Guide Specification for High Performance Concrete for Bridges

Michael A. Caldarone, Peter C. Taylor,
Rachel J. Detwiler, Shrinivas B. Bhide

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Guide Specification for High-Performance Concrete for Bridges

FIRST EDITION

by Michael A. Caldarone, Peter C. Taylor, Rachel J. Detwiler, and Shrinivas B. Bhidé

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An organization of cement companies to improve and extend the uses of portland cement and concrete through market development, engineering, research, education, and public affairs work.

KEYWORDS: AASHTO, abrasion resistance, admixtures, aggregates, air-entrained concrete, air void analyzer, alkali-carbonate reactivity, alkali-silica reactivity, ASR, ASTM, bridge, cement, cementitious materials, chemical admixtures, chloride ion penetration, cold weather, compressive strength, consistency, corrosion inhibitors, crack control, cracking, creep, curing, D-cracking, deck, durability, finishing, flowing concrete, footing, freeze/thaw durability, fly ash, girder, guide specification, high-performance concrete, hot weather, mass concrete, mixture proportioning, modulus of elasticity, pier, placing, portland cement concrete, performance, properties, quality assurance, quality control, ready mixed concrete, scaling resistance, SCC, self consolidating concrete, shrinkage, silica fume, slag cement, spacing factor, standards, structural concrete, sulfate resistance, supplementary cementitious materials, temperature control, tests, trial batches, volume changes, and water-cementitious materials ratio, w/cm.

ABSTRACT: This guide specification is intended to serve as a guide for developing specifications for all high performance concretes supplied for highway bridges, whether produced by a ready mix supplier, a general contractor, or in a permanent plant of a precast concrete manufacturer. For the purposes of this specification, high performance concrete (HPC) is considered as concrete engineered to meet specific needs of a project; including: mechanical, durability, or constructability properties. The document provides mandatory language that the specifier can cut and paste into project specifications. It also includes guidance on what characteristics should be specified in a given case, and what performance limit is needed to ensure satisfactory performance for a given element or environment.

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WARNING: Contact with wet (unhardened) concrete, mortar, cement, or cement mixtures can cause SKIN IRRITATION, SEVERE CHEMICAL BURNS (THIRDDEGREE), or SERIOUS EYE DAMAGE. Frequent exposure may be associated with irritant and/or allergic contact dermatitis. Wear waterproof gloves, a long-sleeved shirt, full-length trousers, and proper eye protection when working with these materials. If you have to stand in wet concrete, use waterproof boots that are high enough to keep concrete from flowing into them. Wash wet concrete, mortar, cement, or cement mixtures from your skin immediately. Flush eyes with clean water immediately after contact. Indirect contact through clothing can be as serious as direct contact, so promptly rinse out wet concrete, mortar, cement, or cement mixtures from clothing. Seek immediate medical attention if you have persistent or severe discomfort.

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Introduction

This document is intended to serve as a guide for developing specifications for high performance concrete for individual projects in all 50 states. It is intended to apply to all high performance concretes supplied for highway bridges, whether produced by a ready mix supplier, a general contractor, or in a permanent plant of a precast concrete manufacturer. For the purposes of this specification, high performance concrete (HPC) is considered as concrete that attains mechanical, durability, or constructability properties exceeding those of normal concrete. The specific meaning of "high performance" depends on the concrete property or properties under consideration, which may or may not include strength. Examples of HPC applications in bridges include:

A bridge deck in a northern climate must resist the ingress of chloride ions and deicer scaling. If there is concern about the potential for cracking, a low modulus of elasticity and/or high creep might be specified, in which case very high compressive strength might be incompatible with the desired properties. Thus the specification should require a chloride ion penetration and scaling resistance. It should require only the strength determined by the Engineer to be necessary for structural or operational reasons (e.g., for opening to traffic by a certain time).

A post-tensioned bridge girder could benefit from a high modulus of elasticity and low creep to minimize deflections and loss of prestress. It most likely will have high strength as a consequence of these properties, or the designer may specify high strength to allow a more efficient design, with fewer girders to support the same load. The specification would thus include criteria for modulus of elasticity, creep, and compressive strength as dictated by the structural design.

A pretensioned, precast girder may be made of self-consolidating concrete. The specification could then include a slump flow as well as the modulus of elasticity, creep, and compressive strength requirements. Alternatively, the specification could omit a consistency requirement and allow the contractor to propose the use of self-consolidating concrete.

A massive bridge pier or foundation must be designed to limit stresses and cracking due to thermal gradients. If high strength, particularly high early strength, is specified for this application, the concrete will be more vulnerable to cracking. In this case, high strength is not consistent with high performance. The specification should not require high strength except at later ages (56 or 90 days), since to limit cracking the concrete most likely will include relatively high percentages of supplementary cementitious materials.

The above examples illustrate different criteria that might be specified for different applications within the same structure. The designer must select the criteria that are important for the specific application. Specifying additional criteria beyond what is needed is likely to increase cost, make it more difficult to meet the criteria that truly are important, or result in unanticipated problems. For example, high strength, particularly high early strength, frequently is achieved through an increase in the cementitious materials content. The resulting heat generated may increase the probability of thermal cracking even for sections of moderate size. Or for a bridge deck, for example, the high stiffness, low creep, and high paste

content that usually accompany high strength may result in cracking due to autogenous or drying shrinkage. If high strength was not necessary, or was needed only at later ages, cracking could be limited by appropriate adjustments to the mix design.

Some criteria (such as chloride penetration) are intended to be used for prequalification of a given mixture, while others (such as compressive strength and air content) are appropriate for use in quality control and acceptance tests. The commentary indicates which of these applications each criterion falls into.

The intended user of this specification is an engineer working either directly for a state or local highway authority or other bridge owner, or for a contractor to a state or local highway authority or bridge owner. The user should be familiar with the characteristics of local materials. The user also should be aware of local durability concerns that may necessitate special measures to prevent premature deterioration of the concrete. This document is intended to be modified by the user to suit local conditions by inserting relevant clauses into the contract specification and by inserting numerical values where required.

The specification is accompanied by a Commentary that provides explanatory notes, examples and guidance for the engineer and contractor in achieving the desired properties.

Note: Throughout this specification, AASHTO standards are given as primary, with the corresponding ASTM standard in parentheses. The two types of standards are not directly equivalent in every case. The user must select one or the other. Where only one is given, there is no corresponding standard.

1.0 Scope

This Specification covers the requirements for materials; methods for proportioning, mixing, transporting, placing, finishing, and curing; and quality control and assurance of high performance concrete bridge elements.

2.0 References

This specification and its accompanying Commentary refer to the following standards, specifications, and publications. Publication dates deliberately are omitted from this listing; the user should refer to the most current version.

2.1 American Association of State Highway and Transportation Officials (AASHTO) (www.transportation.org/aashto)

AASHTO M 6, *Standard Specification for Fine Aggregate for Portland Cement Concrete*

AASHTO M 80, *Standard Specification for Coarse Aggregate for Portland Cement Concrete*

AASHTO M 85, *Standard Specification for Portland Cement*

AASHTO M 154, *Standard Specification for Air-Entraining Admixtures for Concrete*

AASHTO M 157, *Standard Specification for Ready-Mixed Concrete*

AASHTO M 194, *Standard Specification for Chemical Admixtures for Concrete*

AASHTO M 240, *Standard Specification for Blended Hydraulic Cement*

AASHTO M 295, *Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete*

AASHTO M 302, *Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars*

AASHTO M 307, *Standard Specification for use of Silica Fume as a Mineral Admixture in Hydraulic-Cement Concrete Mortar and Grout*

AASHTO T 22, *Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens*

AASHTO T 23, *Standard Method of Test for Making and Curing Concrete Test Specimens in the Field*

AASHTO T 24, *Standard Method of Test for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*

AASHTO T 27, *Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates*

AASHTO T 96, *Standard Method of Test for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*

AASHTO T 119, *Standard Method of Test for Slump of Hydraulic-Cement Concrete*

AASTHO T 121, *Standard Method of Test for Mass per Cubic Meter (Cubic Foot), Yield, and Air Content (Gravimetric) of Concrete*

AASHTO T 126, *Standard Method of Test for Making and Curing Concrete Test Specimens in the Laboratory*

AASHTO T 141, *Standard Method of Test for Sampling Freshly Mixed Concrete*

AASHTO T 152, *Standard Method of Test for Air Content of Freshly Mixed Concrete by the Pressure Method*

AASHTO T 160, *Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete*

AASHTO T 161, *Standard Method of Test for Resistance of Concrete to Rapid Freezing and Thawing*

AASHTO T 196, *Standard Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method*

AASHTO T 277, *Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*

AASHTO T 318-02, *Standard Method of Test for Water Content of Freshly Mixed Concrete Using Microwave Oven Drying*

AASHTO PP 34, *Standard Practice for Estimating the Cracking Tendency of Concrete*

AASHTO LRFD, *Bridge Design Specifications,* U.S. 3rd Edition, 2004

AASHTO Quality Assurance Guide Specification, 1996

2.2 American Society for Testing and Materials International (ASTM International) (www.astm.org)

ASTM C 31, *Standard Practice for Making and Curing Concrete Test Specimens in the Field*

ASTM C 33, *Standard Specification for Concrete Aggregates*

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

ASTM C 42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*

ASTM C 94, *Standard Specification for Ready-Mixed Concrete*

ASTM C 131, *Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*

ASTM C 136, *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*

ASTM C 138, *Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete*

ASTM C 143, *Standard Test Method for Slump of Hydraulic-Cement Concrete*

ASTM C 150, *Standard Specification for Portland Cement*

ASTM C 157, *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*

ASTM C 172, *Standard Practice for Sampling Freshly Mixed Concrete*

ASTM C 173, *Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method*

ASTM C 192, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*

ASTM C 231, *Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*

ASTM C 260, *Standard Specification for Air-Entraining Admixtures for Concrete*

ASTM C 295, *Standard Guide for Petrographic Examination of Aggregates for Concrete*

ASTM C 403, *Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance*

ASTM C 441, *Standard Test Method for Effectiveness of Pozzolans or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction*

ASTM C 457, *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*

ASTM C 469, *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression*

ASTM C 494, *Standard Specification for Chemical Admixtures for Concrete*

ASTM C 512, *Standard Test Method for Creep of Concrete in Compression*

ASTM C 595, *Standard Specification for Blended Hydraulic Cements*

ASTM C 618, *Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete*

ASTM C 666, *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*

ASTM C 672, *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals*

ASTM C 779, *Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces*

ASTM C 856, *Standard Practice for Petrographic Examination of Hardened Concrete*

ASTM C 944, *Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method*

ASTM C 989, *Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars*

ASTM C 1012, *Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution*

ASTM C 1017, *Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete*

ASTM C 1064, *Standard Test Method for Temperature of Freshly Mixed Portland Cement Concrete*

ASTM C 1074, *Standard Practice for Estimating Concrete Strength by the Maturity Method*

ASTM C 1105, *Standard Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction*

ASTM C 1157, *Standard Performance Specification for Hydraulic Cement*

ASTM C 1202, *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*

ASTM C 1240, *Standard Specification for Silica Fume Used in Cementitious Mixtures*

ASTM C 1260, *Standard Test Method for Potential Alkali Reactivity of Aggregate (Mortar-Bar Method)*

ASTM C 1293, *Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction*

ASTM C 1567, *Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)*

ASTM C 1582, *Standard Specification for Admixtures to Inhibit Chloride-Induced Corrosion of Reinforcing Steel in Concrete*

ASTM C 1602, *Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete*

2.3 U.S. Department of Transportation, Federal Highway Administration (FHWA) (www.fhwa.dot.gov)

FP-03, *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects*

FHWA High Performance Concrete Tool Kit, U.S. Department of Transportation, Federal Highway Administration, Publication NO. FHWA-RD-97-097, 30 May 1997.

