/C192M, C685/C685M, C1077, and E329.

ACI provides certification programs for:

- Concrete Field Testing Technician—Grade I
- CSA-Based Concrete Field Testing Technician—Grade I
- Concrete Flatwork Finisher & Technician
- Specialty Commercial/Industrial Flatwork Finisher & Technician
- Concrete Strength Testing Technician
- Concrete Laboratory Testing Technician—Level 1 & 2
- Aggregate Base Testing Technician
- Aggregate Testing Technician—Level 1 & 2
- Concrete Construction Special Inspector & Associate
- CSA-Based Concrete Construction Special Inspector
- Concrete Transportation Construction Inspector
- Associate Concrete Transportation Construction Inspector
- Tilt-Up Supervisor & Technician
- Shotcrete Nozzleman (Dry Mix Process)
- Shotcrete Nozzleman (Wet Mix Process)

ACI continues to develop new certification programs in response to expanding industry needs.

In addition to having a technical understanding of the principles involved in the assigned construction, inspectors must have practical experience. They should know how and why the work is to be done in a particular way. Inexperienced but technically trained inspectors should undergo on-the-job training under the supervision of more experienced individuals before working alone. Employers also should encourage inspectors to keep their skills up to date by continuing their technical training. Employers can assist in this effort by providing periodic training courses.

1.2.3 Authority—The quality-assurance/quality-control duties and responsibilities of the owner, engineer, contractor, and supplier should be clearly detailed in the contract documents and thoroughly understood by all parties. These duties and responsibilities should be reviewed at a preconstruction or concrete preplacement meeting. Clearly defining each party's authority, responsibilities, and lines of communication before concrete is placed will have a positive impact on the concrete placement.

At the start of each job, the inspector's supervisor should clearly explain the authority that the inspector has and the actions to be taken in various situations that may be encountered. These duties and responsibilities should be provided to the inspector in writing. The inspector and his firm should contractually be a legal agent of the owner or contractor to have the authority to stop work. An inspector may be authorized to:

- Stop work until preliminary conditions (such as preparation of forms, contact surfaces, subgrades, construction joints, and the placing of reinforcement) are satisfactory and accepted and until inspection personnel are available to observe concreting operations; and
- Stop work if the materials, equipment, or workmanship do not conform to the contract documents.

In both cases, the inspector is usually authorized to take direct action in concert with the contractor's supervisors. Inspectors should stop work only as a last resort, when it is evident that nonconforming concrete will result from continuing operations, and only if authority to do so has been established. In cases where only construction personnel may



Fig. 1.5—A preconstruction meeting between certified inspector and contractor will help prevent problems during concreting operations.

stop work, inspectors should promptly advise responsible construction supervisors and owners or owner representatives of all nonconforming conditions as soon as they are identified. Actions taken by the contractor in response to this notification should be documented. Matters of general policy or major points not covered or clearly conveyed in the specifications should be discussed with the inspector's supervisor and then with the engineer, usually via request for information (RFI) communication (Fig. 1.5).

***1.2.4 Relations with contractors, supervisors, and workers**—Inspectors representing the owner should cooperate with the contractor consistent with the owner's interests to help reduce construction costs and improve schedules. Inspections should be made promptly when requested, and conditions that will lead to unsatisfactory work should be pointed out to the contractor immediately to avoid the waste of materials, time, and labor. Inspectors should be on the job during reinforcement placement before concreting and whenever concrete is being placed, finished, cured, or repaired.

The inspector should not delay the contractor unnecessarily nor interfere with the contractor's methods unless it is evident that the work will be unacceptable. Demands should never be made of the contractor that are not in accordance with the contract documents. If the contract documents permit a choice of methods, the inspector may suggest, but not demand, which of the methods should be used. At no time is the inspector permitted to direct the work of the contractor.

Inspectors should maintain an impersonal, agreeable, and helpful attitude toward contractors and their employees. They should never accept personal favors nor criticize the contractor's organization or workers. By dealing fairly and recognizing and commending good work, inspectors usually can gain the respect and cooperation of the contractor's supervisors and workers.

Establishing a clear line of communication also is important. Formal communications should be conducted only with the authorized representatives of the contractor, preferably in the form of advisements relating to the quality of the work in progress. Matters involving a potential change in project costs, completion time, or other important factors should be referred to the owner or owner's authorized representative.

The inspector usually deals directly with the subcontractors' supervisors. If subcontractor response to nonconforming conditions is unsatisfactory, the matters requiring correction should be immediately referred to the general contractor who is legally responsible for the work.

The same basic approach to inspection and the inspector's relations with work crews applies to inspectors employed by the contractor. In many instances, however, the working relationships with supervisors and work crews are affected by the fact that everyone is employed by the contractor. Inspectors should receive instructions from the contractor's management detailing the guidelines for these relationships.

1.2.5 Safety—Inspectors should follow all rules for job-site safety established by the owner or contractor. Job-site safety is everyone's responsibility. Any time inspectors observe unsafe conditions or practices, they should immediately advise the responsible contractor, and then document the safety advisement as well as any actions taken by the contractor in response. Inspectors should contact their supervisors for additional guidance on reporting safety issues.

1.3—Importance of clear specifications

This manual describes work practices and procedures necessary to ensure satisfactory concrete construction. It cannot be emphasized too strongly, however, that inspectors are governed strictly by the requirements of the contract documents defining the work. This manual and other references should be used only as information sources and to provide additional guidance on items not covered by the contract documents. In some situations, inspectors will be governed by performance criteria supplied by management or provisions of applicable building codes or regulations that state criteria in terms of the desired results but without stating the methods for achieving the desired results. Likewise, inspectors may be governed by prescriptive criteria that define the exact materials or processes to be executed.

Because the contract documents are the main criteria governing the decisions and actions of an inspector, clear specifications and drawings are essential. Much of the friction in construction arises from differing interpretations of vague or incomplete contract documents. The contract documents should be specific and carefully written. Although the inspector has no control over the contract documents for the project under construction, job-site inspectors should provide feedback to the designer by suggesting changes to their supervisors that may be appropriate to established requirements.

A common and erroneous assumption is that specified tolerances for lines, grades, dimensions, and surface finish apply to the setting of forms, screeds, and grade strips. This, however, is not the case. Tolerances apply only to the completed concrete. Forms, screeds, and grade strips should be set at the exact position indicated on the drawings, insofar as possible, so the finished concrete will conform to the required measurements.

Designers usually determine the governing measurements of line, grade, and dimensions. When required by their duties and responsibilities, inspectors check the alignment of forms and screeds and the positioning of reinforcement and

REFERENCE LIBRARY

In addition to standards and publications referenced in the contract documents, concrete inspectors should keep a reference library of the latest editions of other useful resources including, at a minimum:

- *Field Reference Manual: Standard Specifications for Structural Concrete for Building with Selected ACI and ASTM References* (ACI SP-15) (American Concrete Institute 1999) (compilation publication containing ACI 301 and related ACI and ASTM documents)
- *Placing Reinforcing Bars* (Concrete Reinforcing Steel Institute 1997a)
- "Building Code Requirements for Structural Concrete (ACI 318)"
- *Concrete Manual* (U.S. Bureau of Reclamation 1988)
- *Design and Control of Concrete Mixtures* (Portland Cement Association 2002)
- "Guide for Concrete Inspection (ACI 311.4R)" and "Batch Plant Inspection and Field Testing of Ready-Mixed Concrete (ACI 311.5)" (both of which are reprinted in Appendixes A and B, respectively, of this manual).
- Chapter 21 lists additional references and standards that may be helpful for specific jobs.
- Chapter 22 provides titles for industry references.

embedded items. They also measure lengths, volumes, and weights as required to ensure that the quantities of materials and finished work meet all requirements. Recognizing that even the most careful measurement can never be exact, inspectors should exercise judgment as to the tolerance to be permitted in specific cases if the contract documents do not state limiting values or permissible tolerances. Because it is impossible to align forms and reinforcing steel to the nearest hundredth of an inch, permissible deviation from exact value should be governed by the effect the deviation would have on the performance or appearance of the structure. For example, a displacement of reinforcing bars of 1/2 in. (12 mm) might be of no consequence in a foundation, but it could seriously weaken a thin slab or impair protection of the bars from corrosion. Standards for tolerances not covered by the contract documents should be established early in the construction period. The most widely referenced source for concrete construction tolerances is ACI 117.

CHAPTER 2—STATISTICAL CONCEPTS FOR QUALITY ASSURANCE

2.1—Quality-control and quality-acceptance inspections

Quality assurance refers to all of the programs and functions involved in obtaining quality concrete that will provide satisfactory service in the finished concrete structure. It includes design, production, sampling, testing, and decision criteria. The two types of inspection employed in quality assurance are quality-control inspection, primarily a function of the contractor, and owner-acceptance inspection, primarily a function of the owner or owner's representative.

Legal requirements for documentary evidence of satisfactory compliance have led to greater use of specifications based on statistical concepts. Using statistics permits decisions about quality to be made with an established degree of confidence. This chapter gives background on the underlying statistical

concepts for use in concrete construction quality-control and owner-acceptance programs, along with examples of the procedures inspectors are likely to encounter. More detailed information on the use of statistics in concrete can be found in the references noted in this chapter.

2.2—Traditional quality assurance

Many specifications that have traditionally been used for concrete are prescriptive rather than performance-based specifications. Prescriptive specifications define the exact materials and detail the operations of the contractor and the equipment to be used to produce the concrete. Such specifications were developed because adequate definitions and test methods for evaluating the performance of the end product were lacking. The values used to define required performance usually were subjective, based on experience and judgment, rather than rational concepts.

Though prescriptive specifications—combined with the skills of experienced designers and the cooperation of experienced contractors—have produced good concrete structures, these structures have not always met the desired quality. Under these specifications, a random, supposedly representative, sample is taken. This sample is tested and the result is compared with the specified value of the particular characteristic. If the test result is within the specified tolerances, the material passes the criteria and is accepted. If the material fails to pass the criteria, the design engineer applies engineering judgment and a decision is made as to whether the material substantially complies and should be accepted, whether the material fails and should be rejected, or whether the material should be retested. Because “substantial compliance” is subjective, it varies from person to person and job to job, creating confusion and disputes. Research has shown that as much as 30% of some construction controlled by traditional methods has been outside the stated limits when closely examined by statistical methods using random sampling, even though it was considered completely acceptable under the control practices used (California Department of Transportation 1968).

When a failing test is encountered, retesting without obtaining a new sample is not appropriate (unless the original test has been performed improperly, in which case the entire test should be voided). Even if the results of the two tests (original test and retest) are averaged, there is a built-in bias because the second test is taken only if the first test fails. For example, consider a material that has a 50% chance of passing the specification limit and a 50% chance of failing. For each test on a certain lot, there is always a 50% chance the result will pass and a 50% chance it will fail. When a test is made and the result fails, however, a retest of the same lot also has a 50% chance of passing and a 50% chance of failing. Because a second test is made only if the original test fails, however, this second test has biased the original test so that the overall probability actually becomes 75% pass, 25% fail. This is illustrated in Fig. 2.1.

2.3—Statistical concepts and procedures

Contract documents written using statistical concepts express quality requirements in terms of target values for

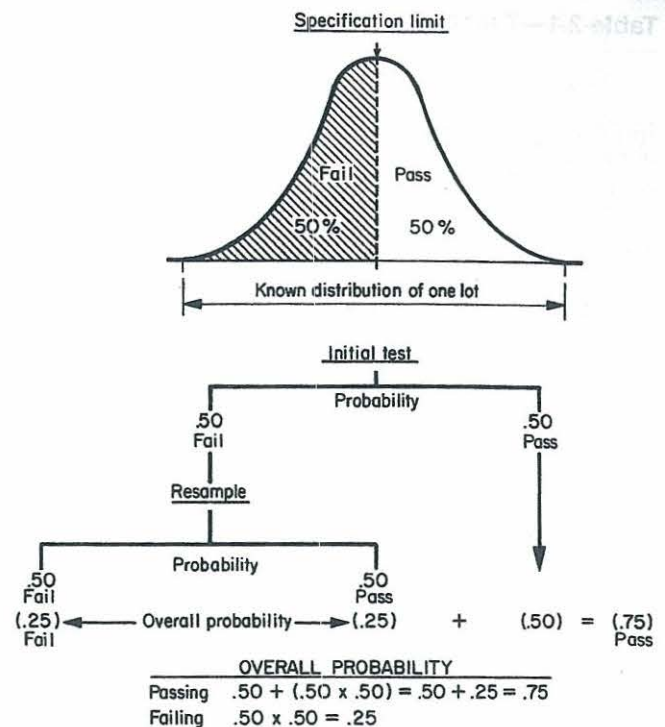


Fig. 2.1—Probability of acceptance when resampling.

contractors and express compliance requirements in terms of plus or minus tolerances from established target values. Tolerances for the target value, prescribed by design needs, can be based on statistical analyses of the variations in materials, processes, sampling, and testing existing in traditional construction practices. Tolerances derived in this manner can be both realistic and enforceable. They take into account all of the normal causes of variation and allow for the expected distribution of test results around the mean. Provisions can be made both for control to the stated level and for control of the variation from this level.

The use of statistical concepts has proved to be very effective and efficient where properly applied. In addition to indicating acceptable and unacceptable materials in construction, statistical methods can indicate gray areas where test results show that a material is not completely in compliance with requirements but can be accepted if permitted by the contract documents. Public agencies, particularly state highway departments, often find statistical concepts valuable for use on projects involving high production rates and large volumes of concrete, such as highway paving, large dams, and airfield paving.

Statistical procedures are based on the laws of probability. For these laws to function properly, the data must be collected by random sampling. A true random sample is one in which all parts of the whole have an equal chance of being chosen (ASTM D3665). Without true random samples, statistical procedures give false results.

It is possible to adapt the use of random numbers to the laboratory, to the field, and to the factory. Random selection is not merely haphazard. Numbers can be selected from a standard table of random numbers or by use of mechanical

Table 2.1—Types of sampling

	Protects against known defects	Protects against unknown defects	Protects against cycles and patterns	Low inherent risk	Low risk in unknown situations	Simple organization	High reliability	Does not require knowledgeable sampler
JUDGMENT SAMPLING (based solely on judgment of sampler)	✓	X	?	?	X	✓	?	X
QUOTA SAMPLING (making judgment distribution of sample by time of day, location, etc., according to distribution of facts)	✓	X	✓	?	X	X	?	X
SYSTEMATIC SAMPLING (selecting successive observations at constant intervals in a sequence)	✓	✓	X	✓	?	✓	✓	✓
STRATIFIED SAMPLING (selecting each of two or more parts independently from a corresponding part)	✓	✓	✓	✓	?	?	✓	✓
RANDOM SAMPLING (selecting sample in such a manner that each individual has the same chance of being chosen)	✓	✓	✓	✓	✓	X	✓	✓

Note: ✓ = satisfies column heading requirement; X = does not satisfy column heading requirement; and ? = may or may not conform to heading, according to situation.

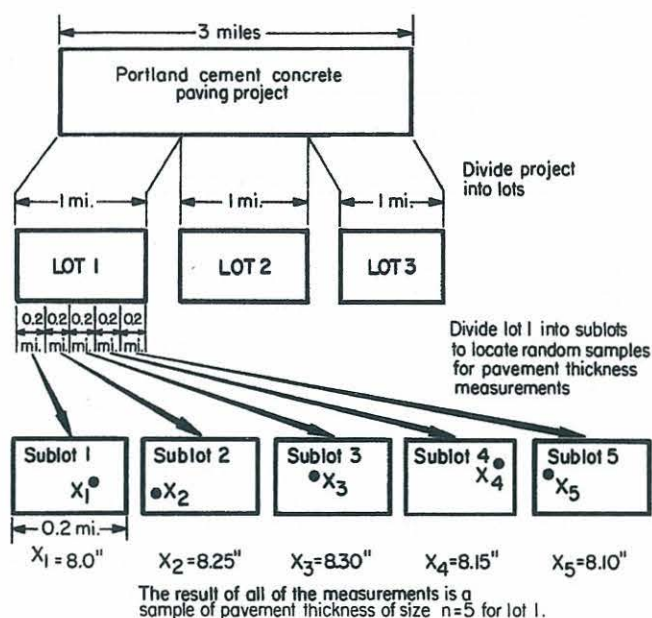


Fig. 2.2—Designation of lots and sublots.

randomizing devices (such as dice or a spinning wheel). They also can be generated by computer. No device, however, is acceptable as random unless it passes certain statistical tests.

ASTM E105 discusses the preparation of sampling plans. Sampling programs commonly used are described in the following paragraphs. Table 2.1 shows the advantages and disadvantages of these programs (McMillan and Tuthill 1987). Normally, no one sampling type is used alone.

1. *Judgment sampling* is based solely on the judgment of the sampler, with no other restrictions. The sampler decides when and where a sample should be taken;

2. *Quota sampling* is a type of judgment sampling based on time of day, geographic area, or some other known distribution of facts;

3. *Systematic sampling* involves selecting successive observations in a sequence, such as of time or area, at constant intervals;

4. *Stratified sampling* involves dividing a given quantity of material into independent parts, each to be sampled separately. Stratified sampling is inherent in any acceptance sampling based on use of sublots; and

5. *Random sampling* involves selecting a sample in such a manner that each increment comprising the lot has the same chance of being chosen for the sample (ASTM D3665; California Department of Transportation 1968).

Examples of sampling by random numbers, using the typical table of random numbers, are given in Appendix 1 (Pennsylvania State University 1974).

In addition to randomness, the concept of lots also is essential to the proper application of statistics to quality-assurance sampling of construction. A lot is a prescribed and defined quantity of material (such as volume, area, tonnage, time of production, or units) produced from the same process for the same purpose. This is the quantity offered for acceptance as a unit, and all sampling and testing requirements are defined in relation to and applied to that quantity. Only by establishing the lot size can the proper sampling location and frequency be selected to determine the quantity of material within the specified limits. Under the concept of lot-by-lot sampling and testing, the process of concrete construction may be thought of as the production of a succession of lots, each presented to the designer for acceptance or rejection. This is illustrated for a concrete pavement in Fig. 2.2. To implement the acceptance plan, each lot is separated into subdivisions of equal sizes, called sublots. Sampling locations are randomized within the boundaries of each sublot. Stratifying or separating the lot into smaller components ensures that there will be no excessive periods without any sampling and testing. It also allows sampling and testing crews to space out their work.

A sample, used in a statistical sense, is that portion of the lot taken to represent the whole. The term is not to be confused with the individual test portions or sample increments that make up the sample. Figure 2.3 illustrates the relationships between test portions, sample increments, and the sample. (Note: In discussions of sampling and testing of materials or

concrete in other chapters, the term “sample” generally refers to each individual portion of material taken for testing.)

Normally, it is desirable for each sample to be made up of four or five sample increments (or four or five sublots per lot). The average of the test values of sublots in a lot, however, will have a range smaller than the range of the values of the individual test portions in the entire lot.

2.4—Basic statistical concepts

2.4.1 Definitions—The following symbols and terms, primarily from ACI 214R, are commonly used in statistical quality-assurance programs for concrete.

n = number of tests in a sample. The total number of test results or values under consideration.

X = an individual test result. Indicates separate test results (may be written X_1, X_2, X_3 , and so on).

$\bar{X}$ = sample average (called “bar X ”).

The arithmetic mean of all test results; the sum of all test result values divided by n , the number of tests

$$\bar{X} = \frac{X_1 + X_2 + X_3 + \dots + X_n}{n}$$

s = sample standard deviation.

The square root of the average obtained by dividing the sum of the squares of the numerical differences of each test result from the sample average by one less than the number of tests

$$s = \sqrt{\frac{(X_1 - \bar{X})^2 + (X_2 - \bar{X})^2 + \dots + (X_n - \bar{X})^2}{n - 1}}$$

A simpler and more adaptable form for desk calculators is

$$s = \sqrt{\frac{1}{n-1} \left(\sum (X_i^2) - \frac{(\sum X_i)^2}{n} \right)}$$

where

n = number of values;

$\sum X_i$ = sum of n values; and

$\sum (X_i^2)$ = sum of squares of n values.

σ = population standard deviation.

The population consists of all possible data. The sample is a portion of the population.

V = coefficient of variation.

The sample standard deviation expressed as a percentage of the average

$$V = \frac{100s}{\bar{X}}$$

R = sample range.

The statistic found by subtracting the lowest value in a data set from the highest value in that set

$$R = X_{max} - X_{min}$$

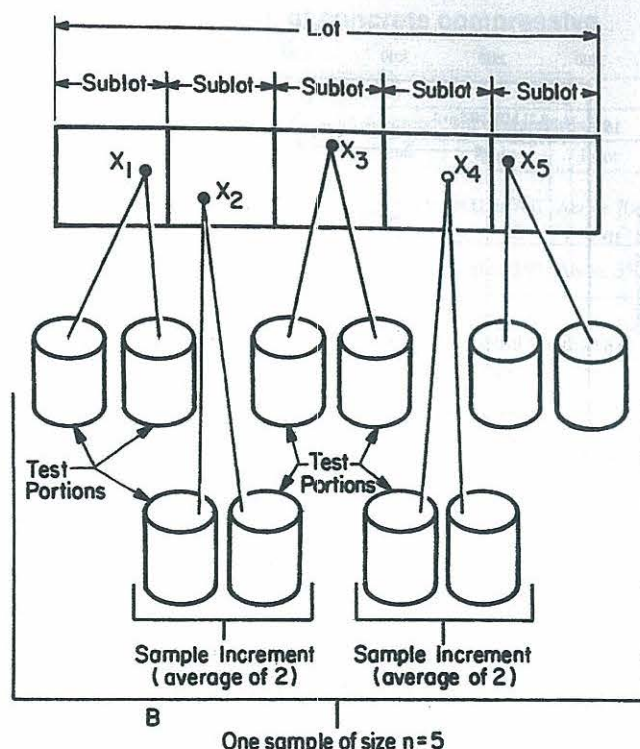


Fig. 2.3—Definition of test portion, same increment sample.

f'_c = specified compressive strength of concrete. Usually 28-day strength, but can be specified at any age.

f'_{cr} = required average compressive strength of concrete. Ensures that no more than the permissible proportion of test results will fall below the specified strength; used as the basis for selection of concrete proportions.

2.4.2 Normal distribution curves—Research has shown that test results of construction materials and operations form a definite pattern grouping around a central value, called the mean. The measurements generally group around the average in a symmetrical pattern, thereby allowing the use of statistics based on the familiar bell-shaped normal distribution curve. Although slight variations from the symmetrical curve can occur, especially when the number of test results is small, the error in assuming normal distribution is usually not large. The assumption of a normal distribution permits the use of established relationships of mean (average) and standard deviation to establish realistic tolerances in the contract documents for selected sample sizes. Determining these tolerances requires the use of statistical analysis and engineering judgment.

A typical distribution of results of compressive strength tests made on a specific concrete mixture (Fig. 2.4) is given in ACI 214R. Superimposed on these plotted results is a normal bell-shaped distribution curve computed from test results. This particular set of test results and distribution curve would be considered a reasonably good fit. All statistical control procedures for concrete construction, including control charts, assume that the distribution of test results

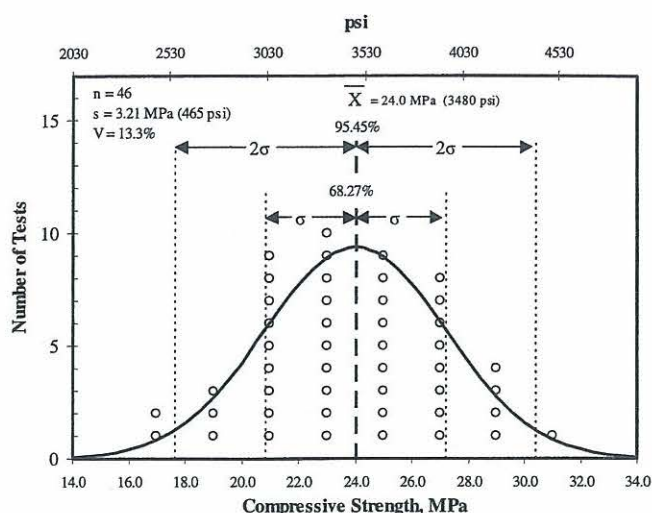


Fig. 2.4—Frequency distribution of strength data and corresponding assumed normal distribution (σ is the estimate of the population standard deviation).