2.4 American Concrete Institute (ACI) (www.aci-int.org)

ACI 121R, *Quality Assurance Systems for Concrete Construction*

ACI 201.2R, *Guide to Durable Concrete*

ACI 207.1R, *Mass Concrete*

ACI 207.2R, *Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete*

ACI 207.4R, *Cooling and Insulating Systems for Mass Concrete*

ACI 209R, *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures*

ACI 211.1, *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete*

ACI 211.2, *Standard Practice for Selecting Proportions for Structural Lightweight Concrete*

ACI 211.3R, *Guide for Selecting Proportions of No-Slump Concrete*

ACI 211.4R, *Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash*

ACI 224R, *Control of Cracking in Concrete Structures*

ACI 232.1R, *Use of Raw or Processed Natural Pozzolans in Concrete*

ACI 232.2R, *Use of Fly Ash in Concrete*

ACI 233R, *Slag Cement in Concrete and Mortar*

ACI 234, *Silica Fume in Concrete*

ACI 301, *Standard Specification for Structural Concrete*

ACI 302.1R, *Guide for Concrete Floor and Slab Construction*

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

ACI 305R, *Hot Weather Concreting*

ACI 306R, *Cold Weather Concreting*

ACI 308, *Standard Practice for Curing Concrete*

ACI 308.1, *Standard Specification for Curing Concrete*

ACI 309R, *Guide for Consolidation of Concrete*

ACI 318, *Building Code Requirements for Structural Concrete*

ACI 345, *Guide for Concrete Highway Bridge Deck Construction*

ACI 363R, *State of the Art Report on High-Strength Concrete*

ACI 363.2, *Guide to Quality Control and Testing of High-Strength Concrete*

2.5 Portland Cement Association (PCA) (www.cement.org)

PCA EB001, *Design and Control of Concrete Mixtures*

PCA IS415, *Guide Specification for Concrete Subject to Alkali-Silica Reactions*

2.6 Precast/Prestressed Concrete Institute (PCI) (www.pci.org)

PCI MNL-116, *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products*

PCI MNL-133, *Bridge Design Manual*

PCI TR-6-03, *Interim Guidelines for the Use of Self-Consolidating Concrete in PCI Member Plants*

PCI TM-103, *Quality Control Technician/Inspector Level III Training Manual*

2.7 National Ready Mixed Concrete Association (NRMCA) (www.nrmca.org)

NRMCA Publication 190, *Guideline Manual for Quality Assurance Quality Control*

NRMCA, *Quality Control Manual*

3.0 Definitions

Bridge: A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 ft (6.5 m) between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it also may include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

Cementitious materials: Portland cements, blended cements, and supplementary cementitious materials (e.g., fly ash, slag cement, silica fume, and calcined clay) used in concrete and masonry construction.

Cold weather: A period when, for more than three consecutive days, the following conditions exist: (1) the average daily air temperature is less than 40°F (5°C) and (2) the air temperature is not greater than 50°F (10°C) for more than one-half of any 24-hr period. The average

daily temperature is the mean of the highest and the lowest temperatures occurring during the period from midnight to midnight.

Consistency: The relative mobility or ability of freshly mixed concrete or mortar to flow; the usual measurements are slump for concrete, flow for mortar or grout, and penetration resistance for neat cementitious paste.

Contract: The written agreement executed between the Owner and the Contractor that sets forth the obligations of the parties including but not limited to the performance of the work, furnishing of materials and labor, and basis of payment.

Contractor: Any individual, partnership, corporation, or joint venture with whom the Owner enters into agreement for construction of the work under the contract documents.

Creep: Time-dependent deformation due to sustained load.

Curing: The maintenance of satisfactory moisture and temperature in concrete during its early stages so that desired properties may develop.

Engineer: The registered engineer designated by the Owner as the accepting authority responsible for issuing the project specification or administering work under the contract documents.

Flowing concrete: Concrete that is characterized by a slump greater than 7½ in. (190 mm) while remaining cohesive.

High performance concrete (HPC): Concrete engineered to meet specific needs of a project; including: mechanical, durability, or constructability properties.

Hot weather: A period when, for more than three consecutive days, the following conditions exist: (1) the average daily air temperature is greater than 77°F (25°C) and (2) the air temperature for more than one-half of any 24-hr period is not less than 85°F (30°C). The average daily temperature is the mean of the highest and the lowest temperatures occurring during the period from midnight to midnight.

Inspector: The Engineer's or Owner's authorized representative who is assigned to make detailed inspections of the quality and quantity of the work and its conformance to the provisions of the Contract.

Mass concrete: A volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat and temperature gradients from hydration of the cementitious materials, and attendant volume change.

Owner: The local, state, or federal highway agency or other public or private entity that will take possession of the bridge on completion or upon transfer.

Quality assurance: The planned activities and systematic actions necessary to provide adequate confidence to the Owner and other parties that the products or services will perform their intended functions. Quality assurance is a management tool.

Quality control: Actions related to the physical characteristics of the materials, processes, and services which provide a means to measure and control the characteristics to predetermined quantitative criteria. Quality control is a production tool.

Self-consolidating concrete (SCC): Highly flowable, non-segregating concrete that can spread into place, fill the formwork, and encapsulate the reinforcement under its own weight without any mechanical consolidation.

Subcontractor: Any individual, partnership, corporation, or joint venture with whom the Contractor enters into agreement for construction of some portion of the work under the contract documents.

Supplementary cementitious materials: Cementitious materials other than portland cements used in concrete and masonry construction (e.g., slag cement, fly ash, calcined clay, and silica fume).

Water-cementitious materials ratio (w/cm): The ratio of the mass (or weight) of water to the mass (or weight) of all cementitious materials in the concrete.

4.0 Performance Requirements

Laboratory tests conducted to ensure that the proposed materials and the proposed mix proportions meet the specified performance requirements shall be conducted by a laboratory accredited by AASHTO (or equivalent) for those tests (or in a PCI-certified plant for the compressive strength and consistency tests).

4.1 Abrasion Resistance

For bridge decks or surface courses, aggregates known to polish shall not be used, or the coarse aggregate shall be tested according to AASHTO T 96 (ASTM C 131). The result shall not exceed_____%.

4.2 Chloride Ion Penetration

4x8-in. (100x200-mm) concrete cylinders shall be _____ cured to an age of _____ and tested in accordance with AASHTO T 277 (ASTM C 1202). The charge passed in six hours shall not exceed _____ coulombs.

4.3 Compressive Strength

The concrete shall meet all of the requirements given in Table 4.3-1 for compressive strength as tested in accordance with AASHTO T 22. Specimens may be either 4x8-in. (100x200-mm) or 6x12-in. (150x300-mm) cylinders.

Table 4.3-1 Required Minimum Compressive Strengths

Compressive strength	Age at test for design strength	Curing

4.4 Creep

The concrete shall meet the requirements for creep as tested in accordance with ASTM C 512. Specimens shall be 6x12-in. (150x300-mm) cylinders. Store at 50% RH at 73°F (23°C) until test. Age at loading, and maximum creep coefficient shall be as shown in Table 4.4-1. Loading shall continue for 180 days.

Table 4.4-1 Maximum Creep Coefficient

Curing	Age at loading	Creep coefficient
Moist cure after demolding until age 7 days		

4.5 Modulus of Elasticity

The concrete shall meet the requirements for modulus of elasticity as tested in accordance with ASTM C 469 and shown in Table 1. Specimens may be either 4x8-in. (100x200-mm) or 6x12-in. (150x300-mm) cylinders moist cured (100% RH at 73.4±3.0°F [23.0±1.7°C]) until age of testing.

Table 4.5-1 Required Minimum Modulus of Elasticity

Modulus of elasticity	Age at test
	28 days
	56 days

4.6 Freeze/Thaw Durability

The concrete shall have a durability factor of at least _____% when tested in accordance with AASHTO T 161, Procedure A (ASTM C 666, Procedure A), except that the age at testing shall be 56 days. Specimens shall be prisms at least 3 in. (75 mm) but not more than 5 in. (125 mm) in width or depth and at least 11 in. (280 mm) but not more than 16 in. (400 mm) in length.

4.7 Scaling Resistance

The concrete shall have a visual rating not greater than __ when tested in accordance with ASTM C 672, except that the specimens shall be _____ cured to age _____ before commencement of the 14-day drying period.

4.8 Shrinkage

The drying shrinkage of the concrete when tested in accordance with AASHTO T 160 (ASTM C 157) shall not exceed _____. Specimens shall be moist cured until the age of _____ and shrinkage shall be monitored for 180 days thereafter. The baseline comparator measurement shall be taken at 24 hours after casting.

4.9 Sulfate Resistance

The sulfate exposure for this Work has been determined to be _____. The combination of cementitious materials in the proportions proposed shall have sulfate resistance at least equivalent to that of Type __ cement and the water-cementitious materials ratio shall not exceed ___.

4.10 Consistency

Concrete of conventional consistency—The concrete shall have a slump not less than ___ nor more than ___ as measured in accordance with AASHTO T 119 (ASTM C 143).

Self-consolidating concrete (SCC)—The concrete shall be classified as self-consolidating concrete and shall be produced such that it can be placed and consolidated without vibration and without segregation. The slump flow[1] shall be not less than ___ nor more than ___.

4.11 Alkali-Silica Reactivity

The aggregates shall be evaluated for potentially deleterious alkali-silica reactivity and mitigating measures taken if necessary, as described in Section 5.2.2.1.

5.0 Materials

5.1 Cementitious Materials

Portland cement shall conform to the requirements of AASHTO M 85 (ASTM C 150) or ASTM C 1157 for the specified type, including the optional requirement for early stiffening. Blended cement shall conform to the requirements of AASHTO M 240 (ASTM C 595) or ASTM C 1157 for the specified type, including the optional requirement for early stiffening. Supplementary cementitious materials not incorporated into the blended cement shall conform to the relevant standards as follows:

- Fly ash and natural pozzolans shall conform to the requirements of AASHTO M 295 (ASTM C 618) for the specified class.
- Slag cement shall conform to the requirements of AASHTO M 302 (ASTM C 989) for the specified grade.
- Silica fume shall conform to the requirements of AASHTO M 307(ASTM C 1240).

For concrete exposed to sulfate attack, the proposed combination of cementitious materials shall meet the requirements of Section 4.9.

Concrete subject to applications of deicing salts shall be restricted to the following maximum limits on the total quantity of supplementary cementitious materials, including supplementary cementitious materials incorporated in blended cement:

- Fly ash or other pozzolans up to 25% by mass of cementitious materials
- Slag cement up to 50% by mass of cementitious materials
- Silica fume up to 10% by mass of cementitious materials
- Mixtures of silica fume, fly ash or other pozzolans, and slag cement up to 50% by mass of cementitious materials, with no more than 10% being silica fume and no more than 25% being fly ash
- Mixtures of fly ash or other pozzolans, and silica fume up to 35% by mass of cementitious materials, with no more than 10% being silica fume and no more than 25% being fly ash

5.2 Aggregates

5.2.1 Grading and Impurities

Fine and coarse aggregates shall conform to the requirements of AASHTO M 6 and M 80 (ASTM C 33), except that the soundness requirement shall be waived.

5.2.2 Durability

Unless the performance history of the aggregate is known, it shall be tested to determine its potential for:

- Alkali-silica reactivity
- Alkali-carbonate reactivity
- D-cracking

An aggregate shall be considered to have an acceptable performance history provided the field concrete made from it is at least 15 years old, the cementitious materials used are comparable (particularly with regard to alkali content and use of supplementary cementitious materials), and the exposure conditions are at least as severe as those in the proposed project. Petrographic examination of the field concrete by ASTM C 856 shall be conducted to verify satisfactory performance. A copy of the petrographer's report shall be submitted to the Engineer. The aggregate shall be approved by the Engineer before it is used in the project.

[1] Aggoun, S., Kheirbek, A., Kadri, E.H., and Duval, R., *Study of the Flow of Self-compacting Concretes,* First North American Conference on the Design and Use of Self-Consolidating Concrete, 12-13 November 2002, Center for Advanced Cement-Based Materials, Northwestern University, Evanston, IL, pp. 259-265.

Test data for comparable aggregate from the same quarry are acceptable for this purpose.

If any of these criteria cannot be met by an acceptable field history, the aggregate must be tested as described in the following sections.

5.2.2.1 Alkali-Silica Reactivity

If an acceptable field performance history is not available, representative samples of siliceous fine and coarse aggregates proposed for use on the project shall be evaluated petrographically in accordance with ASTM C 295 and by the mortar bar test, ASTM C 1260.

Aggregate evaluated in accordance with ASTM C 295 and determined to contain more than the following quantities of reactive constituents, expressed as percent by mass, shall be considered potentially reactive[2]:

- Optically strained, microfractured, or microcrystalline quartz exceeding 5.0%
- Chert or chalcedony exceeding 3.0%
- Tridymite or crystobalite exceeding 1.0%
- Opal exceeding 0.5%
- Natural volcanic glass in volcanic rocks exceeding 3.0%

Aggregate tested in accordance with ASTM C 1260 and exhibiting mean mortar bar expansions at 14 days greater than 0.10% shall be considered potentially reactive.

Aggregates considered potentially reactive by either of the above methods may be further evaluated by ASTM C 1293. Aggregates exhibiting expansions greater than 0.04% at 1 year shall be considered potentially reactive. Aggregates exhibiting expansions no more than 0.04% and demonstrating no prior evidence of reactivity in the field shall be considered innocuous.

If an aggregate is determined to be potentially reactive in accordance with the above protocol, or if the field performance of the aggregate indicates that it is reactive regardless of the results of any of the above tests, an appropriate mitigation measure shall be specified. The effectiveness of the mitigation measure shall be verified by ASTM C 1567 to limit the 14-day expansion to a maximum of 0.10%.

Data from past field performance and/or ASTM C 1293 tests using the same aggregate, if available, also may be used to demonstrate satisfactory performance.

No substitution of any material in the concrete is permitted without testing to verify its performance with regard to alkali-silica reaction.

5.2.2.2 Alkali-Carbonate Reactivity

Representative samples of fine and coarse aggregates comprised of calcitic dolomites or dolomitic limestones proposed for use on the project shall be evaluated petrographically in accordance with ASTM C 295. Aggregates characterized by relatively large, rhombic crystals of dolomite set in a finer-grained matrix of calcite, clay and microcrystalline quartz shall be considered potentially reactive and shall be evaluated in accordance with ASTM C 1105 using the proposed cement-aggregate combinations. Cement-aggregate combinations exhibiting mean expansion values greater than 0.015% at 3 months, 0.025% at 6 months, or 0.030% at 1 year shall be considered potentially reactive.

Aggregates found by the above measures to be potentially reactive may be used only when diluted with a nonreactive aggregate. The suitability of the mixture of aggregates shall be verified by ASTM C 1105 to result in mean expansions not greater than 0.015% at 3 months, 0.025% at 6 months, or 0.030% at 1 year.

5.2.2.3 D-Cracking

For bridge decks that will be subject to freezing and thawing, coarse aggregates shall be tested for susceptibility to D-cracking unless their performance history is known. Test data or field performance data for comparable aggregates from the same quarry are acceptable for this purpose. Any of the following test methods are acceptable:

- Washington Hydraulic Fracture test[3]

[2] *Guide Specification for Concrete Subjected to Alkali-Silica Reactions,* Portland Cement Association, IS415, September 1998, 8 pages.

[3] Janssen, Donald J. and Snyder, Mark B., *Resistance of Concrete to Freezing and Thawing,* SHRP-C-391, Washington, DC: Strategic Highway Research Program, National Research Council, 1994, 301 pp.

- AASHTO T 161 (ASTM C 666), extended to 350 cycles; the durability factor is calculated from the expansion of the specimens.
- Iowa Pore Index Test[4,5]

Aggregates failing these tests shall not be used.

5.3 Water

Mixing water for concrete shall comply with ASTM C 1602.

5.4 Chemical Admixtures

Chemical admixtures shall comply with AASHTO M 154 (ASTM C 260), AASHTO M 194 (ASTM C 494), or ASTM C 1017, as applicable. Corrosion inhibitors also shall be tested in accordance with ASTM C 1582.

The manufacturer shall certify that all admixtures contain no purposefully added chlorides, and that the chloride ion content of the admixtures in the quantities proposed is below the limits given by ACI 201.2 *Guide to Durable Concrete* (0.01% by mass of cementitious materials).

6.0 Submission and Design Requirements

6.1 Concrete Mixture Proportioning

The Contractor shall be responsible for concrete mixture proportioning. Concrete shall be proportioned to meet the performance requirements detailed in the contract documents and Sections 4 and 5.

Concrete mixture proportions shall be designed in conformance with ACI 211.1, 211.2, 211.3R, or 211.4R and verified by trial batches.

At least 30 days before delivery of the concrete, the manufacturer of the concrete shall submit to the Engineer a statement detailing the materials, sources, and proportions of materials to be used for each grade of concrete to be supplied. No substitutions shall be allowed without the approval of the Engineer, who may require a resubmission of test data.

The statement shall include the purchaser's name, contract number, concrete manufacturer's name, mix design number, primary and backup production facility locations, intended mix use, air content, and slump ranges for each intended use.

6.2 Concrete Production Facility Certification

The manufacturer of the concrete shall submit a current certification of the concrete production facility, including the concrete production facility and delivery fleet as issued by the National Ready Mixed Concrete Association (NRMCA) for the plant(s) proposed for use. For concrete batched for or within a precast concrete plant, submit proof of current certification in the Precast/Prestressed Concrete Institute's Plant Certification Program.

6.3 Concrete Materials

Test data for all concrete-making materials shall be provided to the Engineer 60 days prior to the start of the Work. All materials shall be approved by the Engineer before being used in the Work. Samples of all concrete-making materials (aggregates, cementitious materials, water, and chemical admixtures) shall be provided when requested by the Engineer prior to or during production of the concrete.

6.4 Temperature Control Methods

During hot and cold weather, the methods to be used to control the temperature of the concrete as placed and the temperature of the in-place concrete during curing shall be submitted to the Engineer by the Contractor. Methods to be used to control the core temperature and temperature gradients during curing shall be submitted to the Engineer by the Contractor. Refer to Section 8.5 for further details and to PCI MNL-116 for standard procedures for precast concrete manufacturing plants.

6.5 Crack Control Methods

The method(s) to be used to control cracking due to shrinkage and/or thermal stresses shall be submitted to the Engineer. All concrete elements with smallest dimension larger than 2 ft. (600 mm) shall require implementation of method(s) to control thermal stresses.

[4] Traylor, M.L., *Efforts to Eliminate D-Cracking in Illinois,* Transportation Research Record, No. 853, 1982, pp. 9-14.

[5] Marks, V.J. and Dubberke, W., *Durability of Concrete and the Iowa Pore Index Test,* Transportation Research Record, No. 853, 1982, pp. 25-31.

The maximum acceptable crack width at the surface for structural elements, including decks, columns, beams, parapets, and abutments shall be _____in. (____mm).

The Contractor shall inspect unformed concrete surfaces and identify and record the width, depth, and density (in linear feet per square foot or linear meters per square meter) of cracks after removal of burlap or curing tarpaulins. Results shall be reported to the Engineer.

6.6 Curing

Curing shall be in accordance with FHWA FP-03, Standard Specifications For Construction of Roads and Bridges on Federal Highway Projects, Section 552.15, and ACI 308R, *Standard Practice for Curing Concrete.* In the event of a conflict between the two documents, FP-03 shall take precedence over ACI 308.