Table 2.2—Required average compressive strength when data are available to establish standard deviation

Specified compressive strength f'_c , psi	Required average compressive strength f'_{cr} , psi
$f'_c \leq 5000$	Use the larger value computed from the following equations: $f'_{cr} = f'_c + 1.34s$ $f'_{cr} = f'_c + 2.33s - 500$
Over 5000	Use the larger value computed from the following equations: $f'_{cr} = f'_c + 1.34s$ $f'_{cr} = 0.90f'_c + 2.33s$

Note: 1 psi = 0.0069 MPa.

(regardless of concrete or material quality) will approximate such a normal curve. Appendix 2 gives additional examples of normal distribution curves.

2.4.3 Applying normal distribution curves to concrete compressive strength—In concrete construction, too much emphasis is often placed on the results of individual cylinder tests. Low tests are inevitable, but an occasional low test should not be a major concern if the result is not too low or the low test does not represent a portion or member of the structure deemed critical by the designer. On the other hand, a few test results above f'_c are not necessarily proof of adequate-quality concrete throughout the entire structure; it is quite possible that lower-strength concrete was placed somewhere in the structure but was not included in the tests.

What is of concern is that the number of standard cylinder test results falling below f'_c is not more than allowed by the contract documents. Typically, contract documents require that the average of all sets of three consecutive test results equals or exceeds the specified strength. If the test results do not meet this criterion, changes to the mixture proportions or improvements in quality control should be made. Furthermore, if an individual strength test result falls below f'_c by more than 500 psi (3.5 MPa) when f'_c is 5000 psi (35 MPa) or less, or by more than $0.10f'_c$ when f'_c is more than 5000 psi

(35 MPa), then the in-place concrete is in question, and appropriate actions should be taken. Refer to the section on evaluation and acceptance of concrete in ACI 318 for further information.

Tests of standard concrete control specimens during construction provide the basis for evaluating the potential strength of the concrete delivered to the job site. Placing, consolidation, and curing techniques will affect the quality of the hardened concrete in the structure. The quality of concrete can best be evaluated by analysis of at least 30 standard tests from a given mixture, as described in ACI 214R.

2.5—Statistical tools

2.5.1 Frequency distributions—Results of concrete compressive strength tests are plotted to form a histogram, or frequency distribution, for which the normal distribution curve can be computed. More importantly, the results are tabulated, and the average $\bar{X}$ and standard deviation s are computed to provide background data to use in selecting a required average strength f'_{cr} for future concrete construction. The required average strength is used to establish proportions that can provide actual concrete strength test results that will meet the specified concrete strength f'_c despite of production variations. This strength is statistically computed based on the $\bar{X}$ and s of the past results. Appendix 3 shows examples of how to compute s and f'_{cr} for given data.

Normally, the concrete inspector will not be concerned with plotting frequency distribution curves for concrete strength tests. The concrete producer or contractor typically determines the f'_{cr} and proportions the mixture to meet this requirement. An average strength higher than the specified strength is needed to ensure that the in-place concrete will be accepted when results of concrete control tests on the upcoming job are compared with the criteria of the contract documents. If the concrete inspector is required to check the f'_{cr} value computed by the contractor or concrete producer, the inspector should use data, particularly the standard deviation s , computed from previous testing of concrete produced by the same concrete plant or contractor.

When ACI 318 is referenced, a concrete production facility that has a previous standard deviation based on at least 30 consecutive strength tests* representing materials and conditions similar to those expected in the new work can be used to proportion for a required f'_{cr} representing the larger of the values computed as shown in Table 2.2, from ACI 318.

All of these required levels of overdesign are based on statistical assumption so that the concrete produced for the new construction can be expected to meet specification requirements when standard cylinders are tested during construction. These levels of overdesign are also based on the assumption that specification requirements are the same as those contained in ACI 318, which require that the average of any three consecutive strength tests equals or exceeds the required f'_c and that no individual strength test (average of two

* Allowance for using as few as 15 consecutive strength results is made by using factors that increase the sample standard deviation. These factors are: 1.16 for 15 tests, 1.08 for 20 tests, 1.03 for 25 tests, and 1.00 for 30 or more tests, as shown in Table 5.3.1.2 of ACI 318. The factors can be interpolated for an intermediate number of tests.

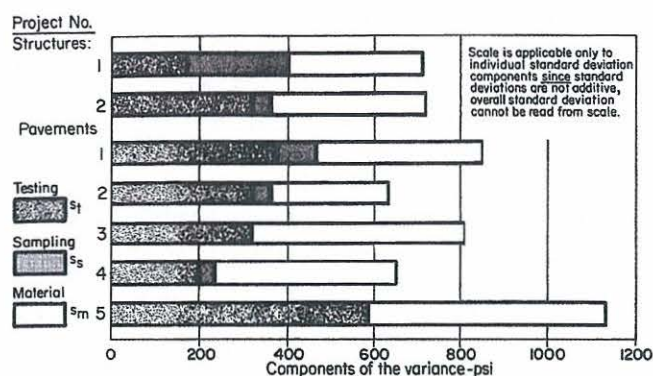


Fig. 2.5—Portland-cement concrete components of standard deviation, compressive strength.

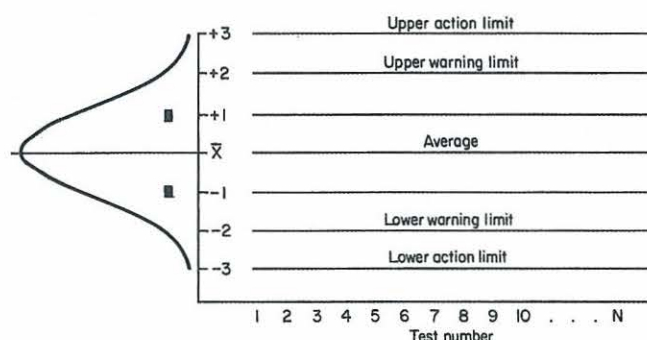


Fig. 2.6—Statistical control chart.

cylinders) falls below the required f'_c by more than 500 psi (3.5 MPa) when f'_c is 5000 psi (35 MPa) or less, or by more than $0.10f'_c$ when f'_c is more than 5000 psi (35 MPa).

The standard deviation of results for compressive strength or any other test is made up of components that are standard deviations of several sub-items. The main components are deviations due to sampling errors, testing errors, and actual variations in the material itself. Figure 2.5 shows the relationships among these sub-items on several different concrete projects. Note that these items are not additive; instead, the relationship consists of

$$s_o^2 = s_t^2 + s_m^2$$

where s_o is the overall standard deviation, s_t is the standard deviation of testing, and s_m is the standard deviation of the actual material properties.

Tables 2.3 and 2.4, from ACI 214R, show typical values for standard deviation of concrete strength tests for various standards of control and for type of testing.

2.5.2 Control charts—Although frequency distributions are used primarily to establish a proposed average compressive strength of concrete before work begins, control charts are the primary statistical tools used for evaluating results of tests on the concrete and concrete materials during construction.

Control charts are basically horizontal line charts. The horizontal lines for an acceptance control chart generally consist of a central line at the specified average, an upper line at the specified upper acceptance limit, and a lower line at

Table 2.3—Standards of concrete compressive strength, $f'_c \leq 5000$ psi

Class of operation	Overall variation				
	Standard deviation for different control standards, psi				
	Excellent	Very good	Good	Fair	Poor
General construction testing	Below 400	400 to 500	500 to 600	600 to 700	Above 700
Laboratory trial batches	Below 200	200 to 250	250 to 300	300 to 350	Above 350

Class of operation	Within-test variation				
	Coefficient of variation for different control standards, %				
	Excellent	Very good	Good	Fair	Poor
Field control testing	Below 3.0	3.0 to 4.0	4.0 to 5.0	5.0 to 6.0	Above 6.0
Laboratory trial batches	Below 2.0	2.0 to 3.0	3.0 to 4.0	4.0 to 5.0	Above 5.0

Note: 1 psi = 0.0069 MPa.

Table 2.4—Standards of concrete compressive strength, $f'_c > 5000$ psi

Class of operation	Overall variation				
	Coefficient of variation for different control standards, %				
	Excellent	Very good	Good	Fair	Poor
General construction testing	Below 7.0	7.0 to 9.0	9.0 to 11.0	11.0 to 14.0	Above 14.0
Laboratory trial batches	Below 3.5	3.5 to 4.5	4.5 to 5.5	5.5 to 7.0	Above 7.0

Class of operation	Within-test variation				
	Coefficient of variation for different control standards, %				
	Excellent	Very good	Good	Fair	Poor
Field control testing	Below 3.0	3.0 to 4.0	4.0 to 5.0	5.0 to 6.0	Above 6.0
Laboratory trial batches	Below 2.0	2.0 to 3.0	3.0 to 4.0	4.0 to 5.0	Above 5.0

Note: 1 psi = 0.0069 MPa.

the lower acceptance limit (if both apply). A true control chart (one used for actual process control) will usually have the center average line plus two lower and two upper lines. The upper and lower lines closest to the average are called warning limits, and the next lines are called action limits. These warning and action limit lines are placed at some multiple of s , the sample standard deviation, above and below the average value.

Figure 2.6 shows a blank sample of a typical actual process control chart (Pennsylvania State University 1974). Note how the control lines are related to a standard distribution curve (shown lying on its side to the left).

Generally, procedures for using actual process-control charts require that when a point (test result) falls on or outside either warning line, the producer should examine its operation to determine what has caused this variation and attempt to correct it. When a point falls on or outside the action line, the producer should stop its operation and make the adjustments necessary to bring the operation under control. Control charts can only indicate that a problem exists, not where it is located.

A horizontal-line control chart can be used to plot both single test results and running averages of a specified number of consecutive test results. Figure 2.7 shows typical charts of this type (Pennsylvania State University 1974).

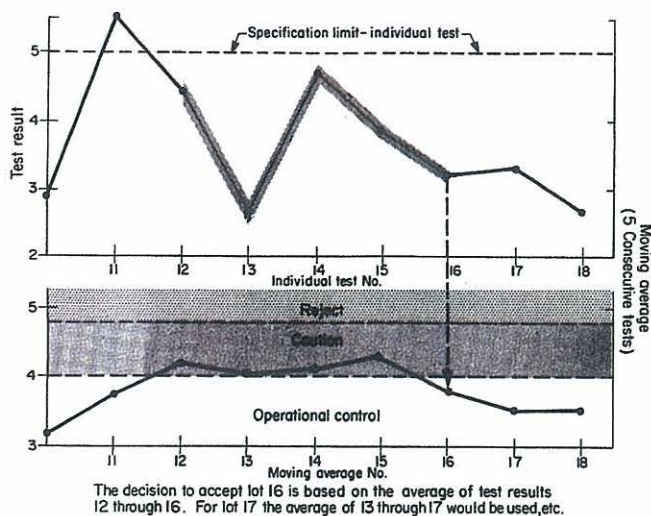


Fig. 2.7—Typical horizontal line control charts.

In addition to control charts for single test results and averages, it is common to use them to record the variability, or range, of test results, either from a lot or over some specified period. These, too, are horizontal line charts, but they normally have only lower and upper control limits.

Figure 2.8 (ACI 214R) shows typical control charts for results of concrete compressive strength tests. Control charts also are commonly required for results of air-content tests. A typical example is shown in Fig. 2.9 (Pennsylvania State University 1974). Control charts are sometimes maintained for results of slump tests, as shown in Fig. 2.10 (California Department of Transportation 1968).

Typically, the only concrete material quality for which control charts are maintained is aggregate gradation, and usually for only a few of the sieve sizes. Where aggregate quality is particularly important, however, charts may be maintained on results of certain quality tests, although statistically, this may be an improper application. Control charts for aggregate gradation and quality are also shown in Fig. 2.10. Appendix 4 shows typical control charts for aggregate gradation (for individual tests and range), moving averages for sand-equivalent test results, and moving averages for grading analysis.

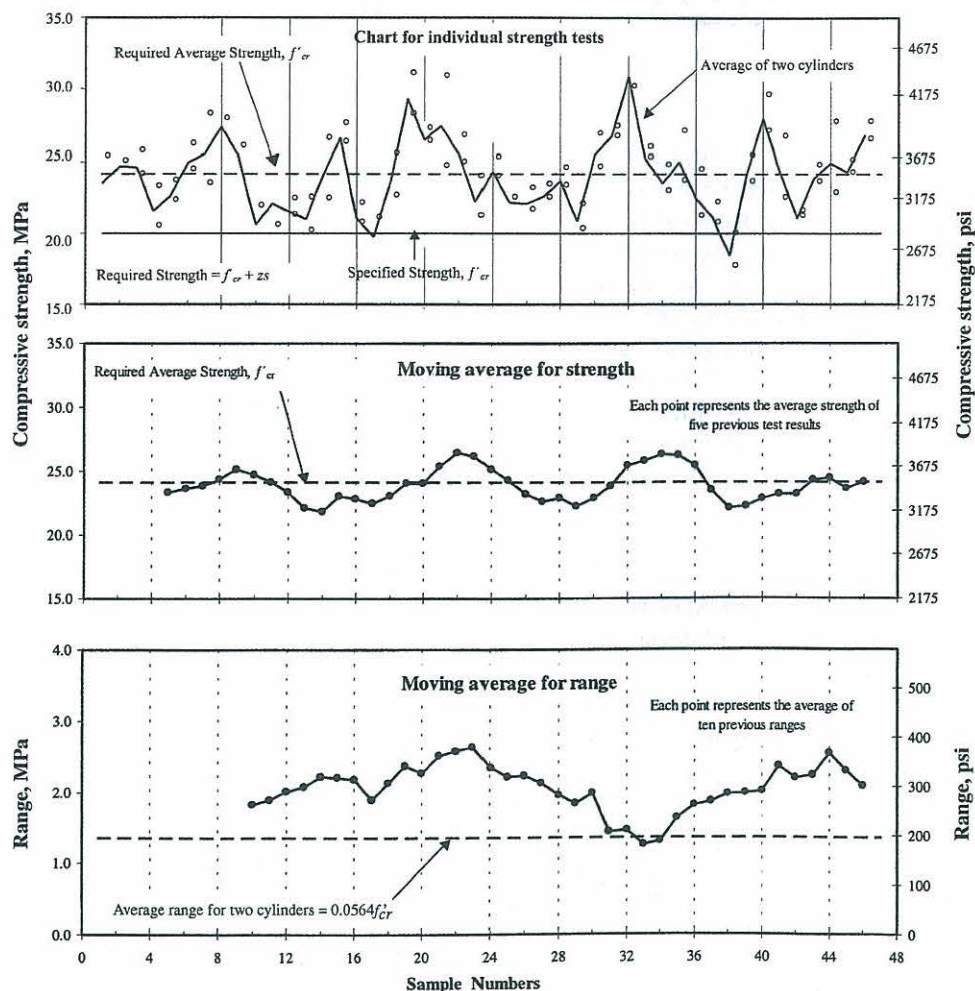


Fig. 2.8—Three simplified quality-control charts: individual strength tests, moving average of five strength tests, and range of two cylinders in each test and moving average for range.

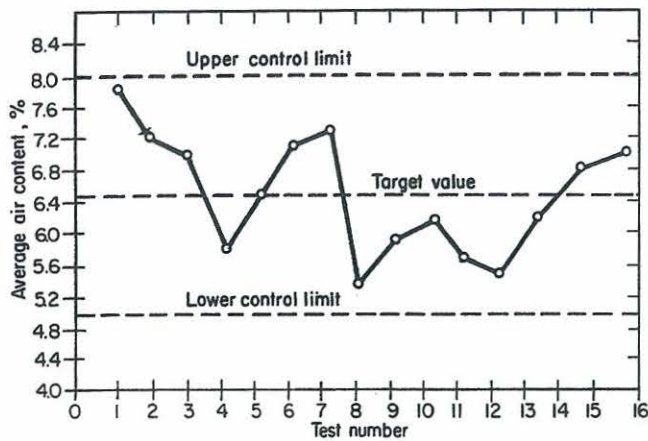


Fig. 2.9—"Bar X" control chart for air content.

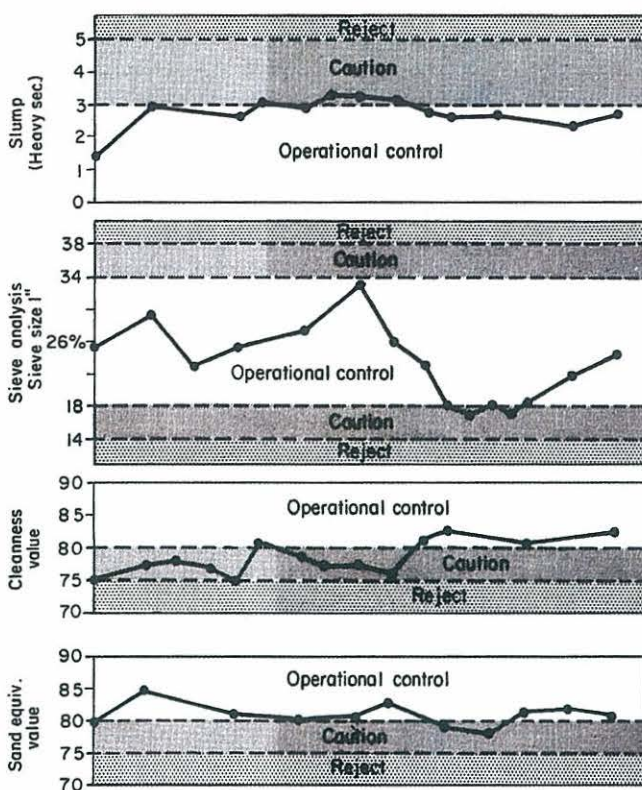


Fig. 2.10—Concrete control chart.

APPENDIX 1—SAMPLING BY RANDOM NUMBERS

A1.1—Example 1: Sampling by time sequence

The task is to sample materials, such as aggregate, concrete, or precast products, at the place of manufacture. Select the sample increments by means of a stratified random sampling plan to distribute the sampling over a half or full day, whichever is more applicable.

The contract documents will define the lot size and the number of sublots and sample increments per lot. For this example, assume the contract documents state that the lot size is a day's production, that five sublots are required from each lot, and that one sample increment per subplot must be obtained. The plant operates for 9 hours (from 7:30 a.m. to

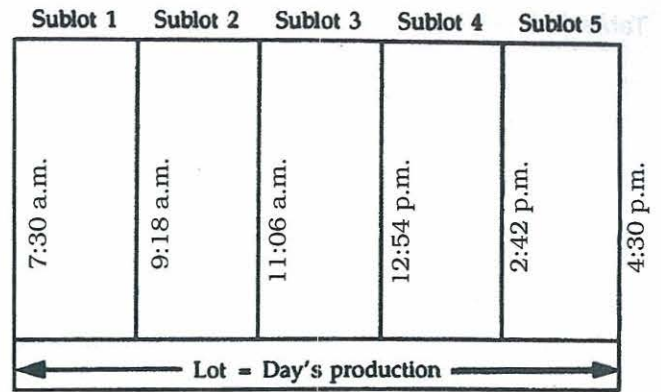


Fig. A1.1—Relationship between lot and sublots, time interval.

4:30 p.m.) to accomplish a day's production. Assume that the plant is running continuously through the lunch period.

Solution:

1. **Lot size**—The lot size is a day's production of 9 hours, starting at 7:30 a.m. and stopping at 4:30 p.m.

2. **Sublot size**—Divide the lot into five equal sublots by selecting five equal time intervals during the 9 hours that the plant is operating. The time interval for each subplot is

$$\begin{aligned} \text{subplot time interval} &= \frac{(9 \text{ h per lot})(60 \text{ min per h})}{5 \text{ sublots per lot}} \\ &= 108 \text{ min per subplot} \end{aligned}$$

Figure A1.1 shows a diagram of the division of the 9 hours of production time into five equal sublots.

3. **Sample increments**—One sample increment per subplot is required, but the exact time within each subplot when the sample increment should be taken is unknown. These times must be selected randomly.

Use the table of random numbers (Table A1.1) to randomize the timing of sample increments. Choose consecutive random numbers from either the X or Y columns. (The inspector's supervisor may specify the particular column to use.)

For this example, use Column X to obtain the timing of five sample increments. Note that Column Y could have been used instead and that any five consecutive values could be selected by starting at any point in the table.

Choose the first five numbers from Column X (0.4721, 0.6936, 0.6112, 0.7930, and 0.0652). To randomize the sampling time within each subplot, use the time interval (108 minutes) computed in Step 2, then multiply this time interval by each of the five random numbers:

- Sublot 1: $0.4721 \times 108 = 51 \text{ min};$
- Sublot 2: $0.6936 \times 108 = 75 \text{ min};$
- Sublot 3: $0.6112 \times 108 = 66 \text{ min};$
- Sublot 4: $0.7930 \times 108 = 86 \text{ min};$ and
- Sublot 5: $0.0652 \times 108 = 7 \text{ min}.$

Add the computed times to the starting times for each subplot to determine the randomized time at which each sample increment is to be obtained. The sampling times are:

Table A1.1—Table of random numbers

Random positions in decimal fractions (four places)					
	X	Y		X	Y
1.	0.4721	R 0.2091	51.	0.6985	L 0.8636
2.	0.6936	L 0.3182	52.	0.3410	L 0.5636
3.	0.6112	R 0.2909	53.	0.5937	R 0.3727
4.	0.7930	R 0.8908	54.	0.6912	R 0.4545
5.	0.0652	L 0.4818	55.	0.0318	R 0.7272
6.	0.04604	L 0.2091	56.	0.1303	R 0.8090
7.	0.0167	L 0.3727	57.	0.6893	R 1.0000
8.	0.0077	R 0.6181	58.	0.3886	R 0.7817
9.	0.6777	R 0.8636	59.	0.0312	R 0.8090
10.	0.8010	L 0.8362	60.	0.0166	R 0.5909
11.	0.3027	L 0.3454	61.	0.4609	L 0.4000
12.	0.9831	L 0.2364	62.	0.0893	L 0.9726
13.	0.7159	R 0.6181	63.	0.4542	L 0.1545
14.	0.3609	R 0.6454	64.	0.9363	R 0.1000
15.	0.8915	L 0.2636	65.	0.8183	R 0.5636
16.	0.6442	R 0.3182	66.	0.9401	L 0.5091
17.	0.1904	R 0.1818	67.	0.5967	L 0.9726
18.	0.6074	R 0.8908	68.	0.7547	R 0.2636
19.	0.7522	R 0.9181	69.	0.0101	R 0.2909
20.	0.7041	L 0.8362	70.	0.2896	L 0.8362
21.	0.5102	R 0.2364	71.	0.8011	R 0.6454
22.	0.2471	L 0.3182	72.	0.6718	L 0.6454
23.	0.5693	L 0.5636	73.	0.5567	L 0.1818
24.	0.8583	R 0.4545	74.	0.0481	L 0.2636
25.	0.3093	R 0.1818	75.	0.4266	L 0.9454
26.	0.9144	R 0.9181	76.	0.3941	R 0.5636
27.	0.7944	L 0.5909	77.	0.9876	L 0.7545
28.	0.8725	R 0.2636	78.	0.6313	R 0.7272
29.	0.0135	R 0.8908	79.	0.6803	R 0.3182
30.	0.2044	R 0.7272	80.	0.7955	L 0.9726
31.	0.2517	L 0.2909	81.	0.7399	R 0.8080
32.	0.2763	L 0.8090	82.	0.9328	L 0.5909
33.	0.0314	R 0.4818	83.	0.1507	L 0.4000
34.	0.9560	L 1.0000	84.	0.3087	R 0.3182
35.	0.4622	R 0.4000	85.	0.7513	L 0.1818
36.	0.1327	L 0.7817	86.	0.6469	R 0.4818
37.	0.6922	L 0.5636	87.	0.2536	R 0.7545
38.	0.0010	L 0.1273	88.	0.1488	R 0.1818
39.	0.7609	R 0.2091	89.	0.9411	L 0.5636
40.	0.5957	L 0.1000	90.	0.0571	R 1.0000
41.	0.3115	R 0.4000	91.	0.4797	R 0.9454
42.	0.3377	R 0.8362	92.	0.0866	R 0.4272
43.	0.5651	L 0.1545	93.	0.2889	R 0.1273
44.	0.4742	R 0.6727	94.	0.4783	L 0.7000
45.	0.9483	L 0.4000	95.	0.0304	R 0.9181
46.	0.2951	R 0.6451	96.	0.8945	R 0.4515
47.	0.0441	L 0.1273	97.	0.4499	R 0.2081
48.	0.9143	L 0.1273	98.	0.9209	L 0.9454
49.	0.5723	L 0.8362	99.	0.5827	L 0.5636
50.	0.6069	R 0.4000	100.	0.4560	L 0.8908

Notes:

X = decimal fraction of total length measured along the road from starting point (or decimal fraction of other units).

Y = decimal fraction of road from outside edge toward centerline (or decimal fraction of other units).

R = measurement from right edge.

L = measurement from left edge.

Sublot no. Sampling time

1	7:30 a.m.	+ 51 min	8:21 a.m.
2	9:18 a.m.	+ 75 min	10:33 a.m.
3	11:06 a.m.	+ 66 min	12:12 p.m.
4	12:54 p.m.	+ 86 min	2:20 p.m.
5	2:42 p.m.	+ 7 min	2:49 p.m.

The first sample increment is obtained from Sublot 1 at 8:21 a.m. The second sample increment is not drawn until 10:33 a.m., when Sublot 2 is in production. The remaining sample increments are obtained during the time when each of the last three sublots is in production.

The five sample increments constitute a sample size of $n = 5$, taken from five sublots of a day's production (the lot). Each increment of this sample size should be tested. The data would then be used to estimate the properties of the lot. In other words, the average, the sample standard deviation s , the coefficient of variation V , or any other statistic would be computed to reflect the statistical property of the lot when it is offered by the contractor or material supplier for acceptance sampling.

A1.2—Example 2: Sampling by material weight

Assume that the contract documents specify a lot size of 3000 tons, with five sublots per lot, and that one sample increment per sublot must be obtained. In addition, assume that the total tonnage required for the project is 15,000 tons. The sampling will be done from the hauling units at the manufacturing plant.

Solution:

Follow the same basic pattern as given for the previous example. First, identify the lot size, and then determine the number of lots, subplot size, and the point at which to obtain the sample increments.

Lot size and number of lots—Because 15,000 tons are required for the project and the lot size is 3000 tons, the total number of lots is

$$\text{number of lots} = \frac{15,000 \text{ tons}}{3000 \text{ tons per lot}} = 5 \text{ lots}$$

2. *Sublot size*—The subplot size is

$$\text{subplot size} = \frac{3000 \text{ tons per lot}}{5 \text{ sublots per lot}} = 600 \text{ tons per subplot}$$

Figure A1.2 shows the relationship between lot and subplot size.

Sample increments—The contract documents require one sample increment per subplot, but which load that should be sampled is unknown because the sample increments have not yet been randomized. Referring to Table A1.1, choose five random numbers from Column Y, starting with No. 17. Then multiply these numbers by each of the five sublots:

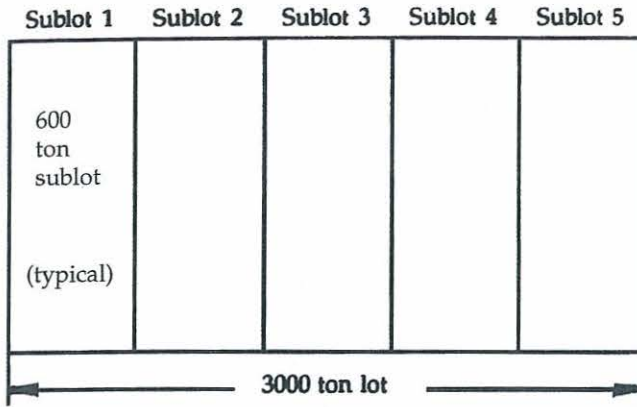


Fig. A1.2—Relationship between lot and sublots, quantity interval.

Sublot no.	Random number	Sublot size, tons	Ton to be sampled
1	0.1818	600	109th
2	0.8908	600	534th
3	0.9181	600	551st
4	0.8362	600	502nd
5	0.2364	600	142nd

Obtain the first sample increment at the 109th ton of the first subplot. Then wait until the first subplot of 600 tons is completed before selecting the second sample increment at the 534th ton of the second subplot. Follow the same procedure for obtaining the remaining three sample increments.

If a cumulative log of the tonnage being produced is kept, the sampling sequence for the lot (3000 tons) would be:

Sublot 1: = 109th ton;
Sublot 2: 600 + 534 = 1134th ton;
Sublot 3: 1200 + 551 = 1751st ton;
Sublot 4: 1800 + 502 = 2302nd ton; and
Sublot 5: 2400 + 142 = 2542nd ton.

Thus, in actual practice, the hauling units containing the 109th ton, 1134th ton, 1751st ton, and so on would be sampled.

A1.3—Example 3: Sampling by depth of concrete pavement

The depth of concrete pavement in a roadway must be sampled for acceptance. Assume that the contract documents require a lot size of 5000 linear feet, with each lot divided into five sublots and one sample increment per subplot. In addition, assume that the pavement width is 12 ft and the project begins at Station 100 + 00 and ends at Station 300 + 00.

Solution:

1. Lot size and number of lots

Lot size = 5000 linear feet

The distance from Station 100 + 00 to Station 300 + 00 is 20,000 ft. Thus, the number of lots is

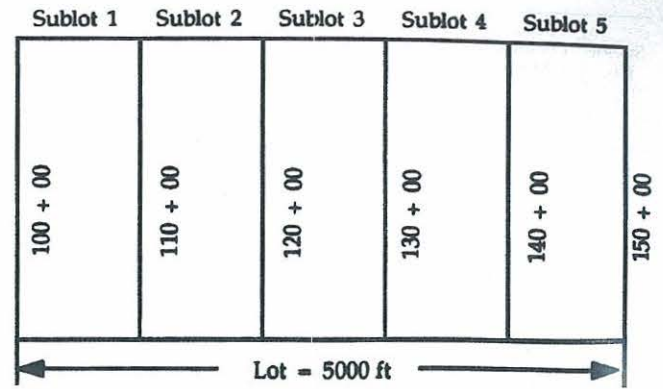


Fig. A1.3—Relationship between lots and sublots, distance interval.

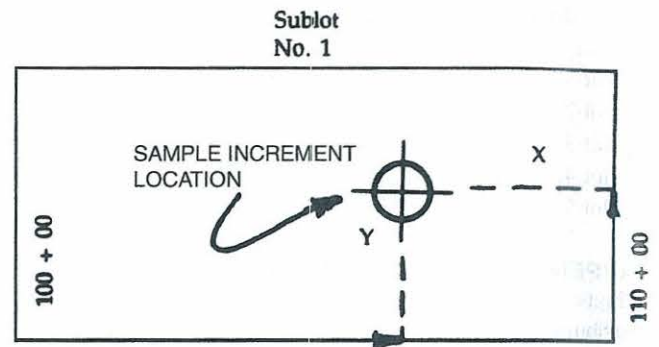


Fig. A1.4—Coordinate system for paved subplot.

$$\text{number of lots} = \frac{20,000 \text{ ft}}{5000 \text{ ft per lot}} = 4 \text{ lots}$$

2. Sublot size—The first lot begins at Station 100 + 00 and ends at Station 150 + 00. Divide the distance between these stations into five equal sublots:

$$\text{subplot size} = \frac{5000 \text{ ft per lot}}{5 \text{ sublots per lot}} = 1000 \text{ ft per subplot}$$

Figure A1.3 indicates how this lot is divided.

Note that random numbers starting somewhere else in Table A1.1, either in Column X or Y, could have been used. The inspector should select the starting point at random to avoid making the time or location of the sample increment predictable.

3. Sample increments—The point at which each sample increment will be obtained must be randomized in both the longitudinal (X) direction and the transverse (Y) direction. This location by X-Y coordinates is illustrated in Fig. A1.4 in which both the station and the offset are chosen in separate randomizing operations.

Referring to Table A1.1, use five consecutive random numbers from Column X and five from Column Y, starting with No. 43. Multiply the X numbers by the length of each subplot, and multiply the Y numbers by the 12 ft width of the roadway wearing surface.

Sublot No.	Longitudinal random number	Sublot size, length, ft	Sample location, ft, from start of sublot	Sample location, ft, from start of lot
1	0.5651	1000	565.1	565.1
2	0.4742	1000	474.2	1474.2
3	0.9483	1000	948.3	2948.3
4	0.2951	1000	295.1	3295.1
5	0.0441	1000	44.1	4044.1
Sublot no.	Transverse random number	Sublot size, width, ft	Location of sample, ft, from edge of roadway	
1	L 0.1545	12	1.8 from left edge	
2	R 0.6727	12	8.1 from right edge	
3	L 0.4000	12	4.8 from left edge	
4	R 0.6451	12	7.7 from right edge	
5	L 0.1273	12	1.5 from left edge	

Coordinates from beginning of lot and from right edge of roadway are, in feet:

Sublot 1: $X = 565.1$ $Y = 10.2$
 Sublot 2: $X = 1474.2$ $Y = 3.9$
 Sublot 3: $X = 2948.3$ $Y = 7.2$
 Sublot 4: $X = 3295.1$ $Y = 4.3$
 Sublot 5: $X = 4044.1$ $Y = 10.5$

APPENDIX 2—NORMAL DISTRIBUTION CURVES

Figures A2.1, A2.2, A2.3, and A2.4 show examples of normal distribution curves and how they vary yet are interrelated.

The sample standard deviation s and the related coefficient of variation V are measures of the scatter or variability of data. As illustrated in the figures, when the frequency distribution is long and flat, s (or V) is large, indicating wide variation. When the variability is small, s (or V) is small, and the data are closely packed. Figure A2.4 also shows that a high value of f'_{cr} is required when the variation is wide.

Figure A2.5 illustrates the proportion of the total area under a standard normal distribution curve for each successive step of distance s from the average (mean). These proportional areas for deviations of $1s$, $2s$, and $3s$ form the basis for statistically based tolerances for test values. For a standard normal distribu-

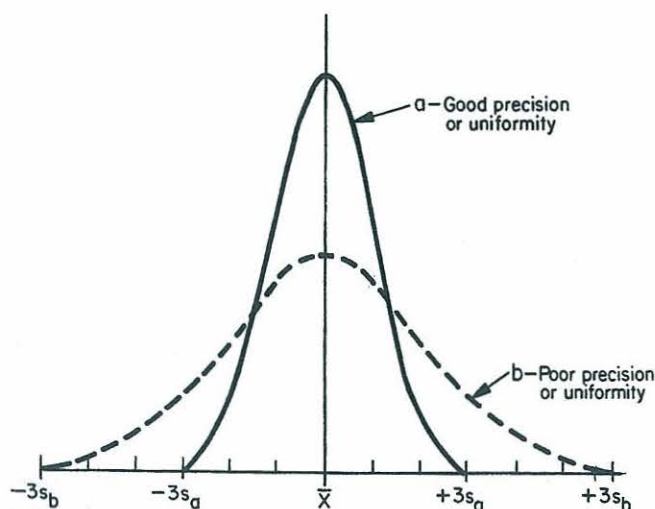


Fig. A2.1—Coordinate system for paved sublot.

tion, approximately 68% of all test values will fall within one standard deviation ($1s$) on each side of the average, approximately 95% will be within $2s$ of the average, and almost no test values (less than 0.3%) will be found outside the $3s$ limits.

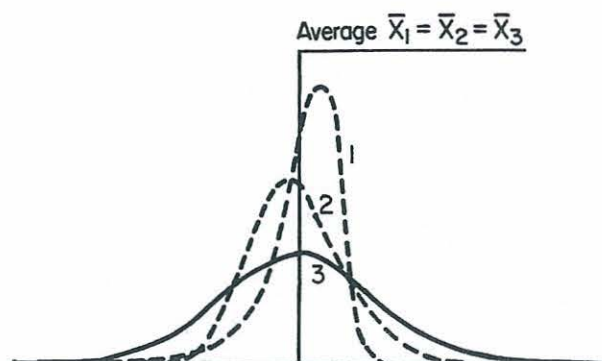


Fig. A2.2—Quite different distributions may have the same average.

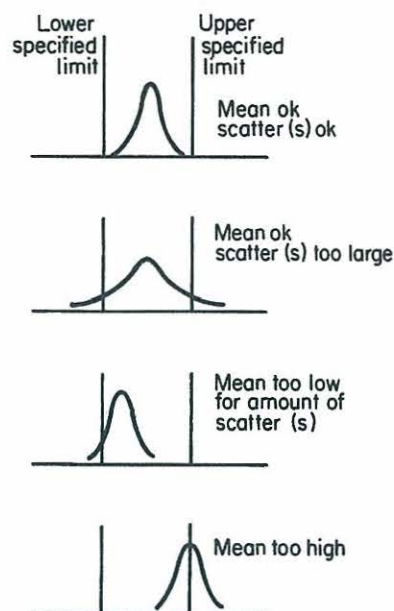


Fig. A2.3—Process control related to specification limits.

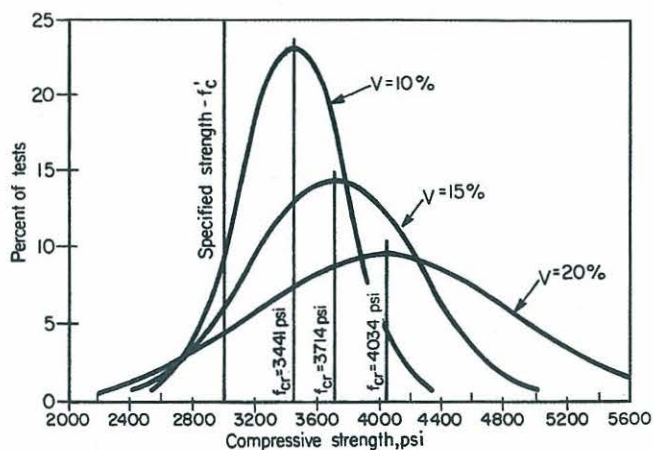


Fig. A2.4—Normal frequency curves and required f'_{cr} for different coefficients of variation.

APPENDIX 3—COMPUTING STANDARD DEVIATION AND REQUIRED AVERAGE CONCRETE STRENGTH

A3.1—Calculating standard deviation s

The sample standard deviation s is defined as the “root mean square deviation” of test results from their average and is computed by the following equation (described previously)

$$s = \sqrt{\frac{(X_1 - \bar{X})^2 + (X_2 - \bar{X})^2 \dots + (X_n - \bar{X})^2}{n - 1}} \quad (\text{A-1})$$

where X_1 , X_2 , and X_n are individual strength test values, and n is the number of tests.

Or, in a simpler form

$$s = \sqrt{\frac{1}{n-1} \left(\Sigma(X_i^2) - \frac{(\Sigma X_i)^2}{n} \right)} \quad (\text{A-1a})$$

where $\Sigma(X_i^2)$ is the sum of the squares of all individual tests, and $(\Sigma X_i)^2$ is the square of the sum of all individual tests. Example 1 shows how to compute s using Eq. (A-1a).

Note that $(n - 1)$ is used rather than the theoretical value n , which is applicable for an unlimited number of tests. The reason is that $(n - 1)$ increases the value of s and tends to compensate for the lesser reliability of a small number of tests.

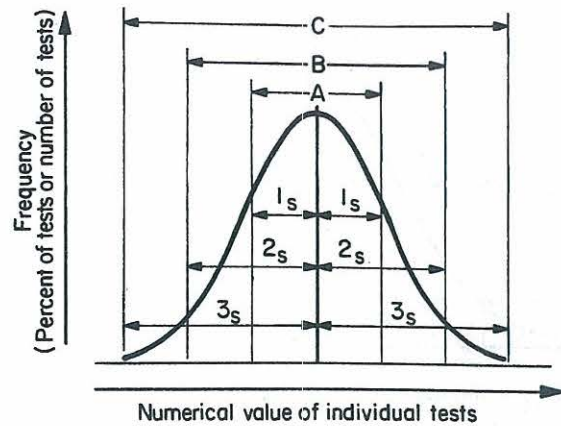
Example 1— Compute s by Eq. (A-1a)

Compressive strength tests of concrete cylinders		
Test no.	X value, psi	X^2 value
1	3315	10,989,255*
2	3090	9,548,100
3	3510	12,320,100
4	2900	8,410,000
5	3690	13,616,100
6	3310	10,956,100
7	3100	9,610,000
8	3490	12,180,100
9	3295	10,857,025
10	3500	12,250,000
11	2910	8,468,100
12	3700	13,690,000
13	3110	9,672,100
14	3790	14,364,100
15	3300	10,890,000
16	3290	10,824,100
17	3105	9,641,025
18	2800	7,840,000
19	3305	10,923,025
20	3495	12,215,025
	$\Sigma X_i = 66,005$	$\Sigma(X_i^2) = 219,264,225$

*Although the squared values have been written herein for illustration, this is unnecessary if calculator permits each squared number summed in memory as it is computed.

$n = 20$ (number of test results)

$$\frac{(\Sigma X_i)^2}{n} = \frac{66,005^2}{20} = 217,833,000$$



Area	Multiple of Std. Deviation (s)	Tabular Value	Actual Area as Decimal Part of Total Area Under Curve
A	1	.3413	2(.3413) = .6826
B	2	.4773	2(.4773) = .9546
C	3	.49865	2(.49865) = .9973

Fig. A.2.5—Division of standard normal distribution.

$$s = \sqrt{\frac{\Sigma(X_i^2) - [(\Sigma X_i)^2 / n]}{n - 1}}$$

$$= \sqrt{\frac{219,264,225 - 217,833,000}{19}}$$

$$s = \sqrt{75,327} = 274 \text{ psi}$$

A3.2—Calculating required average strength f'_{cr}

After the standard deviation has been computed, valuable information is available based on the theoretical normal probability curve. Regardless of the shape of the theoretical curve and the value of s , the area under the curve between $(\bar{X} + s)$ and $(\bar{X} - s)$ will always be 68.2% of the total area under the curve, and the area under the curve between $(\bar{X} + 2s)$ and $(\bar{X} - 2s)$ will be equal to 95.4% of the total. Considering only the half of the curve representing values less than $\bar{X}$, 34.1% of the total area will fall between $\bar{X}$ and $(\bar{X} - s)$, which leaves 15.9% of the area under the curve for values less than $(\bar{X} - s)$, as shown in Fig. A3.1.

These same percentages will apply for the number of tests involved as well as for an area. Therefore, 15.9% of the tests for any normal curve will fall below $(\bar{X} - s)$.

Table A3.1, from ACI 214R, is an adaptation of a table from a math handbook (the normal probability integral) altered to show the percentage of concrete strength tests falling below f'_c as a function of the required average strength f'_{cr} . For example, just as Fig. A3.1 shows that 15.9% of the tests will fall below $(\bar{X} - s)$, Table A3.1 shows that if

$$f'_{cr} = f'_c + s$$

then 15.9% of the tests will fall below f'_c .

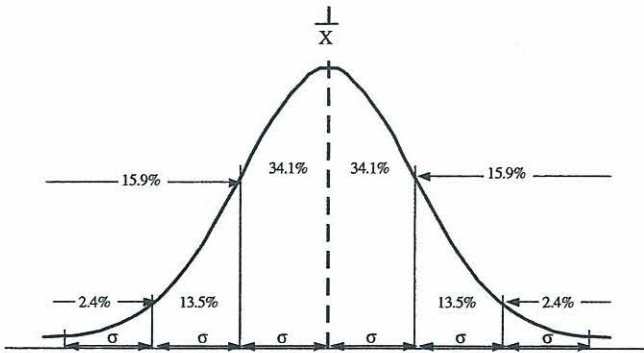


Fig. A3.1—Approximate distribution of area under the normal frequency distribution curve based on deviations from $\bar{X}$ in multiples of the population standard deviation σ .

Table A3.1—Expected percentages of test lower than f'_c

Required average strength f'_{cr}	Percent of low tests	Required average strength f'_{cr}	Percent of low tests
$f'_c + 0.1s$	46.0	$f'_c + 1.6s$	5.5
$f'_c + 0.2s$	42.1	$f'_c + 1.7s$	4.5
$f'_c + 0.3s$	38.2	$f'_c + 1.8s$	3.6
$f'_c + 0.4s$	34.5	$f'_c + 1.9s$	2.9
$f'_c + 0.5s$	30.9	$f'_c + 2.0s$	2.3
$f'_c + 0.6s$	27.4	$f'_c + 2.1s$	1.8
$f'_c + 0.7s$	24.2	$f'_c + 2.2s$	1.4
$f'_c + 0.8s$	21.2	$f'_c + 2.3s$	1.1
$f'_c + 0.9s$	18.4	$f'_c + 2.4s$	0.8
$f'_c + s$	15.9	$f'_c + 2.5s$	0.6
$f'_c + 1.1s$	13.6	$f'_c + 2.6s$	0.45
$f'_c + 1.2s$	11.5	$f'_c + 2.7s$	0.35
$f'_c + 1.3s$	9.7	$f'_c + 2.8s$	0.25
$f'_c + 1.4s$	8.1	$f'_c + 2.9s$	0.19
$f'_c + 1.5s$	6.7	$f'_c + 3.0s$	0.13

Table A3.1 is useful in establishing the required average strength and determining the probability of low tests occurring when s is known. With s computed from project data and f'_c established, Table A3.1 can be used to compute f'_{cr} . For example, assume that a designer would like to limit the probability of tests falling below 3000 psi to 5%, and the expected standard deviation of the concrete is 560 psi. The concrete should be designed for an average strength of 3925 psi based on the formula from Table A3.1 for 5% low test results

$$f'_{cr} = f'_c + 1.65s \text{ (by interpolation)} = 3000 + 1.65(560) \\ = 3000 + 925 = 3925 \text{ psi}$$

APPENDIX 4—CONTROL CHARTS ON CONCRETE MATERIALS

Typical control charts for aggregate gradation for individual tests and for range are shown in Fig. A4.1 and A4.2 (Pennsyl-

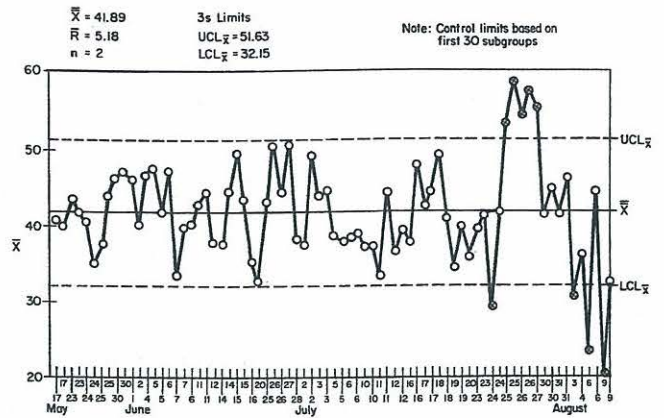


Fig. A4.1—Control chart for average ($\bar{X}$), 3/8 in. sieve, percent passing.