The Contractor shall submit written descriptions of the method(s) to be used for the curing of all bridge elements to the Engineer for review and approval. Hot- and cold-weather curing practices shall be employed when weather conditions warrant (see definitions of hot and cold weather). See Section 6.4 for temperature control requirements. In addition, if cracks appear on the surface of the concrete during construction, placement shall be discontinued until corrective measures are implemented.

Curing shall begin within 15 minutes or 6 ft (1.9 m) of final finishing.

For concrete to be used in the bridge decks, barrier rails, approach slabs, and barrier slabs, the Contractor shall comply with ACI 302.1R and ACI 308R. If silica fume, fly ash, or slag cement is used, the Contractor shall limit finishing operations to screeding, bull floating, and grooving. Continuous fogging above the surface of the concrete shall be used during the finishing operation and maintained until the concrete surface can support wet burlap without deformation. Free-standing water shall not be permitted on the surface of the concrete prior to final set. As soon as the surface of the concrete will support wet burlap or cotton mats without deformation, the Contractor shall apply wet burlap or wet cotton mats to the textured concrete surface. The concrete shall remain continuously wet with a fog nozzle system or soaker hoses for 7 days and until a compressive strength of 3200 psi (22 MPa) is reached. The use of polyethylene sheeting or plastic-coated burlap blankets shall not be permitted.

For concrete intended for use in prestressed concrete or when strengths are in excess of 6000 psi, temperatures shall be monitored by two recording thermometers showing the time-temperature relationship per 200 ft (60 m) of bed. For girders, one thermometer shall be located at the center of gravity of the top flange and one at the center of gravity of the bottom flange. For piles, one thermometer shall be located midway between the outside corners of the pile and the nearest edge of the center void. If there is no void, only one thermometer shall be provided at the center of gravity of the cross section. Initial application of heat to accelerate curing shall begin only after the concrete has reached its initial set as determined by ASTM C 403. When used, steam shall be at 100% RH. Application of heat shall not be directly on concrete. Concrete temperature shall be increased at a rate not exceeding 40°F (22°C) per hour until the desired concrete temperature is reached. The concrete temperature shall not exceed ___°F (___°C). Heat curing may continue until the concrete has reached the release strength. The Contractor shall detension strands before the internal concrete temperature has decreased to 20°F (11°C) less than its maximum temperature.

6.7 Quality Control Plan

See Section 7.2 for the description of the Quality Control Plan to be submitted by the Contractor to the Engineer.

7.0 Quality Management

7.1 Quality Assurance

The Owner or Owner's representative shall prepare and carry out a Quality Assurance Plan to assure that the final product will perform its intended function. The Quality Assurance activities shall not relieve the Contractor of Quality Control responsibilities under the terms of the Contract. The Quality Assurance Plan documents the Owner's quality objectives. At a minimum, the Quality Assurance Plan shall include the following:

- Owner's policy statement
- Quality objectives
- Scope of work under the Quality Assurance Plan
- Organization and reporting relationships
- Authority and responsibilities of the various organizations and contractors
- Description of overall quality assurance system, including which organizations are required to establish and implement quality assurance programs

7.2 Quality Control

Before the start of the work, the Contractor shall submit to the Engineer a written Quality Control Plan in accordance with Section 153 of FHWA FP-03, "Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects," or for precast concrete manufacturers certified under the PCI Plant Certification Program, submit applicable sections of the plant Quality System Manual. The Quality Control Plan shall include :

- Process control testing:
  - ▲ Materials to be tested.
  - ▲ Tests to be conducted.
  - ▲ Location of samples extracted.
  - ▲ Frequency of testing.
- Inspection and control procedures:
  - ▲ Preparatory phase
    - ■ Review all contract requirements.
    - ■ Ensure compliance of component materials to contract requirements.
    - ■ Coordinate all submittals.
    - ■ Ensure capability of equipment and personnel to comply with contract requirements.
    - ■ Ensure preliminary testing is accomplished.
    - ■ Coordinate surveying and staking.
  - ▲ Start-up phase
    - ■ Review contract requirements with personnel who will perform the work.
    - ■ Inspect start-up of work.
    - ■ Establish standards of workmanship.
    - ■ Provide necessary training.
    - ■ Establish detailed testing schedule based on production schedule.
  - ▲ Production phase
    - ■ Conduct inspection during construction to identify and correct deficiencies.
    - ■ Inspect completed phases before Owner's scheduled acceptance.
    - ■ Provide feedback and system changes to prevent repeated deficiencies.
  - ▲ Description of records: List the records to be maintained.
  - ▲ Personnel qualifications
    - ■ Document the name, authority, relevant experience, and qualifications of person with overall responsibility for inspection system.
    - ■ Document the names, authority, and relevant experience of all persons directly responsible for inspection and testing.
  - ▲ Subcontractors: Include the work of all subcontractors. Provide details of how each subcontractor will fit into the overall organization of the project, including lines of communication and authority between contractor and subcontractors, and among subcontractors.

The plan may be implemented wholly or in part by a Subcontractor or an independent organization. However, the administration of the program, including compliance with the plan and its modifications, and the quality of the work, remain the responsibility of the Contractor.

The Contractor's Quality Control program shall be well managed and the testing results shall be representative of actual operations. All quality control tests, inspections and approvals shall be documented by the Contractor and shall be kept on site for the use of the Contractor's personnel and shall be immediately available to the Owner's personnel for quality assurance and audit purposes. The Quality Control Plan shall contain sufficient detail to serve as a reference summary and schedule for all quality testing, inspection and approval processes carried out by the Contractor and its agents.

No portion of the work shall begin until after the Quality Control Plan covering that portion of the work has been accepted by the Engineer.

8.0 Production of Concrete

8.1 General

The volume of material in the mixer shall not exceed the rated mixing capacity of the drum.

Proper facilities shall be provided to enable inspection of the quality and quantity of the materials and the processes used in the manufacture and delivery of the concrete. The inspector shall be provided with all reasonable facilities for securing samples to determine whether

the concrete and its component materials are being supplied in conformance with the specification.

Mixers shall be emptied of wash water and returned concrete before charging with a new batch of concrete. The entire contents of the mixer shall be discharged before recharging.

8.2 Equipment

The concrete production facility and transport equipment shall conform to the certification requirements of the National Ready Mixed Concrete Association, the PCI Plant Certification Program, or equivalent. Documentation of the certification shall be provided to the Engineer on request.

The concrete production facility shall have either radio or telephone communication with the placement operation personnel.

All mixers shall be capable of combining the ingredients of the concrete into a thoroughly mixed and uniform mass, and of discharging the concrete so that the within-batch uniformity requirements of AASHTO M 157 (ASTM C 94) are met.

8.2.1 Within-Batch Uniformity

Mixing equipment used shall produce uniform concrete in accordance with the requirements of AASHTO M 157 (ASTM C 94).

The minimum sample size for determination of within-batch uniformity shall be 1 cu ft (30 liters). Samples for uniformity determination shall be taken after discharge of approximately 15% and 85% of the batch.

8.2.2 Non-Agitating Equipment

Concrete that is completely mixed in a stationary mixer may be transported in non-agitating equipment. The bodies of such equipment shall be smooth water-tight steel containers equipped with gates that permit control of the discharge of the concrete. Covers shall be used to protect the concrete during inclement weather. The concrete shall be discharged at the site, without segregation, in a thoroughly mixed and uniform mass so as to meet the within-batch uniformity requirements of AASHTO M 157 (ASTM C 94). Unless approved by the Engineer, discharge of the concrete shall be completed within 30 minutes after introduction of the mixing water to the cementitious materials and aggregates.

8.2.3 Agitating Equipment

Concrete that is completely mixed in a stationary mixer may be transported in agitator trucks or truck mixers. The equipment shall be operated at the speed of rotation designated by the manufacturer of the truck as the agitating speed. The concrete shall be discharged at the site, without segregation, in a thoroughly mixed and uniform mass, so as to meet the uniformity requirements of AASHTO M 157 (ASTM C 94). Except as specified for hot weather concrete, and unless approved by the Engineer, discharge of the concrete shall be completed within 1½ hours after introduction of the mixing water to the cement and aggregates.

8.3 Measurement of Materials

Measurement of all constituent concrete-making materials used shall be in accordance with AASHTO M 157 (ASTM C 94).

When there is evidence of inaccurately produced batches of concrete, recalibration of the scales and admixture dispensers may be required.

When ice is used as part of the mixing water, the ice shall be measured by mass.

8.4 Mixing

Mixing equipment shall comply with AASHTO M 157 (ASTM C 94).

Mixers shall be rotated at the speed recommended by the manufacturer of the mixer.

Mixing time shall be measured from the time that all concrete ingredients are in the mixing unit. The minimum mixing time for concrete shall be as recommended by the equipment manufacturer or the minimum time required to produce concrete meeting the uniformity acceptance criteria of AASHTO M 157 (ASTM C 94), whichever is greater.

Unless otherwise indicated by the mixer manufacturer, when a truck mixer is used for complete mixing and is charged to its maximum rated mixing capacity, each batch of concrete shall be mixed for not less than 70 nor more than 100 revolutions of the drum.

After completion of mixing, the truck mixer drum shall be rotated at the designated agitating speed until discharge of concrete commences.

When a stationary mixer is used for partial mixing of concrete prior to transferring to a truck mixer, the mixing time shall be no more than is required to intermingle the ingredients. After transfer to a truck mixer, further mixing at the designated mixing speed shall be carried out.

For concrete containing silica fume batched separately from the cement (that is, not a component of blended cement), the silica fume shall be added to the aggregate with the cement. Silica fume shall not be placed first in the mixer. Silica fume shall not be added to the mixer in pulpable bags.

8.5 Temperature Control

The concrete temperature at the time of discharge from the truck shall be at or between 50°F (10°C) and 90°F (32°C). The temperature of the cementitious materials shall be less than 150°F (65°C) immediately prior to batching. During curing, the maximum concrete temperature shall not exceed _____°F (_____°C) and the minimum temperature of concrete shall not fall below 50°F (10°C).

8.5.1 Cold Weather

During cold weather (see Section 3.0 for definition), special precautions shall be employed when producing, placing, finishing and curing the concrete to protect it from the effects of cold weather. Method(s) to be used to control the concrete placement temperature shall be submitted by the concrete supplier. Method(s) to be used to control the concrete temperature and temperature gradients during curing shall be submitted by the Contractor.

Water brought into direct contact with the cementitious materials shall have a temperature less than 104°F (40°C). The concrete production facility shall have a water temperature indicator installed such that the batch operator can ensure that the temperature restrictions are met for each batch. Provision shall be made for heating aggregates in the concrete production facility storage bins. Aggregates shall be free of ice, snow, and frozen lumps before being placed in the mixer. The temperature of concrete shall not be less than 50°F (10°C) at the time of placement. The mix water and/or aggregates may be heated to not more than 104°F (40°C). Provision must be made to ensure that the material is heated evenly before being placed in the mixer.

8.5.2 Hot Weather

Hot weather (see Section 3.0 for definition) concreting practices shall apply during hot weather (refer to definition of "hot weather"). Precautions shall be employed when producing, placing, finishing, and curing the concrete to protect it from the effects of hot weather. Method(s) to be employed to control the concrete placement temperature shall be submitted to the Engineer by the Contractor. Method(s) to be used to monitor weather conditions during concrete placement, control plastic shrinkage cracking, and control the concrete temperature and temperature gradients during curing shall be submitted to the Engineer by the Contractor.

When ice is added to the concrete, it shall be completely melted by the time the concrete mixing is completed. Unless approved by the Engineer, when the air temperature exceeds 82°F (28°C) and the concrete temperature exceeds 77°F (25°C), concrete delivered by means of agitators or truck mixers shall be discharged within 1 hr after the introduction of the mixing water.

Plastic shrinkage control procedures shall be employed when the evaporation rate of the freshly placed concrete exceeds the bleeding rate. Method(s) to be used shall effectively reduce the rate of moisture loss from the concrete surface or replenish moisture to the surface lost to evaporation. Fog spraying, if used, shall be at a rate sufficient to maintain a sheen of moisture on the surface, but no ponding of water. Excess moisture shall not be finished into the concrete. Allow the water to evaporate just prior to finishing.

8.5.3 Control of Temperature Differences

Unless it can be demonstrated by engineering analysis that it is not detrimental to the structure, the maximum temperature differential between the interior and exterior concrete shall be limited to 35°F (19°C). For precast products that use the addition of heat to accelerate curing, the maximum cooling rate for products that have achieved transfer or stripping strength is 50°F (28°C) per hour. Standard practices should be followed for transferring products from forms to storage that have demonstrated acceptable results.

8.6 Trial Batches and Mockups

Laboratory trial batches shall be made as a condition of final approval of the mix design. All specified properties shall be verified in accordance with the test methods prescribed in Section 4.

In addition, the Contractor shall be responsible for conducting a field trial batch of the concrete. At least 60 days prior to placing high performance concrete, a full-size trial batch of concrete shall be produced and tested. Field trial batches of concrete shall originate from each production facility that will be used to supply the concrete. Trial batches shall be delivered to the site of the work as directed by the concrete purchaser. When the concrete is delivered in a ready mixed concrete truck, the volume of the trial batch shall be the volume of concrete normally supplied by the truck. Field mockups shall be constructed as required by the Engineer to verify all techniques to be used for transport, placement, consolidation, finishing, and curing of the concrete member.

When an approved ready mixed concrete operation is currently supplying or has supplied a class of concrete within the last ____ months requiring comparable performance, permission may be granted by the Engineer to use the concrete mixture proportions from that operation without the need for the full range of laboratory or field trial batches provided that:

- There is no change in the source of any material.
- There has been no significant change in quality of the concrete-making materials.
- The proposed concrete mix design meets all specified requirements.
- The conditions of field placement are substantially the same as for the previous job.
- Documentation of all test data is submitted to the Engineer.

The Engineer shall indicate in writing which tests are not required.

8.7 Site Addition of Materials

When a truck mixer is used at agitating capacity no adjustment shall be made to the load of concrete.

EITHER: When a truck mixer is used for mixing of the concrete, no water from the truck water system shall be added after the initial introduction of the mixing water to the load of concrete. Water shall not be added to the load of concrete at any time.

OR: The amount of water added shall be recorded on the concrete delivery ticket. In no case shall the total amount of water in the concrete be such as to exceed the specified water-cementitious materials ratio.

Water-reducing admixture may be added to the concrete when the measured slump is less than that specified.

Air-entraining admixture may be added to the concrete prior to discharge to increase the air content to that specified. The use of air-detraining admixtures is expressly prohibited.

Site introduced admixtures shall be added to the batch by means of a pipe or wand that can introduce the product to the center of the drum using an automated metering device. Only trained personnel shall be allowed to introduce admixtures at the jobsite. A method statement by the contractor for the site addition of admixtures shall be submitted and a record of jobsite additions shall be maintained and available at the project site at all times.

When any material is added to the concrete, the concrete batch shall be mixed for an additional 30 revolutions (or more if necessary) at the designated mixing speed so that the uniformity requirements of Annex 1 of AASHTO M 157 (ASTM C 94) are met. In this situation it is permissible to exceed the maximum of 100 revolutions total. The uniformity shall be monitored before placement of the concrete.

8.8 Delivery Tickets

With each batch of concrete, the concrete manufacturer shall provide to the Contractor a copy of the delivery ticket, on which shall be printed, stamped, or written the following information:

- Name of concrete manufacturer, and name or number of concrete production facility
- Serial number of delivery ticket
- Date
- Class or designation of the concrete
- Truck number
- Name of Contractor
- Name and address of project

- Time of batching, or of first mixing of cementitious materials and aggregates
- Time at which concrete discharge must be completed
- Moisture corrections for aggregate moisture
- Quantities of each mixture component
- Total batch volume
- Maximum water that may be added to the mix at the project
- Quantities of materials added at the site, including water and admixtures, if any
- Specified compressive strength (or other specified performance criterion)

Commentary on Guide Specification for High-Performance Concrete for Bridges

C1.0 Scope

The *Guide Specification* provides appropriate wording to specify each of the following criteria for high performance concrete: abrasion resistance, chloride ion penetration, compressive strength, creep, modulus of elasticity, freeze/thaw durability, scaling resistance, shrinkage, sulfate resistance, consistency, and alkali-silica reactivity. For a given bridge element, it is anticipated that the specifier will select at most four criteria. The performance criteria are given in Sections 4.1-4.11 of the specification, with guidance in the corresponding sections of the Commentary for modifying these criteria to suit local conditions.

Table C1.1 summarizes how to select criteria for various bridge elements. Table C1.2 summarizes the test methods and standards discussed in this guide specification.

C2.0 References

C3.0 Definitions

C4.0 Performance Requirements

Specify only the performance grade required for each characteristic, as additional requirements add to the cost of the material without necessarily providing additional performance benefits. In some cases, high performance by one criterion may even detract from the performance by another criterion. FHWA recommendations for the application of HPC Grades are shown in Table C4.0 for reference. In some cases these differ from the recommendations of this guide specification.

Note that FHWA's default age at test is 56 days. For certain jobs, other ages, or accelerated curing regimes, may be more appropriate to specify either in addition to or instead of 56 days. If so, they must be explicitly specified. Examples are given in the following sections.

Table C1.0-1 Which Criteria to Specify for Which Bridge Elements

Performance characteristics	Element			
	Deck	**Girder**	**Pier**	**Footing**
Abrasion resistance	If abrasion from snowplows is a concern, either specify abrasion resistance criterion for concrete or criteria for aggregate.	Do not specify.	If abrasion due to ice floes, floating debris, or boats is a concern, either specify abrasion resistance criterion for concrete or criteria for aggregate.	Do not specify.
Chloride Ion Penetration	If exposed to deicing salts or salt spray, specify maximum limit on AASHTO T 277.	If exposed to salt spray, specify maximum limit on AASHTO T 277.	If exposed to salt spray, specify maximum limit on AASHTO T 277.	Do not specify.
Compressive Strength	Specify strength(s) and age(s) if the structural behavior depends on having a certain strength; high early strength could increase risk of cracking.	Specify strengths for de-tensioning of prestressing strand, transport of precast elements to site, and opening of bridge to traffic.	Specify strength(s) and age(s) as determined by structural design. High strengths at early ages may increase risk of thermal cracking.	Specify strength(s) and age(s) as determined by structural design. High strengths at early ages may increase risk of thermal cracking.
Creep	Do not specify. Too low creep may contribute to cracking of the deck.	Specify maximum allowable creep to control prestress losses and long-term deflections.	May specify maximum allowable creep to control long-term deflections.	Do not specify.
Modulus of Elasticity	Do not specify. Too high modulus of elasticity may contribute to cracking of the deck.	Specify minimum allowable modulus of elasticity to control deflections.	May specify to limit deflections.	Do not specify.
Freeze/Thaw Durability	If exposed to freezing and thawing, specify a minimum durability factor for AASHTO T 161.	Do not specify.	Do not specify unless the pier is partially submerged in water or saturated soil subject to freezing.	Do not specify unless the foot-ing is in saturated soil subject to freezing.
Scaling Resistance	If exposed to deicing salts, specify a maximum visual rating on ASTM C 672.	Do not specify.	Do not specify.	Do not specify.
Shrinkage	Do not specify.	Do not specify.	Do not specify.	Do not specify.
Sulfate Resistance	Do not specify.	Do not specify.	If exposed to sulfate soils or ground-water, specify that cementitious material(s) and w/cm must meet requirements for severity of exposure.	If exposed to sulfate soils or groundwater, specify that cementitious material(s) and w/cm must meet requirements for severity of exposure.
Consistency	Allow contractor to select consistency to achieve consolidation. Specify variability limits.			
Alkali-silica reactivity	Ensure that aggregates used are not potentially reactive, or take appropriate control measures.			