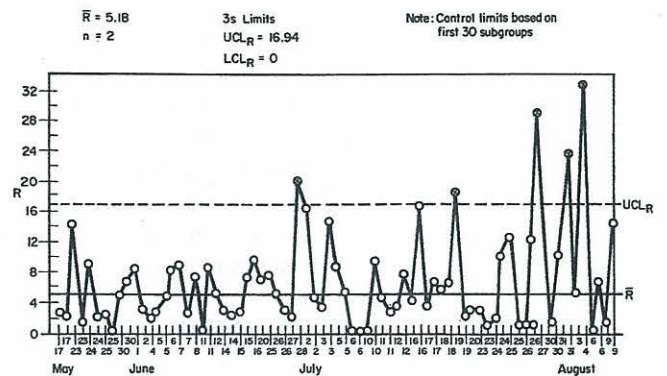


Fig. A4.2—Control chart for range (R), 3/8 in. sieve, percent passing.

vania State University 1974). Plot gradation represents individual percent retained on the particular sieve.

Example calculations and control charts for moving average (five tests) for sand-equivalent test results and for coarse-aggregate material passing the 1 in. sieve are shown in Examples 1 and 2, which are adapted from the California Department of Transportation (1968).

A4.1—Example 1: Calculations to determine moving averages for sand-equivalent test

The procedure and calculations are presented in Table A4.1.

Assume the contract documents require an individual test of not less than 73, and a moving average of not less than 75. The caution zone for this example was arbitrarily set between 75 and 80. The contractor discontinued operations and took steps to correct the deficiency before accepting additional material. Refer to Fig. A4.3 for a plot of the data.

A4.2—Example 2: Calculations to determine moving averages for 1-1/2 x 3/4 in. concrete aggregate (maximum variation of percentage of material passing 1 in. sieve)

The procedure and calculations are presented in Table A4.2.

The contractor's proposed average (job formula) is 26% passing the 1 in. sieve. Assume that the contract documents allow an individual test variation of 14% and a moving average variation of 12% from the average submitted by the

Table A4.1—Example 1: Calculations to determine moving averages for sand-equivalent test

Test no.	Date	Individual test result (minimum 73)	Moving average*	
			Sum [†] divided by number of results (minimum 75)	Rounded to
1	8/11	79 [‡]	—	—
2	8/14	85	$164 \div 2 = 82.0$	82
3	8/16	84	$247 \div 3 = 82.7$	83
4	8/18	72 [§]	$320 \div 4 = 80.0$	80
[§] Waived and accepted by engineer.				
5	8/22	80	$400 \div 5 = 80.0$	80
6	8/24	75	$396 \div 5 = 79.2$	80
7	6/25	74	$385 \div 5 = 77.0$	77
8	8/28	68	$369 \div 5 = 73.8$	—
Materials rejected and results not shown on control charts. Operation discontinued and significant steps taken by contractor to correct deficiency before accepting additional material.				
9	8/29	79	—	—
10	8/31	80	$159 \div 2 = 79.5$	80
11	9/5	81	$240 \div 3 = 80.0$	80
12	9/7	83	$323 \div 4 = 80.7$	81

*Results rounded up to the next whole number using the same number of significant figures as in the individual test result.
[†]Sum of the five most recent individual test results, including the current test result.
[‡]Show test result as the first value on the moving-average control chart.

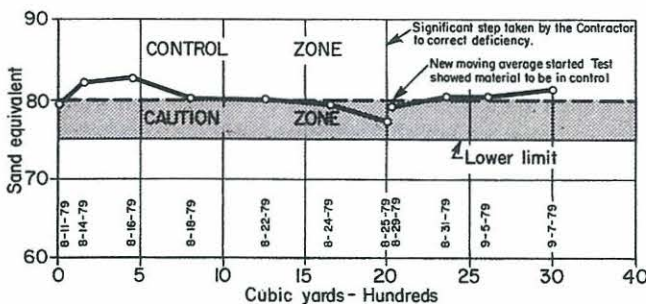


Fig. A4.3—Control chart of moving averages for sand equivalent.

contractor. The caution zone for this example was arbitrarily set between 4% higher than the lower limit and 4% lower than the upper limit. Refer to Fig. A4.4 for a plot of the data.

CHAPTER 3—INSPECTION AND TESTING OF MATERIALS

Materials are inspected and tested to ensure that they meet contract requirements and that they are properly stored, handled, and used in the work. If materials are inspected for acceptance before being shipped to the job site, the field inspector should check their condition once they reach the site, looking for degradation that may have occurred during shipment and storage. The contractor should make available to the inspector all records pertaining to material shipments and tests and certifications.

This chapter discusses sampling and testing procedures for materials commonly used in concrete construction, including cement, aggregates, mixing water, admixtures, reinforcement, curing compounds, and joint sealants. Standard methods of testing concrete are discussed in Chapter 19.

Table A4.2—Example 2: Calculations to determine moving averages for 1-1/2 x 3/4 in. concrete aggregate

Test no.	Date	Individual test result (limits 12 to 40%)	Moving average*	
			Sum [†] divided by number of results (limits 14 to 38%)	Rounded to
1	6/5	27 [‡]	—	—
2	6/6	24	$51 \div 2 = 25.5$	26
3	6/8	28	$79 \div 3 = 26.3$	26
4	6/12	29	$108 \div 4 = 27.0$	27
—	6/14	35	$143 \div 5 = 28.6$	29
—	6/16	34	$150 \div 5 = 30.0$	30
—	6/20	42 [§]	$168 \div 5 = 33.6$	34
[§] Waived and accepted by engineer.				
8	6/22	38	$178 \div 5 = 35.6$	36
9	6/26	40	$189 \div 5 = 37.8$	38
10	6/27	42	$196 \div 5 = 39.2$	—

^{||}Materials rejected and results not shown on control charts.
The contractor discontinued operations and took steps to correct deficiency before accepting additional material.

*Results rounded up to the next whole number using the same number of significant figures as in the individual test result.
[†]Sum of the five most recent individual test results, including the current test result.
[‡]Show test result as the first value on the moving-average control chart.

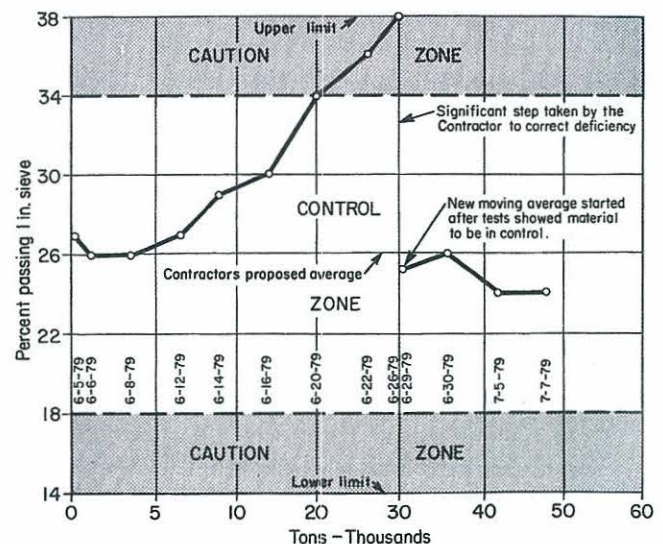


Fig. A4.4—Control chart for moving averages for grading analysis.

3.1—Cement

3.1.1 Standard types—Five standard types of portland cement are used in concrete, as specified in ASTM C150:

- **Type I**—A cement for general use when the special properties of other cements are not needed;
- **Type II**—A cement for general use that has moderate sulfate resistance and moderate heat of hydration;
- **Type III**—A cement for use when high early strength is desired;
- **Type IV**—A cement for use when low heat of hydration is necessary, such as for mass concrete; and
- **Type V**—A cement for use when high sulfate resistance is required, such as for structures in contact with soils

or groundwater having high sulfate content, and for concrete in contact with concentrated domestic sewage.

ASTM C150 also specifies three types of air-entraining cement: IA, IIA, and IIIA, which correspond to Types I, II, and III portland cement. These cements contain an air-entraining admixture that is interground with the clinker during manufacture. Air-entraining cement, however, may not be available in all markets, and better control of air content in concrete can usually be achieved by adding the air-entraining admixture at the mixer. For more information on air entrainment and air content, refer to Chapters 5, 6, and 19.

3.1.2 Blended cements and other hydraulic cements—

Blended cements containing SCMs, as given in ASTM C595, are available in certain localities. These include:

- Type IS: portland blast-furnace slag cement;
- Types IP and P: portland-pozzolan cement;
- Type I(PM): pozzolan-modified portland cement;
- Type S: slag cement; and
- Type I(SM): slag-modified portland cement.

Hydraulic cements set and harden by reacting chemically with water. All portland cements and blended cements are hydraulic cements. ASTM C1157 is a performance specification that lists six types of hydraulic cement:

- Type GU: general use;
- Type HE: high early strength;
- Type MS: moderate sulfate resistance;
- Type HS: high sulfate resistance;
- Type MH: moderate heat of hydration; and
- Type LH: low heat of hydration.

The first comprehensive data on the use of SCMs in concrete in North America were reported in 1937 by Davis et al. SCMs have been widely used in massive structures, such as dams, as well as in structural concrete. They can improve workability, reduce heat of hydration, decrease permeability, improve sulfate resistance, and reduce expansion caused by reactions between certain aggregates and the alkalis in cement. Furthermore, using SCMs can result in greater economy, particularly if the concrete compressive strength is specified at ages exceeding 28 days. ASTM C618 provides requirements for three classes of fly ash and natural pozzolans:

- Class N: Raw or calcined natural pozzolans, such as some diatomaceous earths, opaline cherts, shales, tuffs, volcanic ashes or pumicites, and some clays and shales;
- Class F: Fly ash that is a finely divided residue resulting from the combustion of ground or powdered anthracite or bituminous coal; and
- Class C: Fly ash produced from burning lignite or sub-bituminous coal and generally having sufficient lime content to have some cementitious properties.

Fly ash and natural pozzolans for use in concrete are discussed in more detail in ACI 232.1R and 232.2R.

In the 1980s, an ultrafine pozzolan, silica fume, gained attention as a mineral admixture, leading to the development of a new generation of high-strength, low-permeability concretes, with strengths exceeding 12,000 psi (85 MPa). A by-product of the industrial manufacture of ferrosilicon and metallic silicon, silica fume has extremely fine particles about 1/100 the size of portland cement grains (about the size

of cigarette smoke particles). For more information about silica fume, consult ACI 234R. Requirements for silica fume are provided in ASTM C1240.

Requirements for slag cement are provided in ASTM C989, which classifies it as Grade 120, 100, or 80, based on compressive strength results of the slag activity index test with portland cement. Slag cement is a by-product of the steel-making process. Molten slag is drawn off the steel furnace and then rapidly spray-cooled with water, creating a granulated slag material. These granules are then finely ground to a cement-like powder. Unlike other pozzolans, slag cement does not need to be combined with portland cement to develop significant strength and cementing properties. Slag cement is described in more detail in ACI 233R.

3.1.3 Optional requirements—ASTM C150 contains additional optional requirements for cement to be applied at the option of the purchaser. They pertain to low-alkali content, false set, 28-day compressive strength, heat of hydration, and sulfate resistance.

3.1.3.1 Low alkali content—When the aggregates to be used contain elements known to be destructively reactive with sodium or potassium oxides (the minor alkalies found in cement), using low-alkali cement or an acceptable pozzolan will better ensure the durability and serviceability of the concrete. As defined by ASTM C150, low-alkali cement contains not more than 0.60% of these oxides.

Usually, concrete made with low-alkali cement will be free of objectionable alkali-silica expansion. Petrographic examination of the aggregate is one method of identifying reactive aggregate (ASTM C295). An indication of the potential alkali reactivity of cement-aggregate combinations can be determined by the mortar bar tests (ASTM C227, C1293, or C1260) and, less reliably, by the quick chemical method (ASTM C289).

3.1.3.2 False set—ACI 116R defines false set as “the rapid development of rigidity in a freshly mixed portland-cement paste, mortar, or concrete without the evolution of much heat, which rigidity can be dispelled and plasticity regained by further mixing without addition of water.” Flash set (or quick set) is defined as “the rapid development of rigidity in a freshly mixed portland-cement paste, mortar, or concrete, usually with the evolution of considerable heat, which rigidity cannot be dispelled nor can the plasticity be regained by further mixing without the addition of water.” The presence of false set and flash set can be determined by ASTM C359 (mortar method) or C451 (paste method). These tests, however, do not address all the mixing, placing, temperature, and field conditions that can cause early stiffening.

Although both of these properties are objectionable, flash set is the most detrimental. Once flash set occurs, considerable water must be added to the concrete to regain plasticity, which reduces strength and greatly increases plastic and drying shrinkage cracking. Cement with flash-set properties usually will fail the requirements for time of set in ASTM C150.

False set generally will have no harmful effects on concrete quality, particularly when transit mixing is used. The longer mixing time will restore plasticity without water addition (and often without personnel being aware that false

set has occurred). With short mixing times, however, severe false setting may require the addition of slightly more mixing water, which can reduce concrete strength and increase drying shrinkage. With very short mixing times, false set may occur after the concrete has been dumped from the mixer, when it is too late to regain plasticity by additional mixing.

3.1.4 Sampling and testing procedures—Cement is usually sampled and tested by the mill, which then issues mill test results and certifications. If the cement remains in bulk storage at the mill for more than 6 months after testing, the material should be retested before shipment. Large users of cement, such as state highway departments, will often sample and test the cement in each silo themselves or hire an independent laboratory to do the testing. Other users typically obtain samples from incoming shipments and have the cement tested by an independent laboratory as necessary.

The plant inspector checks incoming shipments against mill test certificates and owner approvals. Inspectors should also make sure that seals on bulk-shipment cars are unbroken. If the cement is to be sampled at the job site, the inspector should use the following procedures to secure a representative sample of the portion in question in accordance with ASTM C183:

- If the cement is in bags, take a small quantity from one bag for each 100 bags or a fraction thereof. After thoroughly mixing these amounts, choose a sample with a sample splitter or by the quartering method (ASTM C702). Samples for testing should be at least 10 lb (4.5 kg) each (or at least 5 lb [2.3 kg] each if they are to be combined into a 10 lb [4.5 kg] sample);
- If the cement is shipped in bulk, take samples with a slotted sampling tube or by drawing off a considerable quantity of cement at the discharge opening. From the quantity secured, blend a composite sample as described previously;
- Place the cement sample in a clean, dry, waterproof container, and close the cover tightly to exclude air and moisture. Wrapping duct tape around the seam of the cover will improve the seal; and
- Identify the sample clearly, both inside and outside the container. Give the date, name of job, name of inspector, car or lot number, brand of cement, quantity represented by the sample, temperature when sampled, authority or reason for sampling, and tests desired.

3.2—Aggregates

Because nearly 70% of the concrete mass is aggregate, examining and testing aggregates for acceptability involves making necessary tests, seeing that the aggregates are properly stored and handled, and checking batching operations. Minimizing variations in the aggregates as batched is important to the production of quality concrete.

3.2.1 Specifications—Standard specifications for aggregates include those for normalweight aggregate (ASTM C33), recommended stock sizes of aggregate for highway construction (ASTM D448), and lightweight aggregate (ASTM C330 and C332). In general, aggregates should be clean, hard, sound, and durable, and particle sizes should be

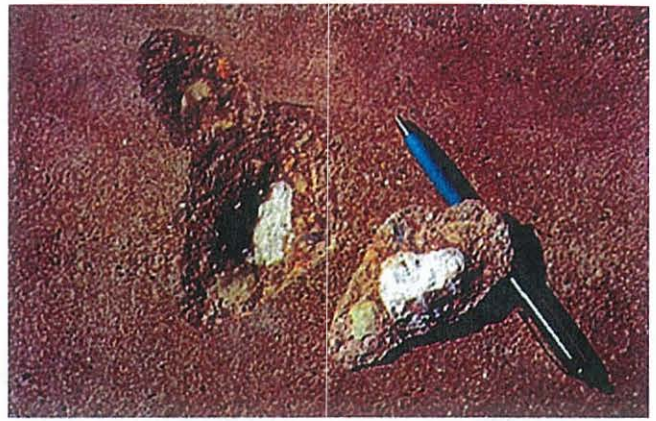


Fig. 3.1—Lightweight chert can cause popouts.

graded within stated limits. Specifications usually require keeping the chosen grading of fine aggregate reasonably uniform by restricting the range of fineness modulus (refer to the discussion in this chapter under Section 3.2.2.3).

ASTM C33 restricts various deleterious substances to small percentages. The following is a list of these substances and the reasons for restricting them in the final aggregate product:

- *Clay lumps and friable particles*—Unsound particles; may increase water demand if they break down during mixing; affect wear resistance and durability; may cause popouts;
- *Material finer than No. 200 (75 μ m) sieve*—Increases demand for mixing water; affects cement paste bond to aggregate; affects the ability to entrain air;
- *Coal and lignite*—May cause detrimental staining of surfaces; cause popouts; interfere with ability of admixtures to entrain air;
- *Soft particles*—Cause popouts; affect durability and wear resistance of concrete; and
- *Lightweight chert (with specific gravity less than 2.40)*—Reduces durability of concrete; causes popouts (Fig. 3.1).

3.2.2 Sampling procedures—Sampling methods should be used conforming to the requirements of ASTM D75. Some local variations in the aggregate source should be expected, and samples should be selected so that the effects of these variations are not over- or under-emphasized. In judging test results, the statistical distribution of the amount of undesirable material and the quality-control charts of previous samplings should be considered, as described in Chapter 2. For example, a single clay lump in one sample does not justify rejection of an entire carload of aggregate unless the sample is obviously representative. Aggregate should only be accepted or rejected in accordance with the contract documents for the project.

3.2.2.1 Where to take samples—Aggregate samples for testing may be taken from conveyors, bins, cars, barges, or stockpiles, but it is best to select samples that will represent the run of material as batched. Thus, sampling from conveyors or from the discharge openings of bins is preferable (Fig. 3.2).

ACI 311.1R-07
Concrete Vibration



Fig. 3.2—Where access is convenient and safe, material may be sampled from hopper to bin.



Fig. 3.3—Aggregate stockpiles are difficult to sample properly.

ASTM D75 does not recommend sampling stockpiles because they are difficult to sample properly (Fig. 3.3).

The most representative sample possible is that from a conveyor belt. For fine aggregate, scoopfuls should be taken as the belt goes by to obtain a bucketful from which the test sample can be split or quartered (ASTM C702). For coarse aggregate, samples should be taken from material removed from a short length of belt while the conveyor is stopped. If arrangements cannot be made to stop the conveyor or if no conveyors are used on the job, samples should be taken from the discharge of coarse aggregate from a chute or bin gate. At least several cubic feet of material should be taken, and the test sample quartered from this amount. Such samples are most representative when quartered from material taken from the first, middle, and last of the material to be tested.

If it is necessary to sample from a stockpile, samples should be made up of at least three increments taken from the top third, at the midpoint, and at the bottom third of the column of the pile.

Methods of sampling from belts, bins, and stockpiles are summarized in ASTM D75.

3.2.2.2 Obtaining representative test samples—When making up samples taken from surface aggregate in a bin, car, barge, or stockpile, portions should be taken at several

Table 3.1—Typical computations of fineness modulus

Percent coarser than sieve number	Sand	No. 4 to 1 in. (4.75 to 25.0 mm) coarse aggregate	Mixture 40% sand and 60% coarse aggregate*
	Cumulative percent by weight retained		
3 in. (75 mm)	—	0	0
1-1/2 in. (37.5 mm)	—	3	2
3/4 in. (19.0 mm)	—	49	29
3/8 in. (9.5 mm)	—	77	46
No. 4 (4.75 mm)	4	96	59
No. 8 (2.36 mm)	15	100	66
No. 16 (1.18 mm)	37	100	75
No. 30 (600 μ m)	62	100	85
No. 50 (300 μ m)	85	100	94
No. 100 (150 μ m)	98	100	99
Total	301	725	555
Fineness modulus	3.01	7.25	5.55

*0.40 $\times$ percent of sand plus 0.60 $\times$ percent of coarse aggregate.

separated points. An excess of unrepresentative material should be avoided, such as fines in the center, coarser material toward the edges, and surface materials wetter or drier than average. All material, however, should be represented in the sample. To obtain a sample from under the surface, a board or similar sampling shield should be inserted into the surface of sloping material to prevent it from running down.

When possible, fine-aggregate samples should be taken from damp material to avoid the segregation that occurs in dry sand. Samples should be taken from below the surface, preferably by driving a sampling tube into the sand at several separated points. When sampling for moisture content, nonuniform moisture distribution should be taken into consideration. For example, a mass of wet sand will be wetter toward the bottom a few hours after water has been applied.

If two or more types of sand or coarse aggregate are to be batched separately and blended in the mixer to produce a specified grading, each type should be sampled and tested separately and the results computed for the blended aggregate from the proportions of each type. If necessary, the proportions can be adjusted to obtain the required grading. The computation method for combining two aggregates into a desired grading is identical to that for overall grading of fine and coarse aggregates, illustrated in Table 3.1.

The sample size depends on the type and number of tests to be made and the maximum nominal size of the aggregate. For example, samples of fine aggregate should contain not less than 25 lb (11 kg), and samples of coarse aggregate with a maximum size of 1 in. (25 mm) should contain not less than 110 lb (50 kg). Appendix 5 includes a table showing minimum sample weights for various maximum nominal aggregate sizes.

Some federal specifications for airport pavements require much tighter limits on the amount of deleterious substances in fine and coarse aggregates than those of ASTM C33. These restrictions may range from 0.1% for soft particles and chert to 0.5% for material passing the No. 200 (75 μ m) sieve. To provide meaningful results and achieve statistical control of tests (Chapter 2) for compliance with these tight speci-

cations, it is necessary to take much larger sample sizes. To obtain a representative test sample of aggregate from a large sample, a sample splitter or the quartering method should be used (ASTM C702).

Unless the particular test calls for an exact amount of material, the nearest approximate amount resulting from quartering or from the use of the sample splitter should be taken. Adjusting the amount to exactly obtain some arbitrary quantity by adding or removing material may change the average characteristics of the sample, and should not be done unless specifically required by the test method.

If samples are to be shipped to a laboratory for testing, a clean container should be used because even a small amount of some materials (such as sugar or fertilizer) can contaminate the sample. The container should be closed tightly to prevent contamination or loss of fines. As when labeling cement samples for shipment, the sample should clearly be identified inside and outside the container. The date, kind of aggregate, quantity represented by sample, location and other conditions of sampling, authority or reason for test, and kind of test desired should be given.

3.2.2.3 Tests for grading—Sieve analysis of aggregates provides information for use in controlling gradation and checking compliance with specified grading requirements.

The amount of the test sample to use for a sieve analysis should be selected in accordance with ASTM C136. To avoid segregation, the sample of fine aggregate should be reduced to the desired size before drying. The sample should be separated into the various sizes with the specified series of sieves, preferably mounted on a mechanical shaker. The sieves should be shaken laterally and vertically, keeping the sample moving continuously over the sieve surfaces. Aggregate fragments on the sieves should not be moved by hand or metal slugs added to the sieves as sieving aids. Sieves, particularly those of small size, should be kept clean and unclogged.

A sieve analysis should be performed only with standard sieves conforming to ASTM E11. With the aid of a conversion chart or graph, it is possible to convert a sieve analysis made using one series of sieves to an analysis using another series of sieves (perhaps having round instead of square openings, or the reverse). The conversion, however, is only approximate. To prevent disputes, the type of sieves specified by the contract documents should be used.

Because aggregates, particularly coarse aggregates, vary considerably within stockpiles and bins, any single test is of limited significance. On a job of moderate size, it may be necessary to perform a sieve analysis once or twice a day and whenever it appears that changes in grading have occurred. If possible, each new test should be averaged with at least two immediately preceding tests on the same material to obtain a more representative analysis of its general run. The average may then be used to adjust the mixture proportioning or to determine if grading requirements have been met.

The fineness modulus is an index number roughly proportional to the average size of particles in a given aggregate; the coarser the aggregate, the higher the fineness modulus. Although the fineness modulus does not indicate grading or distinguish between a single-size aggregate and a

PRINCIPAL TESTS ON AGGREGATES

Concrete inspectors are concerned with three types of tests on aggregates:

1. Initial laboratory acceptance tests for suitability as to grading, cleanness (silt and organic impurities), soundness and durability, abrasion resistance, deleterious materials, foreign substances, and mineral composition.
2. Secondary laboratory tests on approved samples to determine physical properties used in mixture proportioning, such as absorption, specific gravity, density, voids, and bulking.
3. Field tests for acceptance or control, such as grading, cleanness, deleterious materials, and moisture content.

The significance of these tests is discussed in ASTM STP 169-C (ASTM International 1994).

graded aggregate having the same average size, it does indicate whether one graded aggregate is finer or coarser than another. Thus, it is used for specification and record purposes and is particularly useful for controlling grading and uniformity. One computation method uses the fineness modulus for aggregate proportioning (Portland Cement Association 2002). ASTM specifications require that the fineness modulus of a shipment of fine aggregate not vary more than a certain amount (in some cases, 0.20) either way from the fineness modulus of an acceptably representative preliminary sample. Appendix 5 discusses how to compute the fineness modulus.

3.2.2.4 Tests for material finer than No. 200 (75 μ m) sieve—The extremely fine mineral material (clay, silt, dust, or loam) occurring in most aggregates causes an increase in the amount of mixing water needed and inhibits the development of entrained air. It also tends to work up to the concrete surface, where it can cause cracking due to shrinkage upon drying. If the fines adhere to the larger aggregate particles, they can interfere with bond between the aggregate particles and cement paste. For these reasons, specifications limit the amount of fine material in aggregates.

The test for percentage of materials finer than the No. 200 (75 μ m) sieve is described in Appendix 5 and in ASTM C117. The process begins by washing the fine material through sieves from a weighed sample of oven-dry aggregate. The remaining aggregate is then oven-dried again and weighed to find the amount of fines removed. Refer to ASTM D2419 for a standard field test for fine mineral material in sand.

Often, in the field, a simpler sedimentation test is used to determine the approximate amount of fines in sand (U.S. Bureau of Reclamation 1988) that does not require drying the test sample. For this test, a clear glass jar or bottle (preferably a 32 oz [1 L] graduated prescription bottle) is filled approximately half full with the sand. Clear water is added until the contents reach a level of approximately 3/4 of the volume of the bottle. The container is vigorously shaken, and the contents are allowed to settle for 1 hour. The depth of the layer of fines on top of the sand is measured; roughly 2% by volume (depth) is the equivalent of 1% by mass. If this exceeds a permissible percentage of the depth of sand and fines, the sand is tested by the more accurate method specified.

To determine the amount of clay lumps or other friable particles, use the method described in ASTM C142, which uses material retained on the No. 16 (1.18 mm) sieve (ASTM C117). For fine aggregate, the material is dried, soaked in distilled water for 24 hours, and then the friable particles are broken by squeezing and rolling between the thumb and forefinger. For coarse aggregate, the material is dried and separated into the four sizes, from No. 4 to 1 in. (4.75 to 25 mm). The different sizes are then soaked in distilled water for 24 hours, and the friable particles are broken as for fine aggregate. The residue is separated from each sample by wet sieving over prescribed sieve sizes, and the amount of friable material is computed as a percentage of the sample mass, as shown in Appendix 5.

In coarse aggregate, fine mineral material or crusher dust in objectionable quantities is often clearly visible. ASTM C33 limits materials passing the No. 200 (75 μ m) sieve to 1% by weight. This limit increases to 1.5% if the fine material is essentially free of clay or shale. Other criteria also apply, as specified in ASTM C33.

Routine tests are made (daily testing recommended) for material finer than the No. 200 (75 μ m) sieve when statistical analysis is desired or when there is reason to believe from the appearance of the aggregate or from sieve analysis that the permissible amount of fine minerals is being exceeded.

3.2.2.5 Tests for organic impurities in fine aggregate—

A small amount of some types of organic material in fine aggregate can delay or prevent hardening of concrete and drastically reduce concrete strength. Organic matter in sand usually occurs in the form of decayed vegetable matter. In certain locations, tannic acid from pine tree roots leaves deposits on particle surfaces that cannot be removed by washing.

A color-comparison test for the presence and approximate amount of organic material is described in ASTM C40. The use of a slightly damp sample is recommended for the test because an excess of surface moisture dilutes the testing solution, and mechanically dried aggregate may lose some of the organic material in handling or by burnoff from drying at high temperatures. The sample is immersed in a 3% (by weight) solution of sodium hydroxide in a clear glass bottle. After vigorous shaking, the sample is allowed to stand for 24 hours. The color of the supernatant liquid is noted and compared with a standard color solution or standard color plate prepared according to ASTM C40. If the solution is clear, lighter than, or equal to the standard color, the fine aggregate is considered to be free of injurious amounts of organic matter. If the solution is darker than the standard color, organic impurities may be present in quantities sufficient to affect the concrete, and it may be necessary to check the effects of these impurities by comparison strength tests of mortar containing the fine aggregate with and without the organic impurities, in accordance with ASTM C87. Coal or lignite particles in fine aggregate can produce a dark solution in the comparison test, but such particles are permissible in fine aggregate provided that the amounts do not exceed the allowable limits in ASTM C33.

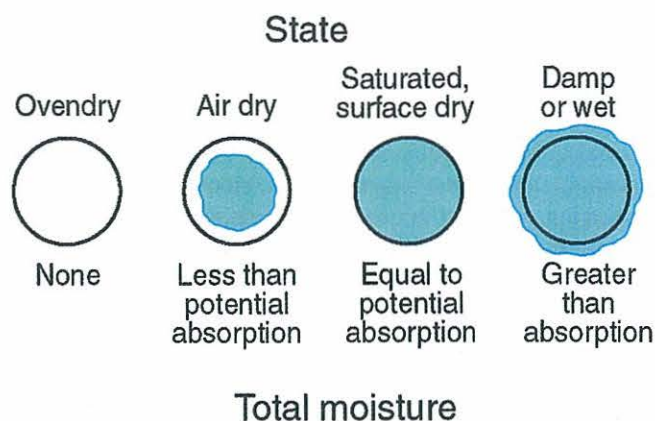


Fig. 3.4—Range of moisture content in aggregate.

The frequency of testing for organic matter depends on the condition and homogeneity of the fine aggregate and on the requirements for statistical evaluation (Chapter 2). For washed fine aggregate with a satisfactory record, conducting the color-comparison test as infrequently as once a week may be sufficient. More frequent testing may be necessary if the amount of organic matter is near the permissible limit, if the supply changes, or if the concrete hardens more slowly than normal.

3.2.2.6 Tests for moisture and absorption—Tests for moisture content and absorption of aggregates are conducted to:

- Determine the amount of water the aggregates contribute to or absorb from a concrete mixture. An increase of 1% in the moisture content of fine aggregate (for example, from 4 to 5%) can increase concrete slump for normal concrete without water-reducing admixtures and decrease compressive strength; and
- Determine the necessary adjustment in weight or volume to secure uniform quantities of equivalent saturated, surface-dry aggregates in the batches to obtain correct concrete yield. If measurement is by volume, the bulking factor should be known. The bulking factor is the ratio of the surface-dry rodded aggregate density to the difference between the damp loose-aggregate density and the weight of surface moisture unit volume of dry, loose aggregate.

The moisture content of aggregates may be in any of four states, as shown by Fig. 3.4:

- *Oven-dry*—Completely dry and fully absorbent;
- *Air-dry*—Dry at the surface with some interior moisture, but in less than the amount required to saturate the particles (or less than the absorption capacity). Thus, the aggregate is somewhat absorbent;
- *Saturated and surface-dry (SSD)*—Ideal condition in which the aggregate neither contributes water to nor absorbs water from the paste; or
- *Moist*—Contains excess moisture on the surface.

A clear understanding of these conditions is necessary for proper aggregate proportioning and batching.

All computations should be based on aggregate in a SSD condition. Though it is not possible to secure aggregates for construction in this ideal condition, simple arithmetic can be

used to convert the measurements on damp or dry aggregate into terms of the equivalent amounts of SSD aggregate. It is only necessary to know the total moisture content and absorption capacity of the aggregate to determine free moisture, as in the following equation

$$\text{free moisture} = \text{total moisture} - \text{absorption capacity}$$

If the total moisture is less than the absorption capacity (as in the case of air-dry aggregate), the surface moisture will be negative, and the aggregate will absorb some of the mixing water.

Tests for determining moisture content and absorption of aggregates are described in detail in *Design and Control of Concrete Mixtures* (Portland Cement Association 2002) and in ASTM C70, C127, C128, and C566. The significance of these tests is discussed in ASTM STP 169-C (ASTM International 1994). For laboratory purposes, the tests are made with a precision that will yield results accurate to the nearest 0.1%.

When determining moisture content of fine aggregate in the field, speed is more important than refinement of test results because information on changes in moisture content is needed in time to make appropriate adjustments in batching. Electrical and nuclear equipment for indicating moisture is available that immediately reflects significant changes in moisture content of fine aggregate. This equipment, however, should be periodically calibrated against ASTM tests to confirm accuracy and to indicate necessary equipment adjustments.

3.2.2.6.1 Absorption—For practical purposes, the total absorption capacity of a given aggregate does not vary. In hard, dense natural aggregates it usually amounts to 1% or less of the aggregate weight, but it may be higher in some natural aggregates. In manufactured aggregates, such as slag cement, it may be as high as 5%. On small jobs, the absorption value for a given type of aggregate often is assumed, but where careful control is desired, the absorption should be accurately determined.

Coarse-aggregate absorption is determined using procedures in ASTM C127, omitting weighing the sample underwater if specific gravity is not to be measured. A damp sample is brought to the SSD condition, weighed, completely dried by heating, and reweighed. The loss in weight after heating represents the absorption capacity. The surface-dry condition is obtained by wiping the wet particles to remove surface water sheen with an absorbent cloth, as per the procedure.

Fine-aggregate absorption is determined following procedures in ASTM C128, omitting the use of the flask if specific gravity is not measured. A damp sample is brought to the SSD condition, weighed, oven-dried, and reweighed. The surface-dry condition is determined using the cone test described in ASTM C128 and in Appendix 5 of this document. At the point in the series of tests when the pile of fine aggregate begins to slump, the sand is saturated and surface-dry. Another sign that the drying sand has become saturated and surface-dry is when it just ceases to adhere to a clean glass rod or jar. Neither of the tests for the SSD condition is

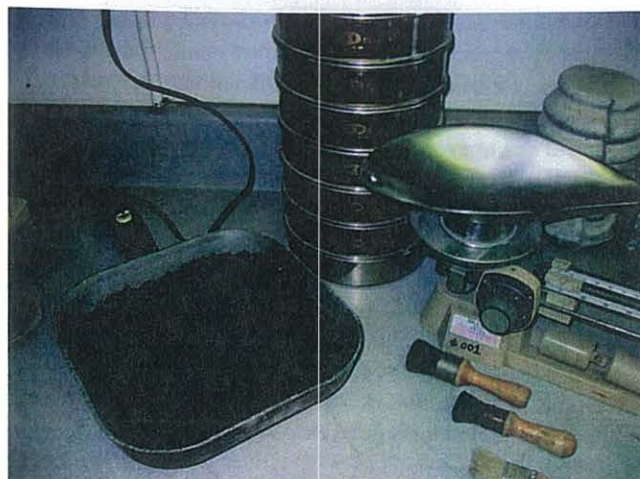


Fig. 3.5—Drying sand on a hotplate to determine total moisture.

suitable for very coarse fine aggregates or for some very angular fine aggregates.

3.2.2.6.2 Surface moisture—To determine surface moisture for either fine or coarse aggregate, a method described in Appendix 5 of this document and in ASTM C566 involves weighing the damp sample, drying it by means of heat, and reweighing it (Fig. 3.5). The surface moisture is calculated by subtracting the established absorption value of the aggregate from total moisture percentage of the damp (stockpile) material. Aggregates that appear to contain no surface moisture should be tested by this method of heating to determine whether surface moisture is either zero or a negative value (indicating that aggregate will absorb mixing water).

A rapid method for determining surface moisture in coarse aggregate involves weighing the damp sample, removing surface moisture by towel-drying to SSD condition, and then reweighing. The moisture loss is divided by the SSD weight to calculate surface moisture.

The surface moisture in fine aggregate can also be determined by the Chapman Flask (ASTM C70). For this test, the specific gravity of SSD aggregate should be known. Several devices using a moisture-reacting powder can be used to provide rapid surface moisture determination of fine aggregate. The accuracy of these tests should be checked daily against the drying methods provided in ASTM C566. Other common methods of determining surface moisture involve weighing the aggregate in air and then under water, or the use of a pycnometer. These methods are described in *Concrete Manual* (U.S. Bureau of Reclamation 1988).

The frequency of testing for moisture content depends on the uniformity of the aggregate supply and the requirements for statistical analysis (Chapter 2). Ordinarily, testing is performed twice a day, and additional tests, particularly of fine aggregates, are made whenever conditions change significantly. To minimize the effect of inaccuracies in sampling and weighing, as large a sample for moisture tests as can be conveniently handled within the time available for drying is used.

3.2.2.7 Tests for specific gravity—To convert a given weight of aggregate into terms of solid volume for computing yield, or to convert solid volume to weight for batching purposes, it is necessary to know the specific gravity of the aggregate. Specific gravity is the ratio of the weight of a given solid volume of material to the weight of an equal volume of water. The specific gravity of most aggregates is approximately 2.65, although some suitable aggregates may have a specific gravity of 2.50 or less, and traprock may have a specific gravity of 2.85 or greater. For a given aggregate, the value is generally constant, and can be considered as such for most purposes. As a routine check, specific gravity should be tested about once a month to confirm that no changes have occurred and any time quarry sources change.

To determine the specific gravity of coarse aggregate (ASTM C127), an SSD sample is weighed in air, and then weighed under water by placing the sample in a wire basket suspended from a scale and immersed in water. Next, the sample is oven-dried and reweighed. The sample should weigh, in pounds, at least 10 times the maximum aggregate size in inches; therefore, a 5 lb (2.3 kg) sample would be the minimum required for 1/2 in (12.5 mm) aggregate. The bulk SSD specific gravity should be calculated as shown in Appendix 5 of this document.

To determine the specific gravity of fine aggregate (ASTM C128), a water-filled calibrated flask is weighed; the same flask filled with water and a known weight of SSD fine aggregate is weighed; the aggregate sample is dried with heat; and the dried sample is reweighed. The specific gravity of fine aggregate can also be determined by weighing the sample in air and then in water, as described for coarse aggregate. Other methods of determining specific gravity are described in *Design and Control of Concrete Mixtures* (Portland Cement Association 2002), including a device developed by the U.S. Bureau of Reclamation that can test for moisture, absorption, and specific gravity.

For SSD aggregates, the specific gravity should be based on the surface-dry condition; for materials that will be batched oven-dry, the specific gravity should be based on the oven-dry condition. For aggregates containing other than these two quantities of moisture, the aggregate weight is expressed in terms of either of these given conditions (plus the excess water) and the corresponding dry or SSD specific gravity is used. Many engineers prefer to base recommended mixture proportions on SSD aggregates. ACI 211.1 uses the oven-dry basis for determining the dry-rodded volume of coarse aggregate. The dry volume is converted to weight with the measured, dry-rodded density, and the SSD weight of coarse aggregate is obtained by adding the amount of water absorbed in 24 hours (ASTM C127). Chapter 6 contains additional information on computing aggregate volume.

3.2.2.8 Tests for voids—The percentage of voids in aggregate usually is computed from the specific gravity and density, lb/ft³ (kg/m³) as follows:

$$\text{percentage of voids} = 100 \left[\frac{(\text{specific gravity} \times 62.4) - \text{bulk density}}{\text{specific gravity} \times 62.4} \right] \text{ (in.-lb)}$$

$$\text{percentage of voids} = 100 \left[\frac{(\text{specific gravity} \times 1000) - \text{bulk density}}{\text{specific gravity} \times 1000} \right] \text{ (SI)}$$

When reporting the amount of voids, information about the aggregate's moisture content and compactness should be provided. For dry-rodded aggregate, the method of determining voids is described in ASTM C29/C29M. A previously used method of determining voids by measuring the amount of water required to fill a container of aggregate is subject to error caused by entrapped air.

3.2.2.9 Tests for bulk density—The bulk density, or weight per unit volume, of aggregate is used to compute the amount of voids in aggregate, to compute proportions of materials, and to convert bulk-volume quantities to weight, or vice versa.

The bulk density of a given type of aggregate varies with the degree of compaction and with the moisture content. In fine aggregate, the bulking, or fluffing apart, of particles by films of surface moisture may reduce the density as much as 25%.

To provide a uniformly reproducible basis of measurement, ASTM has adopted a standard test method for aggregate bulk density (ASTM C29/C29M). This test consists of compacting the dry aggregate into a cylindrical container of calibrated volume and then weighing the aggregate. In one method of compaction, the container is filled in three layers, each layer rodded 25 times; the excess aggregate is then struck off level with the top of the container. The standard container sizes are:

Maximum size of aggregate	Container		
	Capacity, ft ³	Inside diameter, in.	Inside height, in.
1 in. or less	1/3	8.0	11.5
1-1/2 in.	1/2	10.0	11.0
4 in.	1	14.0	11.2

Note: 1 in. = 25.4 mm and 1 ft³ = 0.0283 m³.

For aggregate of maximum size greater than 1-1/2 in. (37.5 mm), the layers of aggregate are compacted by jiggling the container instead of by rodding. Although not specifically stated in the standard method, the term "dry" should be interpreted as either SSD or oven-dry, depending on the basis of mixture computation used.

ASTM C29/C29M also provides for measurement of the density of dry loose or damp loose aggregate. This density is determined by filling the standard container heaping full in one layer. The excess aggregate is struck off level with the top of the container without downward pressure, and the aggregate is weighed.

Lightweight aggregate's bulk density should be determined by the shoveling procedure described in ASTM C29/C29M. The aggregate is tested in an oven-dry condition.

Values of aggregate bulk density are generally considered to be accurate within 0.5 to 1%, although it is possible to make bulk density tests with greater accuracy. Aggregate bulk density should be tested for when mixture proportions are made on the bulk density basis and whenever it is apparent that the type or grading of the aggregate has changed.

3.2.2.10 Bulking factor—Except when using volumetric batching and mixing equipment conforming to ASTM C685/C685M, batching of aggregates by volume should be avoided because large fluctuations can occur (Portland Cement Association 1994). When volume batching is unavoidable, contract documents should state whether the basis of measurement is damp loose, dry loose, or dry-rodded volume. To compute the number of cubic feet (cubic meter) of damp loose aggregate corresponding to 1 ft³ (1 m³) of dry-rodded aggregate—a ratio called the bulking factor—the density should be determined under both conditions and the moisture content of the damp aggregate

$$\text{bulking factor} = \frac{\text{density of surface-dry rodded aggregate}}{\text{density of damp loose aggregate} - \frac{\text{weight of surface moisture in unit}}{\text{volume of damp loose aggregate}}}$$

3.2.2.11 Undesirable substances and properties—Contract documents may require ASTM tests to check for undesirable substances and properties of aggregates. These tests include C40, C88, C123 (coal and lignite), C131, C142, C227, C289, C295, C535, C586, C666/C666M, and D4791. Substances in aggregate that have a deleterious effect on concrete quality are discussed in detail in ASTM STP 169-C (ASTM International 1994).

Cherts, clayey limestones, and some other aggregates are subject to considerable volume changes during wetting and drying or freezing and thawing. Aggregates susceptible to spalling can be detected by petrographic examination supplemented by visual inspection, sorting, and weighing. Often, these materials also cause high losses in the sodium or magnesium sulfate soundness test and in freezing-and-thawing tests.

Because lightweight chert (specific gravity less than 2.40) contributes to popouts at concrete surfaces, ASTM C33 limits the material in coarse aggregate to 3 to 8%, depending on exposure conditions. Flat or elongated particles also are considered undesirable under some conditions, and can be sorted out of a sample and weighed to determine the percentage. Petrographic examination of aggregates can identify elements that are undesirably reactive with alkalis in cement, such as opal, chalcedony, tridymite, and acid or intermediate volcanic glasses.

Lightweight materials and soft and weak materials usually reduce concrete strength and quality. They can quickly be separated by flotation in a heavy liquid, as described in ASTM C123. Most deleterious substances adversely affecting the chemical activity (setting) of cement are organic and can be detected by the colorimetric test.

3.3—Water

Potable water is the quality criterion typically specified for mixing water. Ordinarily, the presence of harmful impurities such as alkalis, acids, decayed vegetable matter, oil, sewage, or excessive amounts of silt will be known. Water of questionable quality should be tested for its effect on strength and time of set versus water known to be satisfactory using test methods from ASTM C109/C109M and C191. ASTM C94/C94M allows the use of mixer wash water in subsequent

batches of concrete if the water complies with optional chemical limits, as confirmed through regular testing of the plant's wash water. This water must also be accurately measured and calculated as part of the allowable mixing water.

Information on water quality requirements are discussed in detail in ASTM STP 169-C (ASTM International 1994).

3.4—Admixtures

The standard specification for chemical admixtures for concrete (water-reducing, retarding, and accelerating) is ASTM C494/C494M. This specification considers seven types of admixtures with various purposes:

- Type A: Water-reducing;
- Type B: Retarding;
- Type C: Accelerating;
- Type D: Water-reducing and retarding;
- Type E: Water-reducing and accelerating;
- Type F: Water-reducing, high-range; and
- Type G: Water-reducing, high-range, and retarding.

In addition, ASTM C1017/C1017M covers chemical admixtures for use in producing flowing concrete, and ASTM C260 gives standard specifications for air-entraining admixtures.

These standards provide methods of testing in the laboratory, but the tests are not intended to simulate job conditions. It is important that mixture proportions be prepared with the admixtures and cement to be used on the job. The inspector should verify that admixtures used on the job comply with all contract requirements and that proper storage and dispensing is provided. For example, contract documents should require that calcium chloride (if allowed) be dissolved before being added to the batch to ensure uniform distribution and uniform acceleration of hydration throughout the mixture. Substituting chemical admixture brands, even though complying with ASTM general types, should not be permitted without verification of performance, because some chemical admixtures perform differently in combination and with different cements and/or mineral admixtures.

As a part of the acceptance of admixtures, either a laboratory analysis should be performed or the manufacturer's statements should be relied upon. Admixtures should be inspected to ensure that they:

- Conform to appropriate specifications;
- Are stored without contamination or deterioration;
- Are accurately measured;
- Are introduced into the batch as specified; and
- Perform as expected as far as can be determined from the concrete as mixed and tested.

If possible, make routine quality-control tests of admixtures for specific gravity, pH, and solids (residue by drying) and compare against data supplied by the manufacturer.