Table C1.0-2. Test Methods and Standards Discussed in this Guide Specification

Application	AASHTO	ASTM	Other
Test methods			
Abrasion	T 6	C 131, C 779, C 944	
Chloride penetration	T 277	C 1202	
Compressive strength	T 22	C 39	
Cracking	pp 34		
Creep		C 512	
Modulus of elasticity		C 469	
Freeze thaw	T 161	C 666	
Salt scaling		C 672	
Shrinkage	T 160	C 157	
Sulfate resistance		C 1012	ACI 201
Slump	T 119	C 143	
SCC consistency			Slump flow, J-ring, column segregation
Alkali silica reaction		C 1293, C 1260, C 1567, C 295	
Alkali carbonate reaction		C 1105, C 295	
D-cracking	T 161	C 666	Iowa Pore Index Test, Washington, Hydraulic Fracture Test
Air content		C 231, C 457	Air void analyzer
Water content	T 318		
Cement content		C 1084	
Materials specifications			
Water	M 157	C 1602	
Cement	M 85, M 240	C 150, C 595, C 1157	
Supplementary cementitious materials	M 295, M 302, M 307	C 618, C 989, C 1240	
Aggregates	M 6, M 80	C 33	
Chemical admixtures	M 194, M 154	C 494, C 260, C 1582	
Ready-mixed concrete	M 157	C 94	
Quality			
QA QC systems			ACI 121, 363
			AASHTO QAGS
			FHWA FP-03
			NRMCA Publication 190
			PCI MNL 116

Table C4.0-1. FHWA Proposed Grades of Performance Characteristics for High-Performance Structural Concrete[6,7]

Performance characteristic[8]	Standard test method	FHWA HPC performance grade[9]		
		1	2	3
Freeze-thaw durability (x = relative dynamic modulus after 300 cycles)	AASHTO T 161 ASTM C 666 Procedure A	70% ≤ x < 80%	80% ≤ x < 90%	90% ≤ x
Scaling resistance (x = visual rating of the surface after 50 cycles)	ASTM C 672	3.0 ≥ x > 2.0	2.0 ≥ x > 1.0	1.0 ≥ x > 0.0
Abrasion resistance (x = average depth of wear in mm)	ASTM C 944	2.0 > x ≥ 1.0	1.0 > x ≥ 0.5	0.5 > x
Chloride penetration (x = coulombs)	AASHTO T 277 ASTM C 1202	2500 ≥ x > 1500	1500 ≥ x > 500	500 ≥ x
Alkali-silica reactivity (x = expansion at 56 d, %)	ASTM C 441	x ≤ 0.20	x ≤ 0.15	x ≤ 0.10
Sulfate resistance (x = expansion, %)	ASTM C 1012	x ≤ 0.10 at 6 months	x ≤ 0.10 at 12 months	x ≤ 0.10 at 18 months
Workability (x = slump, y = slump flow)	AASHTO T 119 ASTM C 143 and proposed slump flow test	x ≥ 6 in. (x ≥ 150 mm)	20 ≤ y < 24 in. (500 ≤ y < 600 mm)	y > 24 in. (y > 600 mm)
Strength (x = compressive strength)	AASHTO T 22 ASTM C 39	8 ≤ x < 10 ksi (55 ≤ x < 69 MPa)	10 ≤ x <14 ksi (69 ≤ x < 97 MPa)	x ≥ 14 ksi (x ≥ 97 MPa)
Elasticity (x = modulus of elasticity)	ASTM C 469	5 ≤ x < 6 * 10^6 psi (34 ≤ x < 41 GPa)	6 ≤ x < 7 * 10^6 psi (41 ≤ x < 48 GPa)	x ≥ 7 * 10^6 psi (x ≥ 48 GPa)
Shrinkage (x = microstrain)	AASHTO T 160 ASTM C 157	800 > x ≥ 600	600 > x ≥ 400	400 > x
Creep (x = microstrain/pressure unit)	ASTM C 512	0.52 ≥ x > 0.38/psi (75 ≥ x > 55/MPa)	0.38 ≥ x > 0.21/psi (55 ≥ x > 30/MPa)	x ≤ 0.21/psi (x ≤ 30/MPa)

These recommendations may differ from those made in this guide specification.

[6] Russell, H. G., Miller, R. A., Ozyildirim, H. C., and Tadros, M. K., *Compilation and Evaluation of Results from High Performance Concrete Bridge Projects—Final Report,* FHWA, U.S. Department of Transportation, 461 pp. To be published.

[7] This table does not represent a comprehensive list of all characteristics that good concrete should exhibit. It does list characteristics that can quantifiably be divided into different performance groups. Other characteristics should be checked.

[8] For non-heat cured products, all tests to be performed on concrete samples moist, submersion, or match cured for 56 days or until test age. For heat-cured products, all tests to be performed on concrete samples cured with the member or match cured until test age.

[9] A given HPC mix design is specified by a grade for each desired performance characteristic. For example, a concrete may perform at Grade 3 in strength and elasticity, Grade 2 in shrinkage and scaling resistance, and Grade 2 in all other categories.

C4.1 Abrasion Resistance

Abrasion resistance is significant for bridge decks that will be subjected to the action of truck traffic or snowplows, and for bridge substructure elements that will be in direct contact with ice floes, floating debris, and boat or ship traffic.

In general, good abrasion resistance is achieved by the use of high-strength concrete and a hard, abrasion-resistant aggregate. Proper finishing and curing of the concrete surface are essential. Specifications are normally based on requiring a given performance from the aggregate and setting a minimum strength. Field tests of abrasion resistance are not commonly required except for troubleshooting and evaluation of rain-damaged surfaces.

Aggregates should be pre-qualified, either based on historical performance or by testing. ASTM C 33 imposes a limit of 50% mass loss in the Los Angeles Abrasion test (AASHTO T 96) but this may be considered too high a value for high performance requirements. As indicated in Table C4.0, the FHWA provides criteria for field testing according to ASTM C 944, which is similar to ASTM C 779, Procedure B. It gives an indication of the relative wear resistance of concrete.

C4.2 Chloride Ion Penetration

Resistance to chloride ion penetration is significant for reinforced and prestressed concrete that will be exposed to chlorides. The most common sources of chlorides from the environment are deicing salts and seawater. Chlorides act as catalysts to the corrosion reactions.

If the crack widths are controlled by providing crack control reinforcement as specified in the *AASHTO LFRD Bridge Design Specifications,* the ability of the concrete to protect the steel from corrosion depends on the quality of the concrete and the cover thickness.

A maximum limit of 1500 coulombs (ASTM C 1202) is appropriate for most bridge elements that will be exposed to chlorides. Values of 1500 to 2500 coulombs would be appropriate for superstructure elements not expected to receive chloride exposure on a continuing basis. However, note that if deck joints are located directly over elements of the superstructure, eventually the joints will leak and salt water will drain onto the superstructure. If possible, locate deck joints elsewhere. If not, specify 1500-coulomb concrete throughout. This test is commonly used as an acceptance indicator. It should be noted that the scatter on the test is large, therefore imposing a limit of less than 1500 coulombs may result in rejection of acceptable concrete

Figures C4.1-1. Los Angeles abrasion test (AASHTO T 96).
Test measures degradation of aggregates resulting from a combination of abrasion, impact, and grinding in a rotating steel drum containing a specified number of steel spheres. As the drum rotates, a shelf plate picks up the sample and the steel spheres, carrying them around until they are dropped to the opposite side of the drum, creating an impact crushing effect. The contents then roll within the drum with an abrading and grinding action until the shelf plate picks up the sample and the steel spheres, and the cycle is repeated. After the prescribed number of revolutions, the contents are removed from the drum and the aggregate portion is sieved to measure the degradation as percent loss. (left: IMG16950, right: IMG16949)

Figure C4.2-1. Test setup for the rapid chloride permeability test (RCPT), also called Coulomb or electrical resistance test (ASTM C 1202). This test method consists of monitoring the amount of electrical current passed through slices of cores or cylinders during a 6-h period. A potential difference of 60 V dc is maintained across the ends of the specimen, one of which is immersed in a sodium chloride solution, the other in a sodium hydroxide solution. The total charge passed, in coulombs, is related to the resistance of the specimen to chloride ion penetration. (IMG16975)

Theoretically, cover thickness and concrete quality can be traded off against one another; however, most current models of service life for concrete structures do not include all of the mechanisms by which chloride ions penetrate concrete[10]. Thus it would be prudent to maintain the cover thicknesses given by the *AASHTO LFRD Bridge Design Specifications* and use high performance concrete as needed to prolong the service life.

Class F fly ash, some Class C fly ashes, calcined clay, slag cement, and silica fume all improve concrete's resistance to chloride ion penetration. However, all of these require sufficient curing time to develop the favorable microstructure that is required. Reducing the water-cementitious materials ratio improves the resistance to chloride ion penetration for all types of concrete. For bridge decks, latex-modified or other polymer-modified concrete overlays are an acceptable alternative to lowering the w/cm ratio.

Elevated curing temperatures increase the permeability and diffusivity of concrete even when they do not actually increase its porosity[11]. However, concretes containing supplementary cementitious materials are less sensitive to this effect than portland cement concretes[12].

Virginia DOT has adopted an accelerated curing regime for use with mixtures containing supplementary cementing materials[13]. Specimens are moist cured for 7 days at room temperature, followed by 3 weeks moist at 100°F (38°C). This regime is reported to be equivalent to 6 months normal temperature curing when tested in accordance with ASTM C 1202.

C4.3 Compressive Strength

The required compressive strength(s) must be determined by the Engineer to ensure that the structure is able to withstand the design load. If control of deflections and/or limitation of prestress losses is desired, explicitly specify the modulus of elasticity and the creep. Do not use strength as a surrogate for either of these properties. **In particular, strength should never be used as a surrogate for durability.** Although the FHWA performance grades specify the age of 56 days at which the concrete is to be tested, it may be necessary to specify different ages for some tests, such as to allow for early opening to traffic, or for de-tensioning the strand for precast prestressed elements. The Engineer should consider how a given test age would affect the design calculations and the behavior of the structure. Compressive strength commonly is used for quality control and quality assurance.

In general, the use of supplementary cementitious materials reduces the early-age strength and increases the later-age strength of the concrete as compared with a portland cement-only concrete. However, highly reactive pozzolans such as silica fume and calcined clay may produce comparable or even higher strengths at early ages. Class C fly ash may either increase or reduce early-

[10] Hooton, R. D. and McGrath, P. F., "Issues Related to Recent Developments in Service Life Specifications for Concrete Structures," *Chloride Penetration into Concrete,* RILEM, 1995, L.O. Nilsson and J.P. Olivier, Eds., pp. 388-397.

[11] Detwiler, Rachel J.; Kjellsen, Knut O., and Gjørv, Odd E., "Resistance to Chloride Intrusion of Concrete Cured at Different Temperatures," *ACI Materials Journal,* Vol. 88, No. 1, January-February 1991, pp. 19-24.

[12] Campbell, Glen M. and Detwiler, Rachel J., "Development of Mix Designs for Strength and Durability of Steam-Cured Concrete," *Concrete International,* Vol. 15, No. 7, pp. 37-39, 1993.

[13] Ozyildirim, C., *Permeability Specifications for High Performance Concrete Decks,* Transportation Research Record 1610, Concrete in Construction, Transportation Research Board, 1998, pp. 1-5.