3.5—Steel reinforcement

Steel for concrete reinforcement is usually purchased under one of the ASTM specifications listed in Chapter 22. (Refer to Chapter 17 for a discussion of prestressing steel.) Standard practice for the purchase and handling of reinforcement is given in *Manual of Standard Practice* (Concrete Reinforcing

PURPOSES OF ADMIXTURES

ACI 116R defines an admixture as "a material other than water, aggregates, hydraulic cement, and fiber reinforcement, used as an ingredient of concrete or mortar, and added to the batch immediately before or during its mixing." Contract documents may require or permit an admixture to be used in concrete for one or more of the following purposes:

- Increase workability without increasing water content, or decrease the water content at the same workability;
- Accelerate the rate of early strength development;
- Increase strength;
- Increase surface hardness;
- Retard or accelerate initial setting;
- Retard or reduce heat evolution;
- Modify the rate of or capacity for bleeding;
- Increase durability or resistance to severe exposure conditions, including application of deicing salts;
- Control expansion caused by reaction of alkalies with certain aggregate constituents;
- Decrease capillary flow of water;
- Decrease permeability to liquids;
- Produce cellular concrete;
- Improve penetration and pumpability of grouts and pumpability of concrete;
- Reduce or prevent settlement, or create slight expansion in concrete or mortar used for filling blockouts under machinery, column and girder spaces, post-tensioning cable ducts, and voids in preplaced aggregate;
- Increase bond of concrete to steel;
- Increase bond between old and new concrete;
- Produce colored concrete or mortar;
- Produce fungicidal, germicidal, and insecticidal properties in concrete or mortar;
- Inhibit corrosion of embedded corrodible metal; and
- Decrease the unit cost of concrete.

Refer to ACI 212.3R and 212.4R for more detailed information.

Steel Institute 1997b). Procedures for inspection of cutting, bending, storing, handling, and placing of reinforcement are described in Chapter 8.

In general, purchase specifications cover the method of manufacture, certain chemical requirements, tests in tension and in bending, surface finish, coating for corrosion protection, marking (for size, grade, and point of origin), and permissible variations in weight. Reinforcement is usually tested for properties at the mill and shipped to the job site in bundles marked with tags. The inspector should verify that each shipment has been tested and certified, that the specified grade has been delivered, and that the reinforcement is not damaged or excessively rusted. If it appears that the reinforcement does not meet the contract requirements, it should not be accepted unless samples sent to a laboratory for testing indicate compliance.

A light film of rust is not objectionable on ordinary reinforcement (in fact, its roughness improves the bond), but a heavy coating with flakes or scales that fly off when the bar is bent or struck with a hammer should be removed. Reinforcement should be clean, and any oil or mortar that has been spilled on it should be cleaned off. Epoxy-coated reinforcement should be produced and installed as provided by ASTM A775/A775M.

Occasionally, Grade 60 or 75 (Grade 420 or 520) high-strength reinforcement will crack or break when bent, especially in cold weather. Any cracked and broken reinforcement should be rejected.

3.6—Curing compounds

Membrane-forming concrete curing compounds are usually used in place of water curing or to provide final curing following a short period of water curing. Clear (with or without a fugitive dye) and pigmented (white or gray) products are available. ASTM C309 provides standard specifications and references for testing methods for curing compounds.

To ensure compliance of curing compounds with contract documents, acceptance should be based on certification by manufacturers or on laboratory testing. Inspection consists of seeing that the compound is:

- Properly labeled and not contaminated, diluted, or altered in any way before application;
- Mixed thoroughly before and during application;
- Applied when concrete surfaces are still damp and full of moisture (formed concrete should be saturated with water before compound application);
- Applied at the specified coverage rate; and
- Kept on the surface for the specified curing time.

See additional discussions on curing in Chapters 10 and 12.

3.7—Joint materials

Joints create openings that should usually be filled or sealed to prevent intrusion of dirt, water, or other unwanted substances. For many years, oil-based mastics or bituminous compounds and metallic materials were the only joint sealants available, and fillers were typically resilient materials such as fiberboard, wood, rubber, or cork. All of these materials are still available today and are used in some instances.

To overcome some of the drawbacks of traditional joint sealants, many new elastomeric materials have been developed in recent years. These materials are flexible rather than rigid at normal service temperatures, and they can be molded in the field or preformed. ACI 504R lists the many types of elastomeric materials and their properties.

The manufacturer will normally test its joint materials to verify performance. The inspector should review the manufacturer's data for conformance to the project specifications or that the material is listed as approved in the project specifications. Once they arrive at the job site, materials should be checked again to see that they are not damaged or contaminated and are properly labeled, stored, prepared, and installed. If laboratory testing is required, representative samples should be prepared to ship to the laboratory.

Instructions for sampling liquid and dry components of a job-mixed mastic joint filler are given in *Concrete Manual* (U.S. Bureau of Reclamation 1988). The liquid component should be thoroughly mixed before taking a sample because the liquid is susceptible to separation. Samples of dry components should be obtained by using a sample splitter or by quartering (ASTM C702). Samples should be shipped in tightly sealed containers.

The successful performance of any joint sealant largely depends on proper installation. Each step in the construction and preparation of the joint to receive the sealant requires careful workmanship and thorough inspection. The contract documents for the work should state the type of sealant, the installation method, and requirements for joint construction and preparation. Each joint should be inspected for cleanliness and dryness before placement of backup materials, primers, or sealants. In the absence of specified temperature restrictions, installation of sealants at temperatures above 90 °F (32 °C) and below 40 °F (4 °C) should be avoided.

APPENDIX 5—AGGREGATE SAMPLING AND TESTING

Aggregate quality should be controlled at the point of production using a proper system of quality-control testing. As a check of quality, inspectors can readily perform the following tests in the laboratory.

These are only summaries of sampling and testing procedures; for complete descriptions, refer to Volume 04.02 of ASTM Standards.

A5.1—Sieve or screen analysis of fine and coarse aggregate: ASTM C136

1. Sample aggregates in accordance with appropriate method given in ASTM D75;
2. Split sample by quartering or using a sample splitter;
3. Use minimum test-sample weights per ASTM C 136;
4. Dry the sample to a constant weight;
5. Weigh to the nearest 0.005 oz (0.1 g) for fine aggregate and 0.1% for coarse aggregate;
6. Place the sample over nest of sieves or screens arranged with decreasing size of openings from top to bottom;
7. Shake the sieves until not more than 1% of the weight of the residue on any individual sieve will pass the sieve during 1 minute of continuous hand sieving;
8. Weigh the amounts retained on each sieve or screen and record totals on a worksheet similar to that shown in Chapter 20;
9. Calculate the percentage on the basis of the total weight of the sample, including any material finer than the No. 200 (75 μ m) sieve determined in accordance with ASTM C117; and
10. Calculate the fineness modulus by adding the cumulative percentages retained on the appropriate set of the following sieves and dividing by 100: No. 100 (150 μ m), No. 50 (300 μ m), No. 30 (600 μ m), No. 16 (1.18 mm), No. 8 (2.36 mm), No. 4 (4.75 mm), 3/8 in. (9.5 mm), 3/4 in. (19.0 mm), 1-1/2 in. (37.5 mm), and larger, increasing in a ratio of 2 to 1.

A5.2—Sampling aggregates: ASTM D75

1. *Sample size*—Minimum composite sample sizes depend on nominal size, as in Table A5.1;
2. *Sampling from conveyor belts*—(a) Secure a minimum of three samples of approximately equal increments selected at random from the entire belt width of a stopped conveyor; (b) place two vertical templates, conforming to the belt, so that the material contained between them will yield the required weight; and (c) scoop all of the aggregate between

Table A5.1

Maximum nominal size of aggregate	Approximated minimum mass of field samples, lb (kg)
No. 8 (2.36 mm)	25 (10)
No. 4 (4.75 mm)	25 (10)
3/8 in. (9.5 mm)	25 (10)
1/2 in. (12.5 mm)	35 (15)
3/4 in. (19.0 mm)	55 (25)
1 in. (25.0 mm)	110 (50)
1-1/2 in. (37.5 mm)	165 (75)
2 in. (50 mm)	220 (100)
2-1/2 in. (63 mm)	275 (125)
3 in. (75 mm)	330 (150)
3-1/2 in. (90 mm)	385 (175)

the templates, and combine with the other samples as required in Step (a);

3. *Sampling from bins or belt discharge*—(a) Secure a minimum of three samples of approximately equal increments selected at random from the unit being sampled, and combine to form the field sample; (b) obtain each sample from the entire cross section of material as it is being discharged; and (c) use a pan supported on rails or a similar device capable of intercepting the entire discharge stream and holding the material without overflowing; and

4. *Sampling from stockpiles*—(a) Avoid sampling from stockpiles if possible; (b) to sample from the face of a stockpile, insert a sample shield of rigid material into the face to prevent material from segregating as it is being sampled; and (c) sample at least three increments from the top third, at the midpoint, and at the bottom third of the volume of the stockpile. Select sample units at random, and combine to form a field sample as required.

A5.3—Materials finer than No. 200 (75 μ m) sieve: ASTM C117

Prepare a moist sample by mixing the sample and reducing it by splitting or quartering. The minimum test sample after drying to a constant weight is shown in Table A5.2;

2. Place the sample in a container of sufficient size so that it can be covered and agitated vigorously without loss;

3. Agitate the sample with sufficient vigor to completely separate all particles finer than the No. 200 (75 μ m) sieve from the coarse particles and to bring the finer particles into suspension;

4. Immediately pour the wash water containing the suspended solids over the nest of two sieves, with No. 16 (1.18 mm) on top and No. 200 (75 μ m) on the bottom;

5. Add a second charge of water to the specimen in the container, and repeat agitation and decantation. Repeat this until the wash water is clear;

6. Return all material retained on the nested sieves by flushing to the container holding the rest of the washed sample;

7. Dry the washed aggregate to constant weight at a temperature of 230 $\pm$ 9 °F (110 $\pm$ 5 °C), and weigh to nearest 0.1% of the weight of the test sample, except if the result is 10% or more; weigh to the nearest whole number; and

Table A5.2—Minimum test sample after drying to constant weight

Nominal aggregate size	Minimum weight
No. 4 (4.75 mm)	0.7 lb (300 g)
3/8 in. (9.5 mm)	2.2 lb (1000 g)
3/4 in. (19.0 mm)	5.5 lb (2500 g)
1-1/2 in. (37.5 mm) or larger	11.0 lb (5000 g)

Table A5.3—Particle sizes and minimum sample weights

Size of particle	Minimum sample weight
No. 4 to 3/8 in. (4.75 to 9.5 mm)	2.2 lb (1000 g)
3/8 to 3/4 in. (9.5 to 19.0 mm)	4.4 lb (2000 g)
3/4 to 1-1/2 in. (19.0 to 37.5 mm)	6.6 lb (3000 g)
Over 1-1/2 in. (37.5 mm)	11.0 lb (5000 g)

Table A5.4

Size of particle making up the sample	Sieve size for removing residue of clay lumps and friable particles
Fine aggregate	No. 20 (850 μ m)
No. 4 to 3/8 in. (4.75 to 9.5 mm)	No. 8 (2.36 mm)
3/8 to 3/4 in. (9.5 to 19.0 mm)	No. 4 (4.75 mm)
3/4 in. to 1-1/2 in. (19.0 to 37.5 mm)	No. 4 (4.75 mm)
Over 1-1/2 in. (37.5 mm)	No. 4 (4.75 mm)

8. Calculate A , the percent passing the No. 200 (75 μ m) sieve, to the nearest 0.1% as follows

$$A = 100 \times \frac{B - C}{B}$$

where B = original dry weight of sample, lb (g); and C = dry weight of sample after washing, lb (g).

A5.4—Clay lumps and friable particles in aggregates: ASTM C142

The aggregate for this test consists of the material remaining after completion of the test for materials finer than the No. 200 (75 μ m) sieve by washing.

1. Dry the aggregate to a constant weight at a temperature of 230 ± 9 °F (110 ± 5 °C);

2. Sieve the dried aggregate to obtain the fine-aggregate portion consisting of particles coarser than the No. 16 (1.18 mm) sieve and smaller than the No. 4 (4.75 mm) sieve. From this, take test samples weighing at least 1 oz (25 g);

3. By sieving, separate the coarse-aggregate portion into test samples of the sizes in Table A5.3;

4. Weigh each test sample and spread it in a thin layer on the bottom of the container, then cover it with water and allow it to soak for 24 ± 4 hours;

5. Break all material that can be broken by squeezing it between thumb and forefinger. Do not use fingernails or press against a hard surface. After breaking all discernible clay lumps and friable particles, separate the detritus from the rest of the sample by wet-sieving over the sizes shown in Table A5.4;

6. Wet-sieve by passing water over the sample through the sieve while agitating until all undersize material has been removed;

7. Remove the retained particles from the sieve and dry at a constant temperature of 230 ± 9 °F (110 ± 5 °C). Allow the particles to cool and weigh to an accuracy of 0.1% of the original test sample; and

8. Calculate P , the percentage of clay lumps and friable particles in fine aggregate or individual sizes of coarse aggregate, as follows

$$P = 100 \times \frac{W - R}{W}$$

where W = weight of test sample, lb (g); and R = weight of particles retained on the designated sieve after test, lb (g); and

9. For coarse aggregate, the percentage of clay lumps and friable particles is the average based on the percentage of clay and friable particles in each size fraction, weighted in accordance with the grading of the original sample before separation for testing. If the aggregate contains less than 5% of any size specified, that size should be considered to contain the same percentage of clay lumps and friable particles as the next larger or next smaller size.

A5.5—Organic impurities in fine aggregate: ASTM C40

1. Obtain a representative test sample of fine aggregate weighing about 1 lb (450 g);

2. Fill the 12 oz (350 mL) graduated glass bottle to approximately the 4.5 oz (130 mL) level with the sample of fine aggregate;

3. Add a 3% NaOH (lye) solution until the volume of the fine-aggregate solution after shaking is approximately 7 oz (200 mL);

4. Stopper the bottle, shake vigorously, and then allow to stand for 24 hours.

5. At the end of 24 hours, hold the test sample and a Gardner color scale or a reference color solution up to a light source and compare the color of the liquid in the bottle to the reference color plates on the Gardner scale. Record the color of the liquid with respect to the reference color plates on the Gardner scale; and

6. If the color of the liquid in the test bottle is darker than that of the No. 3 plate on the Gardner color scale, the fine aggregate being tested may contain deleterious organic compounds, and further tests may be required.

A5.6—Specific gravity and absorption of coarse aggregate: ASTM C127

1. Select the minimum test sample size from the sample by splitting or quartering. Remove all material passing the No. 4 (4.75 mm) sieve;

2. Thoroughly wash the test sample and dry to a constant weight at a temperature of 230 ± 9 °F (110 ± 5 °C). Cool in air at room temperature, and then immerse in water at room temperature for 24 ± 4 hours;

3. Remove the specimen from the water and roll it in a large, absorbent cloth until all visible films of water are removed. Weigh the specimen in this SSD condition to the nearest 0.02 oz (0.5 g) or 0.05% of the sample weight, whichever is greater;

4. After weighing, immediately place the SSD specimen in a sample container (wire basket of No. 6 [3.35 mm] or finer mesh of approximately 1 to 2 gallons [4 to 7 L]) and weigh it in water at $73.4 \pm 3^\circ\text{F}$ ($23 \pm 2^\circ\text{C}$). The container should be suspended in water, shaken to remove entrapped air, and supported on a thin wire;

5. Oven-dry the specimen to a constant weight at a temperature of $230 \pm 9^\circ\text{F}$ ($110 \pm 5^\circ\text{C}$), cool it at room temperature for 1 to 3 hours, and weigh;

6. Calculate the bulk specific gravity on the SSD basis

$$\text{bulk specific gravity (SSD)} = B/(B - C)$$

where B = weight of SSD specimen in air, lb (g); and C = weight of the saturated specimen in water, lb (g); and

7. Calculate the percentage absorption as follows

$$\text{absorption, percent} = 100 (B - A)/A$$

where A = weight of oven-dry specimen in air, lb (g).

A5.7—Specific gravity and absorption of fine aggregate: ASTM C128

1. Select, by splitting or quartering, approximately 2.2 lb (1 kg) of fine aggregate from the sample;

2. Dry the sample in a suitable pan to a constant weight at a temperature of $230 \pm 9^\circ\text{F}$ ($110 \pm 5^\circ\text{C}$);

3. Allow the sample to cool, cover with water, and let stand for 24 ± 4 hours;

4. Decant excess water, then spread the sample on a flat surface exposed to warm, circulating air to ensure uniform drying. Continue until the specimen is in a free-flowing condition;

5. To check dryness, place a portion of the sample in a conical mold held firmly on a smooth, nonabsorbent surface with the large diameter down. Lightly tamp the aggregate surface 25 times with a metal tamper weighing 12 ± 0.5 oz (340 ± 15 g) and having a flat, circular tamping face approximately 1 in. (25 mm) in diameter, then lift the mold vertically. If surface moisture is still present, the fine aggregate will retain the molded shape. Continue drying with stirring, and test at frequent intervals until the tamped fine aggregate slumps slightly upon removal of the mold. This indicates that it has reached the SSD condition;

6. Immediately introduce 18 ± 0.4 oz (500 ± 10 g) of the SSD fine aggregate into the pycnometer (flask) and fill with water to approximately 90% of capacity. Roll, invert, and agitate the pycnometer to eliminate all air bubbles, and then add water to bring the water level to the calibrated capacity. Keep the temperature at $73.4 \pm 3^\circ\text{F}$ ($23 \pm 2^\circ\text{C}$). Determine the total weight of pycnometer, specimen, and water to nearest 0.005 oz (0.1 g);

Table A5.5—Nominal aggregate size and minimum weight of test sample

Nominal size of aggregate	Minimum weight of test sample
No. 4 (4.75 mm)	1.1 lb (0.5 kg)
3/8 in. (9.5 mm)	3.3 lb (1.5 kg)
1/2 in. (12.5 mm)	4.4 lb (2 kg)
3/4 in. (19.0 mm)	6.6 (3 kg)
1 in. (25.0 mm)	8.8 lb (4 kg)
1-1/2 in. (37.5 mm)	13 lb (6 kg)
2 in. (50 mm)	18 lb (8 kg)
2-1/2 in. (63 mm)	22 lb (10 kg)
3 in. (75 mm)	29 lb (13 kg)
3-1/2 in. (90 mm)	35 lb (16 kg)
4 in. (100 mm)	55 lb (25 kg)
6 in. (150 mm)	110 lb (50 kg)

7. Remove the fine aggregate from the pycnometer, dry to a constant weight at $230 \pm 9^\circ\text{F}$ ($110 \pm 5^\circ\text{C}$), cool in air at room temperature for 1/2 to 1-1/2 hours, and weigh;

8. Weigh the pycnometer filled to its calibration capacity with water at $73.4 \pm 3^\circ\text{F}$ ($23 \pm 2^\circ\text{C}$);

9. Calculate the specific gravity (SSD) of the aggregate as follows

$$\text{bulk specific gravity SSD} = \frac{S}{B + S - C}$$

$$\text{bulk specific gravity dry} = \frac{A}{B + S - C}$$

where A = weight of oven dried specimen in air, lb (g); B = weight of pycnometer filled with water, lb (g); C = weight of pycnometer with specimen and water, lb (g); and S = weight of the SSD specimen, lb (g); and

10. Determine the moisture absorption of the fine aggregate. Immediately weigh the SSD fine aggregate and dry to a constant weight at a temperature of $230 \pm 9^\circ\text{F}$ ($110 \pm 5^\circ\text{C}$). Allow to cool, and weigh to the nearest 0.005 oz (0.1 g). Calculate the percentage of absorption as follows

$$\text{absorption, percent} = 100(S - A)/A$$

where A = weight of oven-dry specimen in air, and S = weight of SSD specimen.

A5.8—Total moisture content of aggregate by drying: ASTM C566

Select a representative sample of the aggregate by splitting or quartering of not less than the weights in Table A5.5;

2. Weigh the moist sample to the nearest 0.1%;

3. Thoroughly dry the sample to a constant weight. If a ventilated oven is used, the temperature should be $230 \pm 9^\circ\text{F}$ ($110 \pm 5^\circ\text{C}$). If a hot plate or heat lamp is used, stir the sample frequently to avoid localized overheating;

4. Allow the dried sample to cool, and weigh to the nearest 0.1%;

5. Calculate P , the total moisture content (percent), of the sample as follows

$$P = 100 (W - D)/D$$

where W = weight of original wet sample, lb (g); and D = weight of dried sample, lb (g); and

6. The surface moisture (free water) is the difference between the total moisture content and the known absorption of the aggregate.

CHAPTER 4—HANDLING AND STORAGE OF MATERIALS

The quality of concrete depends on the quality of the ingredients used, particularly the cement and aggregates. Problems such as high or low yield, low strength, and deterioration often can be traced to poor handling and storage of these materials.

To help ensure consistent, uniform production of high-quality concrete, inspectors need to confirm that the cement, aggregates, and other materials used are stored and handled properly. In addition to the recommendations given herein, comprehensive guidelines for material transport and handling are provided in ACI 304R.

4.1—Cement

4.1.1 Storage and hauling of bulk cement—Bulk cement should be stored in weatherproof, properly ventilated bins to prevent moisture accumulation (Fig. 4.1). The bins should have:

- Smooth interior walls shaped to allow removal of all cement;
- Inverted truncated cones at the bottom;
- Air-diffuser flow pads to facilitate loosening of cement that has settled; and
- A gate and conveyer system for batching, with the conveyor isolated from the elements.

On large jobs, the silos should be periodically emptied and inspected for a buildup of cement, which should be removed before refilling. This inspection should be made whenever a new supplier's cement or a new type of cement is put into an existing silo.

Avoid exposing the cement to air, because moisture in the air causes partial hydration. If cement becomes slightly lumpy during storage, its use may be permitted if most lumps are soft enough to be crushed between the thumb and fingers. If harder lumps exist, they should be removed by screening (provided that the contract documents do not prohibit this practice). If an excessive number of lumps are encountered, the cement should be tested to verify suitability before use. Cement properties to be verified should be confirmed with the engineer, although tests for compressive strength and time-of-set are usually conducted in these circumstances.

Bulk cement is typically transported in tanker trucks, rail cars, or barges. The equipment used to haul cement should be leakproof and free of contaminants. The chutes and boots used for loading transport vehicles should be regularly inspected. To minimize cement loss, the boot should extend into the truck or compartment. Clogged bin vents should be avoided.

4.1.2 Storage of bagged cement—Bagged cement may be used on small projects. To protect the bags from both ground



Fig. 4.1—Storage of bulk cement in weatherproof and properly ventilated bins is essential to prevent moisture accumulation.



Fig. 4.2—Keep aggregate piles clean and separate to minimize segregation.

moisture and the elements, it is best to store them in an enclosed building and on pallets above the floor. If the bags need to be stored outside, watertight coverings should be used.

When removing bagged cement from storage, the oldest cement should be used first. If the cement has developed warehouse set, the bags should be rolled out on a hard surface to restore flowability. Cement with lumps that do not break down easily should be discarded.

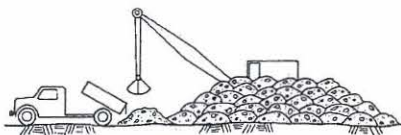
Additional recommendations are given in ACI 304R.

4.2—Aggregates

To achieve uniform concrete production, it is important to keep the grading and moisture content of aggregates as consistent as possible and to protect them from contamination. Figures 4.2 to 4.4 illustrate methods for aggregate handling and storage. Additional recommendations are given in ACI 304R.

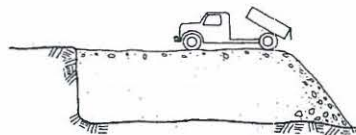
4.2.1 Storage in stockpiles—If aggregates are stored in piles on the ground, the following measures should be taken to prevent contamination and segregation:

- Pave the area, lay planks, or leave the ground covered by a bottom layer of undisturbed aggregate several inches deep;
- Do not allow a crane bucket containing other aggregates or materials to swing over the aggregate pile;



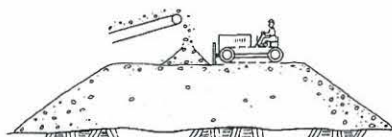
PREFERABLE

CRANE OR OTHER MEANS OF PLACING MATERIAL IN PILE IN UNITS NOT LARGER THAN A TRUCK-LOAD WHICH REMAIN WHERE PLACED AND DO NOT RUN DOWN SLOPE.



OBJECTIONABLE

METHODS WHICH PERMIT THE AGGREGATE TO ROLL DOWN THE SLOPE AS IT IS ADDED TO THE PILE OR PERMIT HAULING EQUIPMENT TO OPERATE OVER THE SAME LEVEL REPEATEDLY.

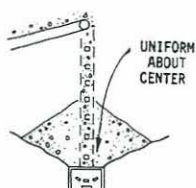


LIMITED ACCEPTABILITY--GENERALLY OBJECTIONABLE

PILE BUILT RADIALLY IN HORIZONTAL LAYERS BY BULLDOZER WORKING FROM MATERIALS AS DROPPED FROM CONVEYOR BELT. A ROCK LADDER MAY BE NEEDED IN SETUP.

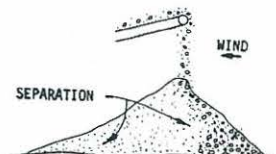


BULLDOZER STACKING PROGRESSIVE LAYERS ON SLOPE NOT FLATTER THAN 3:1. UNLESS MATERIALS STRONGLY RESIST BREAKAGE, THESE METHODS ARE ALSO OBJECTIONABLE.



CORRECT

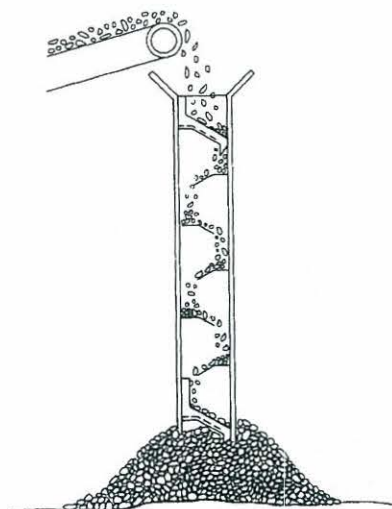
CHIMNEY SURROUNDING MATERIAL FALLING FROM END OF CONVEYOR BELT TO PREVENT WIND FROM SEPARATING FINE AND COARSE MATERIALS. OPENINGS PROVIDED AS REQUIRED TO DISCHARGE MATERIALS AT VARIOUS ELEVATIONS ON THE PILE.



INCORRECT

FREE FALL OF MATERIAL FROM HIGH END OF STACKER PERMITTING WIND TO SEPARATE FINE FROM COARSE MATERIAL.

UNFINISHED OR FINE AGGREGATE STORAGE (DRY MATERIALS)



WHEN STOCKPILING LARGE-SIZED AGGREGATES FROM ELEVATED CONVEYORS, BREAKAGE IS MINIMIZED BY USE OF A ROCK LADDER.

FINISHED AGGREGATE STORAGE

NOTE: IF EXCESSIVE FINES CANNOT BE AVOIDED IN COARSE AGGREGATE FRACTIONS BY STOCKPILING METHODS USED, FINISH SCREENING PRIOR TO TRANSFER TO BATCH PLANT WILL BE REQUIRED.

Fig. 4.3—Correct and incorrect methods of handling and storing aggregates.



Fig. 4.4—Concrete mixing plant and aggregate storage. Bins contain different size aggregates.

- Build up piles of coarse aggregate in layers so that aggregate does not run down the slopes at the edges of the pile, causing segregation;
- Avoid excessive handling, which can cause segregation and degradation;
- Allow ample space between piles or use partitions to separate adjacent piles;
- Do not mix or store aggregates from different sources in the same pile;
- Use each pile of aggregate until it is gone. Separate batching produces more consistent, reliable results than attempts to batch materials blended with clam buckets or bulldozers;
- Do not allow aggregate to freefall from drop heights so great that breakage can occur or the wind can separate fine and coarse materials. If necessary, use baffles or rock ladders to break the fall and prevent excessive segregation and breakage; and
- Reduce segregation of fine aggregate dry enough to be free flowing by dampening.

4.2.1.1 Sand and lightweight aggregates—The requirements for stockpiling of lightweight aggregate are the same as for normalweight aggregate, except that more care should be exercised to prevent crushing of the aggregate. Because lightweight aggregate can be highly absorbent, it may be necessary to continually soak stockpiles for several days before batching to prevent slump loss of concrete.

Washed sand should be allowed to drain as long as necessary to reach a uniform moisture content in accordance with ACI 304R. Because of sequences in stockpiling operations, large sand stockpiles may have widely varying moisture content, leading to difficulty in maintaining uniform concrete slump and water-cement ratio.

4.2.2 Storage in bins—Aggregates of different sizes should be stored in separate bins. When each bin is filled, the aggregate should be dropped vertically into the middle to avoid segregation. If segregation occurs, a baffle, splitter, or rock ladder should be used and the bin kept as full as possible at all times to minimize the tendency for the aggregate to segregate. Bin bottoms should be shaped to facilitate uniform discharge.

4.2.3 Finish screening—Finish screening of aggregate is a process that removes excess fines and adjusts proportions of aggregate sizes to conform to the specified grading. Spray bars are sometimes used in the finish-screening process to enhance removal of fines. When finish screens are used in batch plants, many of the concerns with accurate primary screening, handling, and stockpiling of aggregate can be eliminated.

With finish screening, roughly proportionate amounts of each size of aggregate should be fed to the screens rather than a single size at a time.

4.2.4 Transporting—Vehicles used to transport aggregate should be clean and free of all contaminants, and the vehicles should be inspected for tightness. Aggregate compartments should be inspected to verify no leakage or holes.

4.3—Supplementary cementitious materials

SCMs should be handled and stored in essentially the same manner as cement. Although many SCMs require tighter storage facilities to prevent leakage, most are generally not as susceptible to deterioration from moisture as is portland cement.

4.4—Admixtures

In practice, most chemical admixtures are delivered in liquid form. They should be protected from freezing. If frozen, proper reblending or remixing of the admixture should be done before the admixture is used, and the manufacturer's recommendations should be followed.

Liquid admixtures should be stored in watertight drums or tanks and protected from freezing. Detailed recommendations for storage and handling are given in ACI 212.3R.

CHAPTER 5—FUNDAMENTALS OF CONCRETE

Quality concrete is a mixture of cement (or cementitious materials), water, aggregates, and, in some cases, chemical admixtures that are designed to perform under expected service conditions (Fig. 5.1). Most quality concrete possesses the same principal requirements.

For freshly mixed concrete, those requirements are:

- **Consistency**—The ability to flow;
- **Uniformity**—An homogenous mixture, with evenly dispersed constituents;
- **Workability**—Ease of placing, consolidating, and finishing; and
- **Finishability**—Ease of performing finishing operations to achieve the specified surface characteristics.

For hardened concrete, they are:

- **Strength**—Resists strain or rupture induced by external forces (compressive, flexural, tensile, torsion, and shear);
- **Durability**—Resists weathering, chemical attack, abrasion, and other service conditions;
- **Appearance**—Meets the desired aesthetic characteristics; and
- **Economy**—Performs as intended within a given budget.

By understanding the nature and basic characteristics of concrete, inspectors are better able to see that these requirements are met. This chapter discusses some of the fundamentals of concrete that affect inspection work. For more details on

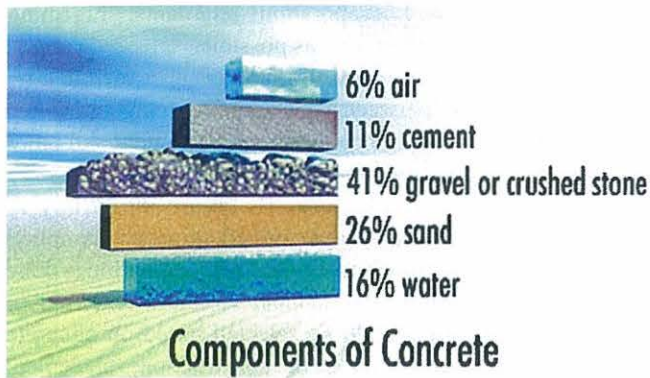


Fig. 5.1—Volumes of components of typical concrete mixture containing an air-entraining agent.

principles and practices that affect concrete quality, refer to the latest editions of ACI 224.1R, ASTM STP 169-C (ASTM International 1994), *Concrete Construction Handbook* (Dobrowolski 1998), *Design and Control of Concrete Mixtures* (Portland Cement Association 2002), *Manual of Standard Practice* (Concrete Reinforcing Steel Institute 1997b) and *Standard Practice for Concrete for Civil Work Structures* (U.S. Army Corps of Engineers 1994).

5.1—Nature of concrete

5.1.1 Components—Concrete has two major components: paste (or binder) and aggregate, with the individual particles of aggregate typically being embedded in and separated by the paste. The paste is a mixture of cement, air, and water, and the aggregate consists of both fine and coarse materials. The total volume of the mixture is equal to the volume of the cement, water, and aggregate plus the volume of air. In many cases, the concrete will also contain SCMs and chemical admixtures.

Air is introduced into concrete during the mixing process. All concrete, no matter how thoroughly consolidated, contains some entrapped air in the form of scattered voids the size of large grains of sand (40 mil [1 mm] or greater). Entrapped air varies with the size of the coarse aggregate. The amount of entrapped air is generally 1.5 to 2% by volume of concrete with 3/4 in. (19.0 mm) aggregate. The volume of entrapped air will increase as the coarse aggregate size decreases, and it will decrease as the size of the coarse aggregate increases.

In addition to entrapped air, concrete may contain entrained air introduced intentionally by use of air-entraining admixtures. The volume of entrained air will remain constant within a range of 1 to 3% regardless of the size of the coarse aggregate. During mixing of the concrete, air bubbles are stabilized and form many spherical voids within the cement paste approximately the size of larger cement grains and finer sands (less than 40 mil [1 mm]). Entrained air is essential to the freezing-and-thawing resistance of concrete. (Refer to the discussion in this chapter on freezing-and-thawing effects.) In addition to improving freezing-and-thawing resistance, entrained air increases the workability and cohesiveness of concrete and retards segregation and bleeding.

5.1.2 Water-cementitious material ratio—The quality of the cement paste contributes most to the compressive strength of concrete. The strength and density of the paste

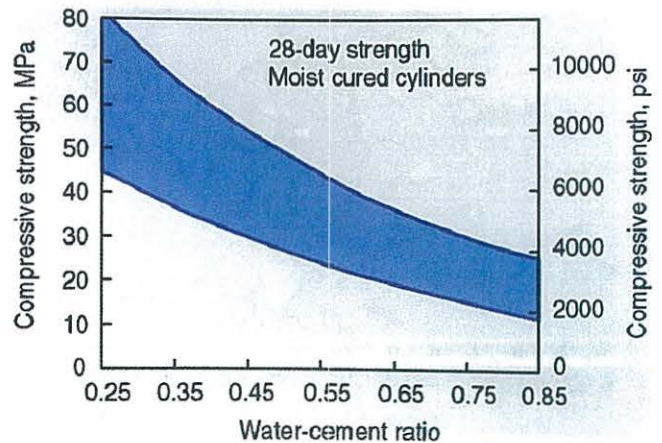


Fig. 5.2—Range of typical strength to water-cement ratio (w/c) relationships of portland-cement concrete on over 100 different concrete mixtures cast between 1985 and 1999. Concrete of lower w/c is stronger and nearly watertight (Portland Cement Association 2002).

depend primarily on the water-cementitious material ratio (w/cm). In determining this ratio, cementitious material is calculated as the combined mass of the cement, fly ash, ground slag, silica fume, and other pozzolans used in the mixture. Higher water contents dilute the paste and weaken the binder (Fig. 5.2). There are several advantages to maintaining a low w/cm in concrete, including increased compressive, flexural, and tensile strengths; lower permeability; increased resistance to weathering; less volume change; and reduced shrinkage cracking. Refer to Chapter 6 for a discussion on proportioning of mixtures for a specified w/cm .

5.2—Fresh concrete

5.2.1 Workability—Fresh concrete needs to be workable during placing and finishing operations. It is important that concrete remains plastic (fluid) throughout the placement without causing segregation or excessive slump loss. Workability depends on the characteristics and quantity of cementitious materials used, w/cm , gradation, shape and surface texture of aggregates, amount of entrained air, and use of chemical admixtures and fibers. The level of workability needed is affected by field conditions, including temperature, haul time, method of placement (conveyor, bucket, or pump), form configuration, and degree of consolidation.

The consistency of concrete is closely related to workability, and is usually measured by the slump test (refer to the discussion in Chapter 7). A high-slump concrete has a wetter consistency than a low-slump concrete.

If the consistency of the concrete is too low, the mixture may appear dry and harsh and be difficult to place and consolidate, resulting in the development of honeycomb (unconsolidated) concrete conditions after hardening. Conversely, a high consistency may lead to segregation, lack of uniformity, lower strengths, and increased permeability of the concrete (unless the high consistency is due to proper use of a high-range water-reducing agent).



Fig. 5.3—Trapped bleed water can cause surface delamination of concrete.

In general, maximum quality for a given concrete mixture will be achieved by placing concrete at the lowest consistency needed to ensure complete consolidation and filling of the form.

5.2.1.1 Bleeding and settlement—In newly placed concrete, the solids (cementitious and aggregate) slowly settle, bringing a layer of water, called bleedwater, to the surface. As a result of settlement, the solids at the bottom of the concrete member become more closely packed (Fig. 5.3), and the space occupied by the hardened concrete is slightly less than when it was freshly placed.

Materials selection will affect the rate and capacity of bleeding in concrete. Chemical admixtures, such as water reducers, may increase or decrease bleeding, depending on the type used. The presence of entrained air provides a more cohesive mixture and alters the settlement process, usually reducing the rate and amount of bleeding. The use of supplementary cementing materials with particle sizes finer than portland cement, such as silica fume, will significantly reduce bleeding.

Bleeding is a normal characteristic of fresh concrete, and will not harm concrete that is properly placed and finished. During hot, dry weather, bleeding may help prevent excessive surface drying and plastic shrinkage cracking. A high amount of bleedwater, however, can delay finishing operations because floating and troweling should never be performed while bleedwater is on the surface. Under severe drying conditions, flatwork surfaces may sufficiently stiffen and subsequently be finished before all bleedwater rises to the surface. This bleedwater will be trapped beneath the hard-troweled finish, resulting in blisters or thin cavities and subsequent scaling of the surface. The use of air-entrained concrete in steel-troweled flatwork also increases the potential for late rising bleedwater being trapped beneath the finished surface, and should be avoided.

The same problem can occur in cold weather when concrete is placed on a cold subgrade or metal deck while the finished surface is exposed to much warmer air temperatures from space heaters in enclosed areas. When placing flatwork under conditions of rapid evaporation and drying, workers can minimize these effects by using windbreaks, shade

covers, fogging, and evaporation retardants. Finishing should also be delayed as long as possible.

5.2.2 Consolidation—Consolidation benefits concrete by reducing the voids between particles, removing unwanted entrapped air, improving bond with reinforcement and other embedded items, and reducing the amount of paste necessary to provide a workable mixture. To ensure uniformity in concrete, consolidation is performed through tamping, rodding, or mechanical vibration. Most consolidation is done using power screeds or mechanical vibrators (external or internal). Equipment should be selected based on concrete mixture characteristics, type of construction, placement method, member size, and reinforcement and formwork conditions. It is important to make sure consolidation is performed correctly. Vibrators should not be held in one place for too long or used to move plastic concrete. Improper consolidation can lead to segregation and loss of entrained air in concrete. Refer to Chapter 9 for more information on proper vibration techniques.

5.2.3 Hydration, setting, and hardening—Hydration is a chemical reaction between cement and water that forms new compounds in concrete. Portland cement consists primarily of four compounds: tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium aluminoferrite (C_4AF). C_3S and C_2S are responsible for early strength gain and late strength development in concrete, respectively. C_3A contributes to the heat of hydration, and C_4AF primarily governs the concrete's color. All types of portland cement contain the same four compounds, but in varying proportions. During hydration, the calcium silicates, which make up approximately 75% of the weight of portland cement, react with water, forming two new compounds: calcium hydroxide (CH) and calcium silicate hydrate (C-S-H). The principal reaction product of the hydration process is C-S-H, a gel-like substance responsible for the setting, hardening, and strength properties of the concrete. Hydration of cement continues at a diminishing rate as long as sufficient moisture and adequate temperatures are available. If the paste is not kept moist by curing, cement hydration ceases when the evaporable water escapes from the paste.

The rate of hydration and time of set is also influenced by the w/cm , cement fineness, use of SCMs and chemical admixtures, and temperature of materials.

5.2.4 Heat of hydration—Hydration is accompanied by liberation of heat. Some of the heat generated escapes through the concrete surface, but part of the heat is retained and raises the concrete temperature. Excessive temperature rise can reduce ultimate concrete strength and produce undesirable stresses that cause cracking as the temperature drops. This is particularly true where temperature differentials exist between parts of the concrete mass and where members are partially restrained.

In mass concrete, the heat escapes slowly, and the temperature rise will be somewhat similar to that shown in Fig. 5.4. Temperature rises in mass concrete can be controlled by cooling materials and handling equipment, reducing cement content, and using supplementary cementing materials and chemical retarding admixtures. Other measures include

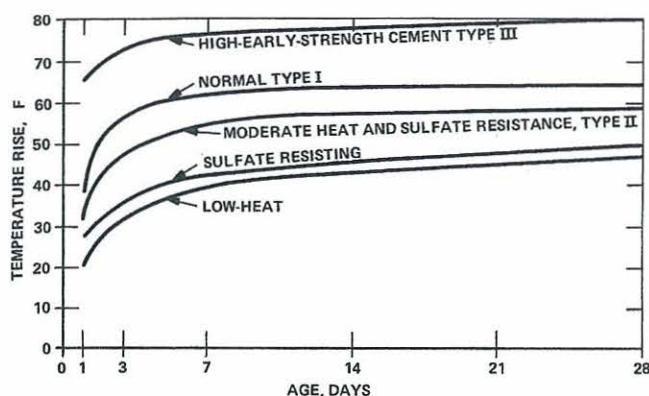


Fig. 5.4—Temperature rise of concrete for various types of cement when no heat is lost (376 lb cement per yd^3).

scheduling placements during cooler periods of the day, placing in lifts, and using embedded pipes with circulating cool water. Refer to Chapter 16 for information on proportioning mass concrete mixtures.

5.3—Hardened concrete

5.3.1 Curing—Curing is essential to the strength development and durability of concrete. Continued cement hydration is dependent on the availability of moisture in concrete, a favorable ambient temperature, and sufficient space for hydration products to form. If the concrete's relative humidity drops to approximately 80% or its temperature falls below freezing, hydration and strength gain will cease. Figure 5.5 shows the relationship between strength gain and moist curing. Adding moisture to dry concrete will resume cement hydration to some degree and continue strength gain. To maximize concrete quality, however, curing should be continuous for the period specified.

Curing is performed by either sealing in the original water in the concrete mixture or by supplying additional water. Chapter 10 provides more detailed information on curing and the protection of concrete during normal, hot, and cold weather conditions.

5.3.2 Strength—Concrete is very strong in compression, but relatively weak in tension. Concrete will crack when loads or stresses due to restraint exceed its tensile strength. Structural reinforcement is used in concrete to transfer the tensile and shear stresses from the concrete to the steel, allowing the concrete to withstand higher levels of stress.

Concrete is often specified to achieve a 28-day compressive strength f'_c expressed in pounds per square inch (psi) or megapascals (MPa). The amount of strength gain depends on cementitious properties (C_3S and C_2S content), w/cm , hydration, and age of the concrete. Under standard laboratory conditions, approximately 50 to 70% of the ultimate compressive strength of concrete (made with Type I cement) will be reached during the first week, and approximately 95% during the first month.

5.3.3 Durability—Although strength is an important characteristic of concrete, other qualities can be even more important for concrete exposed to harsh environments. In general, the principal causes of deterioration in concrete are

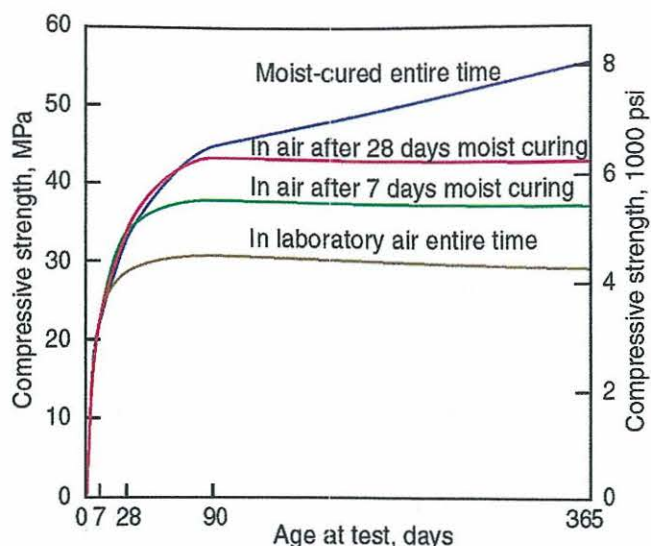


Fig. 5.5—Concrete strength increases with age as long as moisture and favorable temperature are present for hydration of cement (Portland Cement Association 2002).

corrosion of reinforcing steel, exposure to freezing-and-thawing cycles, alkali-silica reaction (ASR), and sulfate attack. All of these problems start with exposure to moisture. Thus, increasing watertightness and reducing the permeability of concrete are the keys to durability. Permeability, or the amount of moisture migration (gas, vapor, and liquid forms) in concrete, is a function of the w/cm , density of the paste, and aggregate gradation.

5.3.4 Chemical attack—Concrete can deteriorate when exposed to aggressive groundwater, acidic rain or condensation, seawater or salt spray, raw sewage, and acids or caustics. The effects vary, but usually include staining, erosion, deterioration of hydration products, cracking, and spalling. Chemicals can also aggressively attack reinforcing steel and other embedded metallic elements.

5.3.4.1 Sulfates—Sulfate attack is one of the most common forms of chemical attack. Sulfates of sodium, potassium, and magnesium, present in alkaline soils and water (rain, groundwater, or seawater), can cause expansion of concrete by reacting with the hydration products in the cement paste. Sodium or potassium sulfates attack calcium hydroxide and form gypsum, which then reacts with calcium aluminate hydrate to form ettringite. Gypsum, however, has a limited solubility, and the rate of attack is relatively slow. Magnesium sulfates can cause more severe deterioration. In addition to reacting with calcium hydroxide and hydrated calcium aluminates, magnesium sulfate will also attack and decompose calcium silicates. Other forms of sulfate attack include salt crystallization and thaumasite. Sulfate salt crystallization causes severe pressure in concrete pores. Thaumasite is very similar to ettringite. The formation of thaumasite eventually results in a softening within the cement matrix, causing disintegration of the concrete. Thaumasite forms in concrete as a result of a reaction between calcium carbonate and calcium silicates with sulfates (van Aardt and Visser 1975). The resistance of

concrete to sulfate attack can be improved by using a sulfate-resisting portland cement with a low C_3A content, such as a Type II cement with moderate resistance, or a Type V cement with high resistance. Resistance to all forms of chemical attack can be improved by reducing the w/cm (0.40 or less).

5.3.4.2 Corrosion—Ideally, concrete will protect embedded reinforcement because of its highly alkaline nature. In a high-pH environment, a dense layer of iron oxide will form on the reinforcement, protecting it from corrosion. If chloride ions are present, however, they can penetrate this protective film. Chloride attack may occur from internal sources, such as contaminated mixing water, aggregates, or the use of a calcium chloride accelerator, or from external sources, such as exposure to seawater or deicing salts. If the protective film is disrupted, the steel will corrode if given sufficient moisture and oxygen. The corrosion product formed is rust, which expands to take up approximately four times its original volume. This increase in volume causes internal stresses to build within the concrete, resulting in cracking and spalling.

To protect from chloride ingress and corrosion, concrete should be kept watertight and have low permeability. Resistance to corrosion can be increased by using a low w/cm (0.40 or less), using SCMs and corrosion-inhibiting admixtures, providing adequate curing, and increasing the concrete cover (the distance between the exposed concrete surface and the reinforcing steel). Other methods of protection include surface treatments, coating the reinforcement, corrosion-resistant or stainless steel reinforcement, sacrificial anodes, and cathodic protection.