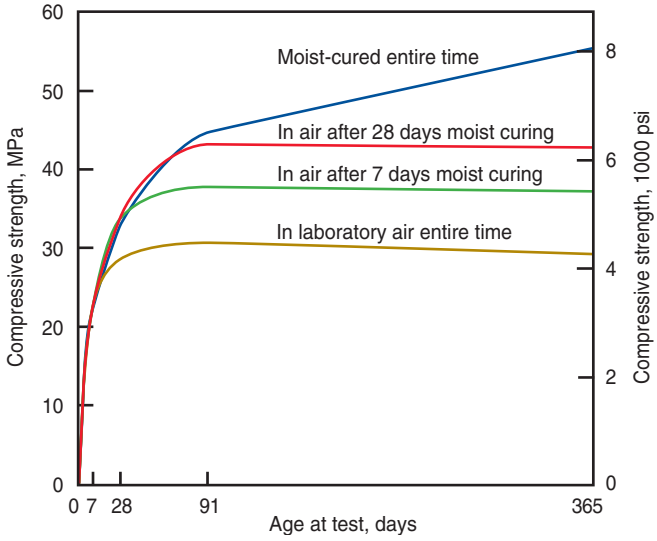


Figure C4.3-1. Compressive strength gain.
Concrete strength increases with age as long as moisture and a favorable temperature are present for hydration of cement.

age strength, depending on its composition, fineness and curing conditions.

Specify the latest age consistent with the requirements of the project, as most supplementary cementitious materials usually take more time to develop the desired properties. The later the specified age, the more flexibility the concrete producer has to meet the requirements at a reasonable cost and without introducing other potential problems.

The choice of coarse aggregate type can limit the ultimate strength of high-strength concrete. If reductions in the water-cementitious materials ratio do not result in increased strength, use a different aggregate or reduce the maximum size of the same aggregate, with appropriate adjustments to the mixture proportions.

C4.4 Creep

Creep is the long-term deformation of concrete under sustained load. Where deflections must be limited or prestress losses must be minimized, the Engineer should determine the allowable creep consistent with these criteria. Where cracking must be minimized (as on a bridge deck subjected to deicing salts), *higher* creep is desirable. Increasing strength and stiffness generally decrease creep, although there is no direct relationship among them[14]. Limits on creep may be imposed for prequalification but are not commonly used for quality monitoring.

The maximum limits for creep for each of the FHWA grades are given in Table C4.0-1. These are based on loading at the age of 28 days, the age assumed in the CEB model. The Engineer may determine that some other age is of more interest. In that case, specify that age in addition to or instead of 28 days. Equations to predict creep are given in ACI 209. Decreasing the required creep coefficient will mean that compressive strengths will increase.

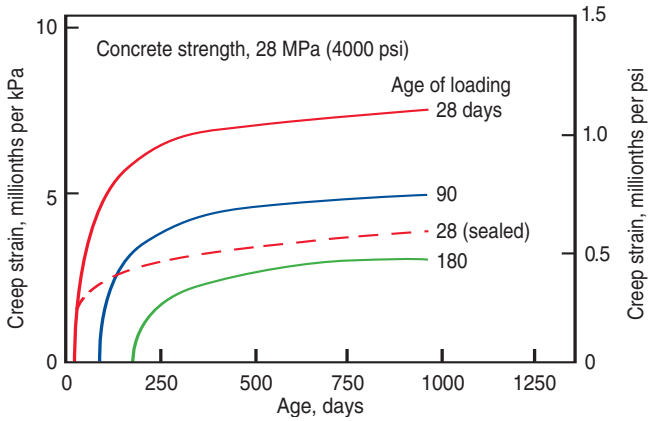


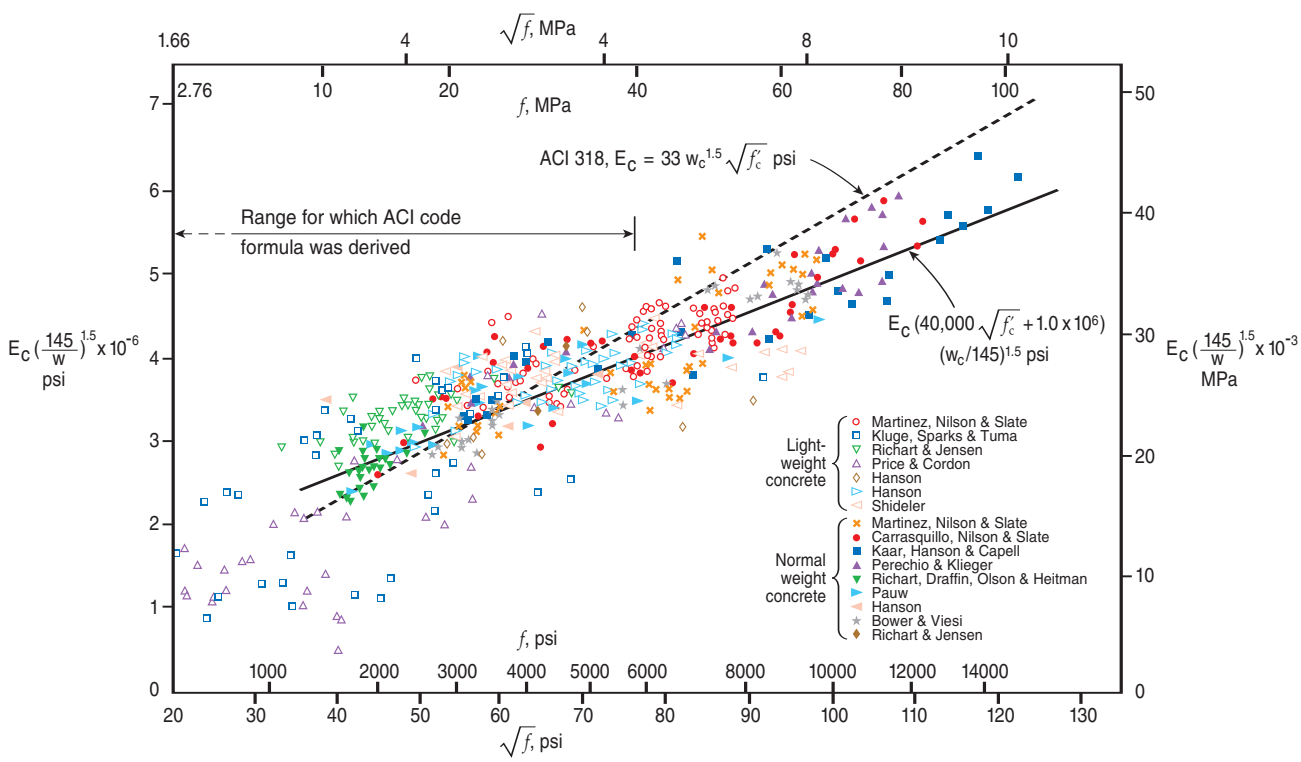
Figure C4.4-1
(left) Creep test. (IMG61175)
(right) Relationship of time and age of loading to creep of two different strength concretes. Specimens were allowed to dry during loading, except for those labeled as sealed (Russell and Corley 1977).

[14] Neville, A.M., *Properties of Concrete,* 4th Edition, 1996, New York: John Wiley & Sons, 844 pp.

C4.5 Modulus of Elasticity

The required modulus of elasticity is determined by the Engineer. In general, it would not be specified at all except for members in which control of deflections is paramount. A high modulus of elasticity is desirable in such applications as tall bridge piers and towers and for long spans where deflections must be minimized. A low modulus of elasticity is desirable when stresses (and cracking) due to restraint of volume changes must be minimized. Limits on modulus may be imposed for prequalification but are not commonly used for quality monitoring.

High modulus of elasticity usually accompanies high compressive strength, although the two are not directly proportional. The modulus of elasticity is dependent on the properties of the coarse aggregate and the proportion of the aggregate in the concrete, as well as on the modulus of elasticity of the cement paste and strength of the bond between paste and aggregate[15]. The strength of the cement paste and the paste-aggregate bond are of major importance to the concrete strength, while the strength of the aggregate matters only when it imposes an upper limit on the strength of the concrete. Stiff aggregates such as basalt confer higher moduli than limestones, which in turn confer higher moduli than lightweight aggregates. Specifying the largest practical maximum size of aggregate and a favorable grading can increase the volume of coarse aggregate in a concrete mixture, which will tend to increase the modulus of elasticity when using an aggregate with a high modulus of elasticity. However, increasing the coarse aggregate size may result in reduced strength in high-strength concrete mixtures.



Note: f is the measured compressive strength of concrete.

Figure C4.5-1. Modulus of elasticity versus concrete strength (ACI 363, Figure 5.3)

[15] Neville, A.M., *Properties of Concrete,* 4th Edition, 1996, New York: John Wiley & Sons, 844 pp.

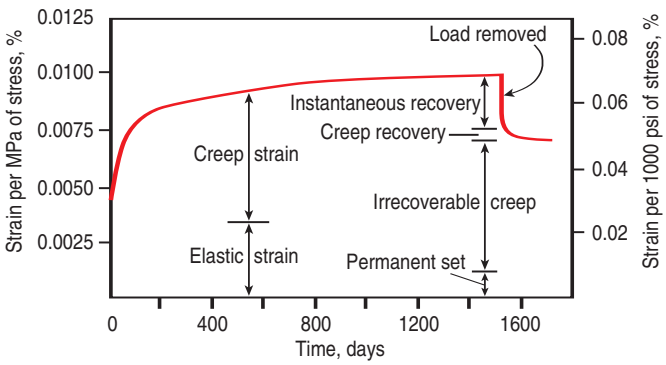


Figure C4.5-2. Combined curve of elastic and creep strains showing amount of recovery. Specimens (cylinders) were loaded at 8 days immediately after removal from fog curing room and then stored at 21°C (70°F) and 50% RH. The applied stress was 25% of the compressive strength at 8 days (Hansen and Mattock 1966).

C4.6 Freeze/Thaw Durability

Where concrete will be exposed to freezing and thawing under conditions of saturation or near-saturation, a durability factor of 90% as determined by ASTM C 666 Procedure A is recommended. Some authorities use limits between 80 and 95%. If concrete is not exposed to freezing cycles, it is not necessary to specify a freeze/thaw durability grade. Limits on freeze/thaw durability may be imposed for prequalification, but quality monitoring should be based on measuring the air content of the mixture.

In general, freeze/thaw durability is conferred by the presence of a finely distributed system of air voids throughout the concrete, adequate strength, and proper attention to consolidation and curing practices. Recommended total air contents as given in Table C4.6-1 may be used for quality assurance purposes; however, the total air content should be correlated with the spacing factor and specific surface as determined by ASTM C 457, as it is not the total volume of air that confers durability. Consideration may be given to using the Air Void Analyzer to monitor variability in the air void system in fresh concrete.