5.3.5 Freezing-and-thawing effects—When concrete is subjected to repeated freezing-and-thawing cycles without protection, damage may result in the form of cracking, scaling, or crumbling. Water expands approximately 9% upon freezing. When concrete is saturated above its critical level (91.7%) and the temperature drops below freezing, the expansion of water produces osmotic and hydraulic pressures that exceed the tensile strength of the paste or aggregates. Failure in the paste occurs when the pressure exceeds the tensile capacity of the capillaries and pores in the paste, causing the cavities to expand and rupture. If porous, low-density aggregates are used in the concrete, water can accumulate in the stone, and resulting pressures from freezing and expansion will burst the aggregate and surrounding paste. If the deleterious aggregate is located near the surface of the concrete, popouts can result.

The use of air-entraining admixtures greatly improves the resistance of concrete to damage from freezing-and-thawing cycles. Entrained air protects concrete by providing pressure-relief chambers comprised of many small, closely spaced air voids. For adequate durability, the following air-system characteristics are recommended:

- Sufficient volume (Table 5.1);
- A specific surface area of $600 \text{ in.}^2/\text{in.}^3$ ($24 \text{ mm}^2/\text{mm}^3$) or greater; and
- A calculated spacing factor between air voids of less than 8 mil (0.2 mm).

Table 5.1—Recommended average air content, %

Exposure	Nominal maximum aggregate size					
	3/8 in. (9.5 mm)	1/2 in. (12.5 mm)	3/4 in. (19 mm)	1 in. (25 mm)	1.5 in. (37.5 mm)	2 in. (50 mm)
Mild	4.5	4.0	3.5	3.0	2.5	2.0
Moderate	6.0	5.5	5.0	4.5	4.5	4.0
Severe	7.5	7.0	6.0	6.0	5.5	5.0

Current field practices measure only the volume of air in fresh concrete, and do not include air-void size or spacing. Spacing and air-void size can be determined in fresh concrete by an air-void analyzer test, and in hardened concrete from petrographic analysis (ASTM C457). The recommended air contents for frost-resistant concrete contained in ACI 201.2R provide about 9% of air in the mortar fraction for severe exposure and about 7% of air for moderate exposure.

The amount of air required in concrete for each level of exposure and nominal maximum aggregate size is shown below. ACI 301 and 318 allow a 1% reduction from Table 5.1 values for concrete with strength exceeding 5000 psi (35 MPa). Refer to Chapters 6 and 7 for a discussion of other aspects of air entrainment.

5.3.6 Alkali-aggregate reactivity—Expansive reactions can occur in concrete between highly alkaline environments and reactive mineral aggregates. The two most common are alkali-carbonate reaction (ACR) and alkali-silica reaction (ASR).

ACR is related to a de-dolomitisation, or the breakdown of dolostones and dolomitic limestones, and the associated expansion of the coarse-aggregate particles.

ASR is a serious, but rare, variety of alkali-aggregate reactivity. ASR is associated with the dissolution of silica (SiO_2) in the aggregate, which reacts with the alkali hydroxides in the pore solution released during cement hydration. The reaction product of ASR is the formation of an alkali-silica gel in the aggregate and surrounding cement paste. This gel has the capacity to swell from absorbing additional moisture, causing cracking, spalling, and displacement in hardened concrete. For ASR to occur, there must be reactive silica in the aggregate, a sufficient amount of alkalis in the concrete pore solution, and sufficient moisture. To control ASR, nonreactive aggregates should be used when possible, and the total alkali content of the concrete should be limited. Use of SCMs (pozzolans) in quantities proven to control ASR can allow the use of locally available aggregates deemed to be potentially reactive.

5.3.7 Volume changes—Concrete volume changes over time due to changes in moisture, temperature levels, and stress. The volume of concrete, when initially hardened, is the largest it will ever be during the life of the concrete. Following initial installation and curing, concrete that is allowed to dry will generally lose volume until an equilibrium state of moisture is reached. Most of the volume change that develops in concrete is due to drying shrinkage of the paste, which occurs as excess water not needed for hydration evaporates. To a lesser extent, concrete will also lose volume due to autogeneous shrinkage of the paste as cement hydration progresses. Many factors influence the degree of drying

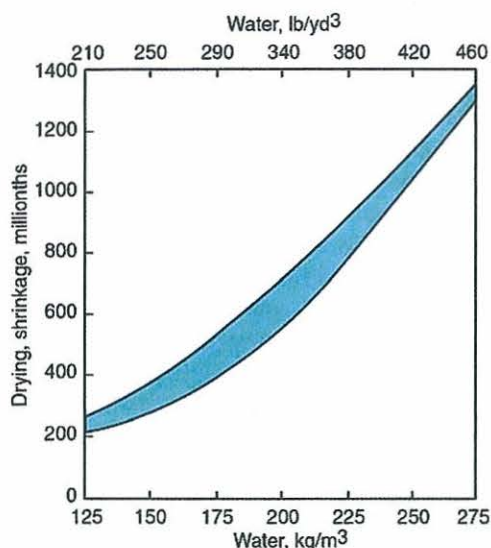


Fig. 5.6—Relationships between total water content and drying shrinkage of concrete. A large number of mixtures with various proportions are represented within the shaded area of the curves. Drying shrinkage increases with increasing water contents (Portland Cement Association 2002).

shrinkage that develops, including w/cm , type of cementitious products employed, choice of aggregates, method of curing, temperature, and time. Generally, keeping the water content as low as possible is the largest factor in limiting the shrinkage that will occur for a given concrete mixture. Typically, concrete mixtures will shrink approximately 2/3 in. per 100 ft (60 mm per 100 m), or approximately 0.06% (ACI 224.1R). Figure 5.6 plots correlations between total water content, cement content, and projected shrinkage of hardened concrete for different mixtures made with the same materials. Aggregate quality, source, or both can have major influence on a given mixture's resistance to shrinkage of the cement paste.

When concrete is restrained by a subgrade, reinforcement, or connections with other members, it may crack as a result of drying shrinkage stress or the combined effects of thermal and drying stresses. If the surface of the hardened concrete dries more rapidly than the interior, differential stresses develop that may cause a network of shallow plastic shrinkage cracks to form on the surface. Unequal drying of opposite surfaces of slabs can cause curling, most noticeably at edges and corners.

When concrete is subjected to external loads, it will deform elastically. If the load is sustained over time, the stress will cause creep (permanent deformation) to develop. The amount of creep that develops depends on the ratio of sustained stress to concrete strength, the humidity of the environment, the dimensions of the element, and the composition of the concrete.

Volume changes in concrete due to thermal stresses can also have a great impact on a structure's behavior. Concrete expands when heated and contracts when cooled. Restraint of this thermal contraction can cause cracking. A temperature change of 100 °F (56 °C) will induce a volume change of approximately 0.06% when calculated from a general coefficient of thermal expansion of 0.0006 per 100 °F (56 °C)

(similar to the change noted previously for concrete as it dries from the saturated to dry state).

Because volume change, stress, and cracking are inevitable, a number of practices and techniques aimed at minimizing and controlling concrete cracking have evolved. In reinforced concrete construction, the reinforcement employed for load resistance will generally control the development of shrinkage or thermal-induced cracks. In unreinforced concrete or concrete with light reinforcement, such as slabs, joints of proper spacing and depth should be formed or saw-cut as soon as possible after the concrete sets to prevent excessive amounts of random cracks from developing. The use of light reinforcement (mesh) helps to keep cracks that do form tightly closed. In members requiring no structural reinforcement, light temperature steel is often employed simply to control thermal cracking by producing many small, insignificant cracks instead of a few large, objectionable cracks.

CHAPTER 6—PROPORTIONING AND CONTROL OF CONCRETE MIXTURES

The required properties of concrete are governed by its use, and these properties should be reflected in the contract documents for the job. To ensure that the desired properties are achieved, it is necessary for inspectors to see that the appropriate mixture proportions are used (Fig. 6.1).

This chapter presents the factors to consider and general principles to follow when proportioning concrete mixtures. Detailed procedures for mixture proportioning are covered in ACI 211.1, 211.2, and 211.3R. Proportioning of concrete mixtures containing one or more admixtures is discussed in ACI 212.4R. Chapter 16 of this manual details proportioning of special types of concrete, including lightweight, high-density, mass, and shrinkage-compensating.

6.1—Factors to consider

The major factors to consider when proportioning concrete mixtures include:

- Durability and strength (compressive or flexural);
- Placeability;
- Appearance; and
- Setting and strength-gain characteristics.

Durability and strength, as required in the contract documents, dictate the w/cm for given materials. Durability requirements might include specific requirements for high strength, high density, abrasion resistance, or low permeability.

Placeability (the workability and consistency of the concrete) is primarily dictated by the proposed concrete placing methods (buckets, chutes, buggies, pumps, or conveyors), the placing conditions (form depth and dimensions, amount of reinforcement, and accessibility), and the consolidation method (spading or internal or external vibration).

Appearance is an important consideration for architectural concrete. Characteristics to examine include the color and type of the coarse aggregate, color and type of the cement, and proportion of fine-to-coarse aggregate (particularly in the case of exposed-aggregate concrete).

Setting time and strength gain can be controlled by using various combinations of portland cement, blended cements,



Fig. 6.1—A concrete inspector evaluates mixture proportions to ensure that they meet design requirements.

chemical admixtures, and pozzolans. Many of the properties of Types III, IV, and V portland cements can be duplicated by the addition of appropriate SCMs, and admixtures to Types I and II portland cements. Various types of chemical admixtures have been developed to control a wide range of concrete properties, including air content, water reduction, set acceleration and retardation, corrosion resistance, and paste rheology.

6.2—Methods of specifying concrete proportions

Generally, contract documents require that the concrete develops a certain strength at a given age within a limiting range of consistency (slump). A maximum w/cm or minimum cement content is often specified as an additional precaution to ensure the necessary durability, impermeability, or workability.

6.2.1 Strength specifications—In contract documents, the specified compressive strength usually is designated as a minimum strength. Even when the required concrete mixture proportions are used, however, an occasional strength test result falling below this minimum can occur. This may lead to confusion and arguments that result in unnecessary modifications to the mixture proportions or requirements for additional testing.

When evaluating the results of concrete strength tests, consider that concrete production is susceptible to variations in ingredients, batching, and testing. Codes for structural design permit occasional strength test results as much as 500 psi (35 MPa) below the specified (design) strength (up to 10% below for concrete designs greater than 5000 psi [35 MPa]). Chapter 2 of this manual and ACI 318 and 301 discuss the issue of occasional low strength tests.

Because of variability in concrete, as well as variability in test specimens and test results, it is necessary to proportion concrete for an average strength far enough above the specified strength f'_c that all but a small proportion of the concrete will equal or exceed f'_c (ACI 214R). The amount of overdesign

needed for proposed mixtures should be statistically based on the previous record of concrete production or the minimum required by code. Some contract documents require an overdesign of up to 1500 psi (10 MPa) when data from previous records indicate extreme variability in production or when no previous data are available. In the absence of such contract requirements, the mixture proportions should be prepared as discussed in Chapter 2. After collecting a history of field strengths for the established mixture proportions and attaining proper control of field operations, it may be possible to adjust mixture proportions.

For pavement concrete, the same approach is used except that the flexural strength, rather than compressive strength, of standard test beams may be required.

6.2.2 Prescriptive specifications—A specification may prescribe the proportions of materials by one or a combination of the following four methods:

1. A fixed or a minimum amount of cement (and admixture, if specified) per volume of concrete;
2. A fixed or a maximum w/cm ;
3. Fixed proportions of cementitious materials, fine aggregate, coarse aggregate, and admixtures; and
4. Limits on the proportion of fine-to-total aggregate (for example, 35 to 50%).

Methods 1 and 2 are often combined, and are more typical than Methods 3 and 4. The slump is also typically specified. For air-entrained concrete, it is common to specify the range of total air in the concrete mixture. If Methods 1, 2, and 4 are combined, specified values, including strength, should be consistent.

If the specified slump range cannot be obtained with the minimum cementitious materials content and maximum w/cm specified, making the following adjustments can remedy the situation:

- Increase the cementitious materials content;
- Change the aggregate types (from angular to rounded, which will reduce the water requirement);
- Change the relative proportions of fine-to-coarse aggregate; and
- Use a water-reducing admixture.

Note that if building codes or project specifications require compliance with ACI 318, then concrete proportions should also satisfy all ACI 318 requirements, such as those for durability given in Chapter 4 of ACI 318.

6.3—Proportioning for specified strength or w/cm

When proportioning to determine the quantity of cement per unit volume of concrete that will produce hardened concrete of the specified strength and durability (assuming proper curing), the quantity of cement needed depends on:

- Cement type and quality;
- Quantity and quality of SCMs;
- Maximum w/cm or water-cement ratio (w/c);
- Consistency of mixture;
- Use of admixtures, alone or in combination;
- Maximum aggregate size and grading;
- Other characteristics of the aggregate, such as particle shape and surface texture; and

- Rate of strength gain desired.

6.3.1 Cement types—Contract documents typically state the type of cement to be used. For ordinary purposes, a normal portland cement (ASTM C150, Type I or ASTM C1157, Type GU) or a blended cement (ASTM C595 or C1157) may be required. Special situations may call for the use of moderate-heat (Type II), high-early-strength (Type III), or high-sulfate-resisting (Type V) cements. When the choice of cement is other than Type I or III, considerations other than strength often dictate the type to use. Nevertheless, a minimum concrete strength at a specified age typically is required. (Refer to the discussion on cement in Chapter 3.)

6.4—Concrete with supplementary cementitious materials

To properly assess the advantages and disadvantages of using SCMs (refer to discussion on SCMs in Chapter 3), laboratory studies of proposed concrete mixtures using actual job materials should be conducted. SCMs generally reduce the rate of early strength development, so concrete strengths at early ages may be less than those of comparable mixtures containing only portland cement at the same water content. Strengths at later ages, however, will be nearly equal or higher. Class N pozzolans may increase mixing water requirements, and thus, at least theoretically, drying shrinkage, but this is not evident in field structures. Class F (fly ash) pozzolan, on the other hand, usually reduces mixing water requirements.

SCMs have proven to be effective in reducing ASR and sulfate attack in concrete. The actual effects of SCMs intended for use can be checked against control mixtures made using 100% portland cement. Detailed information on the use of SCMs in concrete is given in *Concrete Manual* (U.S. Bureau of Reclamation 1988), *Standard Practice for Concrete for Civil Work Structures* (U.S. Army Corps of Engineers 1994), and *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete* (Malhotra 1992).

6.4.1 Mixture proportioning and control—Because SCMs are often used to produce concrete with unique or enhanced properties, it is desirable to test resulting mixtures for these properties before using them in structures. The recommendations of ACI 211.1 for trial mixture proportioning should be followed using the established specific gravity of each material in calculations.

Generally, the dry mass of portland cement, pozzolans, and other cementitious materials are combined to determine the total cementitious content of a mixture and, subsequently, the w/cm . Because portland cement and fly ash may be similar in color, batching and storage equipment should be carefully identified to differentiate between these materials. Silica fume is sometimes combined with water in a slurry solution to facilitate handling and batching during mixing operations. The amount of water added to the batch as part of the slurry solution should be determined and properly accounted for in calculating the w/cm .

6.4.2 Water-cementitious material ratio—If aggregate of approximately the same grading and maximum size and from the same source is used—and other materials remain

Table 6.1—Relationships between w/cm and compressive strength of concrete

Compressive strength at 28 days,* psi (MPa)	w/cm , by weight	
	Non-air-entrained concrete	Air-entrained concrete
6000 (40)	0.41	—
5000 (35)	0.48	0.40
4000 (28)	0.57	0.48
3000 (21)	0.68	0.59
2000 (14)	0.82	0.74

*Values are estimated average strength for concrete containing not more than percentage of air shown in Table 6.3.4(a) of ACI 211.1. For constant w/cm , the strength of concrete decreases as air content increases.
Note: Relationship assumes maximum aggregate size of approximately 3/4 to 1 in. (19.0 to 25.0 mm); for a given source, strength produced for given w/cm will increase (as will cement content) as maximum aggregate size decreases. Refer to Sections 3.5 and 6.3.2 of ACI 211.1.

the same—the potential compressive strength of concrete is nearly constant when the w/cm is held constant. If, however, different aggregates and different gradings (particularly the maximum aggregate size) are used, the necessary w/cm for a desired slump and strength will vary somewhat, and it may change substantially when admixtures are used. Nevertheless, average relationships of w/cm and strength usually are sufficient for proportioning a trial or starting mixture.

The necessary w/cm should be estimated from a published table of w/cm and corresponding strength values. Table 6.1 (adapted from ACI 211.1) gives approximate strengths for different w/cm for air-entrained and non-air-entrained concrete. Strengths obtained from these w/cm will generally be conservative. If there is a history of usage of given materials, a review of available test information can provide useful data, particularly as to the water requirement and strength level of the concrete.

6.4.3 Aggregate selection—After determining the w/cm , the next step is to find the aggregate proportions that will provide a workable mixture with a minimum amount of paste.

The nominal maximum aggregate size is usually limited by placing conditions, dimensions of structural members, and spacing of reinforcing steel. Well-graded and well-shaped aggregate of large maximum size has less total volume of voids than smaller aggregate. Therefore, concretes made with larger aggregates that are properly graded and shaped require less mortar and, thus, less water, per unit volume (Fig. 6.2). Generally, for average strength requirements, the largest maximum aggregate size that is most economical and consistent with the dimensions of the structural elements and placing conditions should be used. When high-strength concrete (8000 psi [55 MPa] or greater) is desired, best results are usually obtained with smaller maximum aggregate sizes of 3/4 or 1/2 in. (19.0 or 12.5 mm).

When conducting mixture proportioning studies and concrete tests to evaluate materials, the inspector should assess the properties of mixtures having various blends of fine and coarse aggregate. The most favorable combination of fine and coarse aggregates should be selected that provides the required concrete properties such as workability and strength. To enhance concrete finishability and pumpability, the amount of pea gravel or flat and elongated small coarse aggregate should be minimized. Also, the percentage

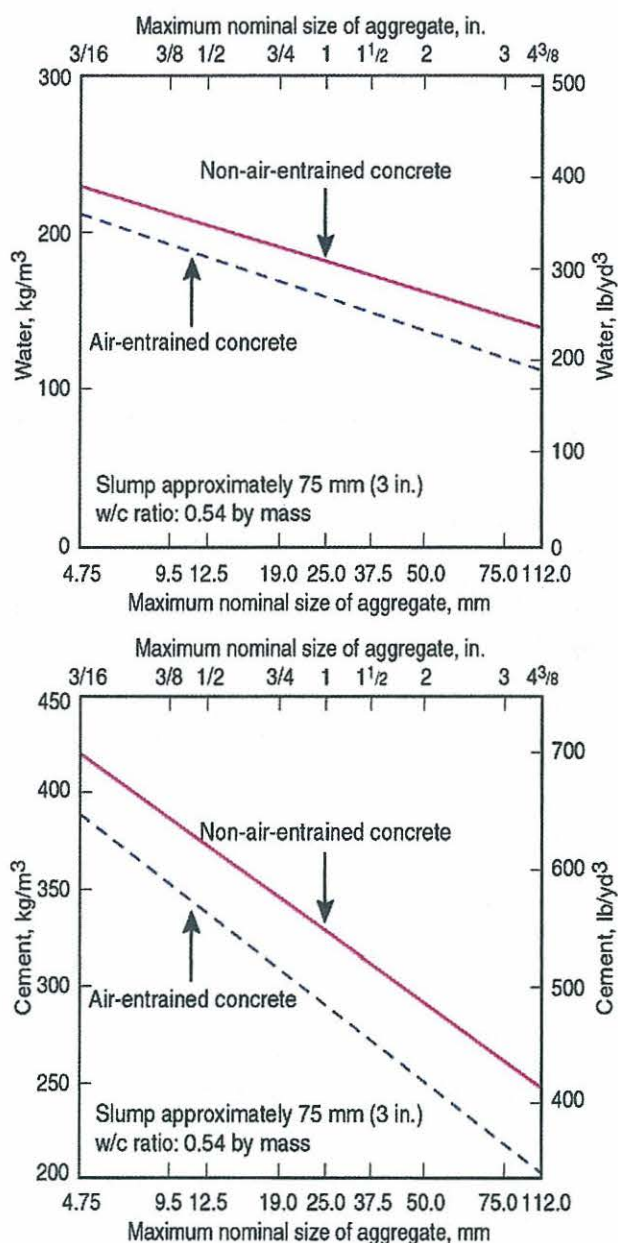


Fig. 6.2—Cement and water contents in relation to maximum size of aggregate for air-entrained and non-air-entrained concrete. Less cement and water are required in mixtures having large, coarse aggregate (Portland Cement Association 2002).

of sand should be kept as low as is practical while still providing the needed fine sizes for good workability and minimal bleeding. Low sand content usually minimizes the water requirement of the concrete and gives the most economical proportions, provided that gap-grading does not occur due to lack of intermediate sizes. Strictly minimizing the proportion of fine aggregate, however, is not always advisable, particularly if using more fine aggregate makes the concrete noticeably easier to place, consolidate, and finish. Low sand contents are desirable in low-slump concrete to maximize strength, but higher sand contents are needed to minimize segregation in higher-slump concretes.

For each percentage point increase in the sand-aggregate ratio, the water requirement increases by approximately 1%.

Other factors being the same, aggregates composed of angular particles require more paste than equal proportions of smooth, rounded particles. When concrete strength is the criterion, however, rounded aggregates are not always preferable because crushed aggregates normally give higher strengths at a given w/cm . When durability requirements do not govern the w/cm , it is sometimes permissible, on the basis of test data, to use a higher w/cm with crushed material than would be required for rounded material. Thus, with a different aggregate, a different w/cm may sometimes be used to achieve the same strength. For pavement concrete designed on the basis of the modulus of rupture (flexural strength), angular aggregates usually are preferable.

Whenever possible, the concrete mixture should be based on trial mixtures using job materials. All mixtures should be workable. The minimum properties generally determined from tests of trial batches are strength (flexural or compressive), slump, percent air, and density. When these tests cannot be made, use recommendations based on experience, such as those of ACI 211.1 and 211.3R as well as *Design and Control of Concrete Mixtures* (Portland Cement Association 2002). Although these recommendations do not take into account differences in strength that may arise from differences in aggregate or cement characteristics, they are conservative enough to provide sound results. The initial mixture proportions should be adjusted as necessary based on test results and observations during production.

6.4.4 Air entrainment—For moderate-strength concrete of a given w/cm , air entrainment will reduce strength about 2 to 6% for each percentage point of added air (Klieger 1952). Less mixing water, however, will be required to provide the same slump because the small air bubbles help to lubricate the mixture. Thus, if slump and cement content remain constant, a lower w/cm results, which will partly compensate for the strength loss.

Coarse-aggregate proportions are usually the same for both air-entrained and non-air-entrained mixtures, but less fine aggregate is needed because the air increases the volume of mortar and improves workability. With some lean (low-cement-content) mixtures, air entrainment may actually increase the compressive strength by reducing the w/cm .

6.4.5 Quantity of paste—For given materials, the optimum mixture proportions will use the least amount of total water per unit volume of concrete to obtain the required slump and workability. With a fixed w/cm , using mixtures having the least paste reduces material costs. Typically, the cementitious material in the paste is the most costly ingredient of the concrete, so using more paste than required unnecessarily adds to the cost of the concrete. When using expensive aggregates, however, the cost of the paste should be balanced against the cost of the aggregate.

Minimizing the paste is also desirable because water in the paste is the primary cause of shrinkage as the concrete hardens and dries. The more water (and the more paste), the greater the drying shrinkage. Also, cement produces heat as it hydrates. Therefore, high cement contents can produce an