Tests such as AASHTO T 152 (ASTM C 231) and AASHTO T 196 (ASTM C 173) measure only the total air content of fresh concrete. The concrete should be tested at the point of placement, as air can be lost during transportation (particularly pumping), placement, and consolidation.

Tests such as ASTM C 457 are used to determine parameters related to the quality of the air-void system, such as the spacing factor and specific surface in hardened concrete. An air-void spacing factor of 0.008 in. (0.20 mm) or less and specific surface of 600 in^2/in^3 (24 mm^2/mm^3) or greater usually will result in satisfactory freeze/thaw performance. The Air Void Analyzer may be considered for use in monitoring the variability of the air void system in fresh concrete, although its correlation with C 457 tests is still to be proven.

It may be useful for purposes of control to determine the air content at the concrete plant. A correlation between air contents at the plant and the site may help prevent rejection of loads or delays.

Figure C4.6-1.
(left) Concrete beams (3x3x11¼ in.) in a freeze-thaw chamber. Specimens are repeatedly frozen and thawed in water and can be tested for changes in length, weight, and dynamic modulus of elasticity. ASTM C 666. Procedure A. (IMG2081)
(right) ASTM C 457. Using a polished section of a concrete sample, the air-void system is documented by making measurements using a microscope. The information obtained from this test includes the volume of entrained and entrapped air, its specific surface (surface area of the air voids), the spacing factor, and the number of voids per linear distance. (IMG64241)

Table C4.6-1. Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Maximum Sizes of Aggregates (ACI 211.1 Table 6.3.3).

Water, lb/yd³ of concrete for indicated nominal maximum sizes of aggregate								
Slump, in.	**⅜ in.***	**½ in.***	**¾ in.***	**1 in.***	**1½ in.***	**2 in.*†**	**3 in.†‡**	**6 in.†‡**
Non air-entrained concrete								
1 to 2	350	335	315	300	275	260	220	190
3 to 4	385	365	340	325	300	285	245	210
6 to 7	410	385	360	340	315	300	270	—
More than 7*	—	—	—	—	—	—	—	—
Approximate amount of entrapped air in non-entrained concrete, percent	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
1 to 2	305	295	280	270	250	240	205	180
3 to 4	340	325	305	295	275	265	225	200
6 to 7	365	345	325	310	290	280	260	—
More than 7*	—	—	—	—	—	—	—	—
Recommended average♦ total air content, percent for level of exposure:								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5■★	1.0■★
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5■★	3.0■★
Severe exposure**	7.5	7.0	6.0	6.0	5.5	5.0	4.5■★	4.0■★

* The quantities of mixing water given for air-entrained concrete are based on typical total air content requirements as shown for "moderate exposure" in the table above. These quantities of mixing water are for use in computing cement contents for trial batches at 68 to 77ºF (20 to 25ºC). They are maximum for reasonably well-shaped angular aggregate graded within limits of accepted specifications. Rounded aggregate generally will require 30 lb (14 kg) less water for non-air-entrained and 25 lb (11 kg) less for air-entrained concretes. The use of water-reducing chemical admixtures, ASTM C 494, also may reduce mixing water by 5% or more. The volume of the liquid admixtures is included as part of the total volume of the mixing water. The slump values of more than 7 in. (175 mm) are obtained only through the use of water-reducing chemical admixture; they are for concrete containing nominal maximum size aggregate not larger than 1 in. (25 mm).

† The slump values for concrete containing aggregate larger than 1½ in. (37.5 mm) are based on slump tests made after removal of particles larger than 1½ in. (37.5 mm) by wet screening.

‡ These quantities of mixing water are for use in computing cement factors for trial batches when 3 in. (75 mm) or 6 in. (150 mm) nominal maximum size aggregate is used. They are average for reasonably well-shaped coarse aggregates, well-graded from coarse to fine.

♦ Additional recommendations for air-content and necessary tolerances on air content for control in the field are given in a number of ACI documents, including ACI 201, 345, 318, 301, and 302. ASTM C 94 for ready mixed concrete also gives air-content limits. The requirements in other documents may not always agree exactly, so in proportioning concrete consideration must be given to selecting an air content that will meet the needs of the job and also meet the applicable specifications.

■ For concrete containing large aggregates that will be wet screened over the 1½ in. (37.5 mm) sieve prior to testing for air content, the percentage of air expected in the 1½ in. minus material should be tabulated in the 1½ in. (37.5 mm) column. However, initial proportioning calculations should include the air content as a percent of the whole.

★ When using large aggregate in low cement factor concrete, air entrainment need not be detrimental to strength. In most cases the mixing water requirement is reduced sufficiently to improve the water-cement ratio and to thus compensate for the strength-reducing effect of air-entrained concrete. Generally, therefore, for these large nominal maximum sizes of aggregate, air contents recommended for extreme exposure should be considered even though there may be little or no exposure to moisture and freezing.

**These values are based on the criteria that 9% air is needed in the mortar phase of the concrete. If the mortar volume will be substantially different from that determined in the recommended practice, it may be desirable to calculate the needed air content by taking 9% of the actual mortar volume.

Figure C4.6-2. Bridge damaged by freeze-thaw attack. (IMG13301)

C4.7 Scaling Resistance

Scaling resistance is necessary when the concrete will be subjected to deicing salts, in which case specify a visual rating of 1 or less as measured by ASTM C 672. If the concrete will not be exposed to de-icing salts, then no specification for scaling resistance is necessary. Limits on scaling resistance may be imposed for prequalification tests, but acceptance monitoring should be based on measuring the air content of the mixture.

ASTM C 672 requires a moist curing period of 14 days before a 14-day drying period. It may be appropriate to specify a different curing period to reflect the curing anticipated in service. ASTM C 672 uses $CaCl_2$ as the deicing chemical unless a different chemical is specified. If the Owner routinely uses a different chemical for this purpose, the chemical used should be specified.

Scaling resistance is obtained by a suitable air-void system as described in Section C4.6, by limiting the water-cementitious materials ratio to a maximum of 0.45, by incorporating a minimum of 564 lb/yd^3 (335 kg/m^3) of cementitious material, by limiting the proportion of supplementary cementitious materials as discussed in Section 5.1, and by proper attention to finishing and curing. Wait until all bleeding has stopped before finishing the concrete so as to avoid trapping bleed water and creating a plane of weakness just under the finished surface. Avoid over-finishing, which can remove air voids from the near-surface concrete where they are most needed. Avoid the use of finishing aids, which consist primarily of water.

Care also should be taken to provide adequate drainage so that water does not remain on the surface of the concrete.

Figure C4.7-1. Deicer-scaling specimens (3x6x15 in.) are frozen and thawed with a salt solution on the surface. A mortar dike holds the solution in place. Specimens are rated by the degree of scaling (ASTM C 672). (IMG2082)

C4.8 Shrinkage

Shrinkage of concrete, as described below, is related to moisture loss to the environment, or to consumption of water in the hydration process. These processes are cumulative; therefore reducing any one of them will reduce the total shrinkage of the system.

There are no standard tests that measure total shrinkage from the time that the concrete is first mixed. AASHTO T 160 (ASTM C 157) measures drying shrinkage after a 28-day (or other specified period) moist cure. Typical values are in the range 400 to 800 microstrain. Specifications requiring lower values will be difficult to meet. This method does not include the effects of autogenous shrinkage or drying in the plastic stage, even though they may be more significant causes of cracking in low water-cementitious materials ratio concrete[16]. AASHTO provisional practice PP 34, "Practice for Estimating the Cracking Tendency of Concrete," (or ASTM C 1581) describes a test method that indicates the cracking tendency from the time of casting.

[16] Holt, Erika E., "Where Did These Cracks Come From?" *Concrete International*, Vol. 22, No. 9, September 2000, pp. 57-60.

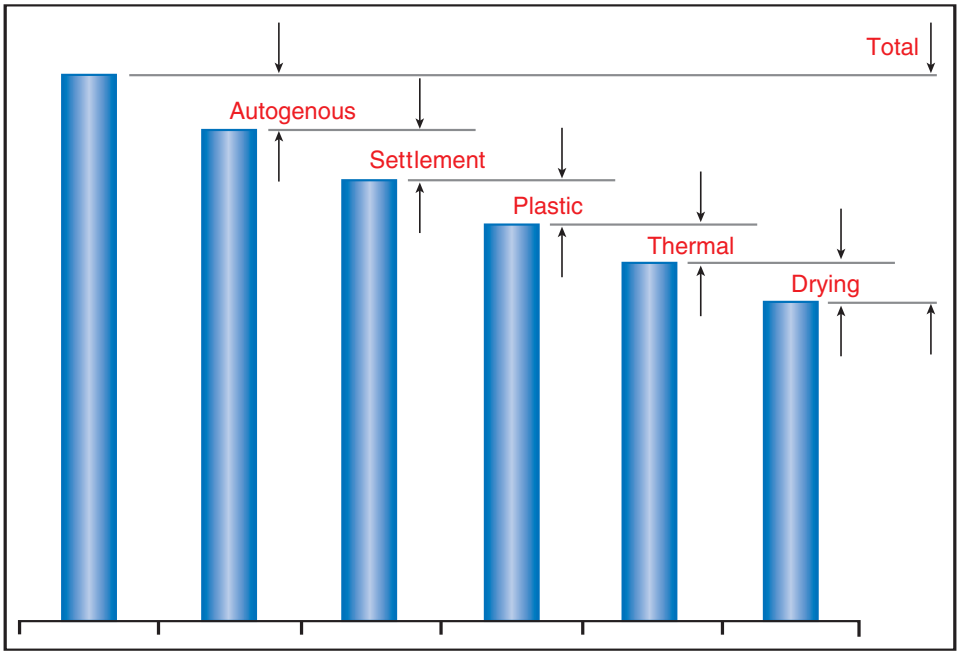


Figure C4.8-1. Total shrinkage is a sum of the individual mechanisms. Minimizing any or all of the shrinkage mechanisms will reduce the risk of cracking.

Figure C4.8-2. Ring shrinkage specimen marked where a crack has occurred. AASHTO PP 34 and ASTM C 1581 are restrained shrinkage tests used to determine the effects of concrete variations on cracking tendency using concrete cast around steel. (IMG16976)

The best way to minimize both plastic shrinkage and drying shrinkage is to reduce the paste content of the mixture, and to pay proper attention to curing. A clause limiting shrinkage determined using ASTM C 157 is included in the guide specification should the Engineer require it for pre-qualification. However, to minimize cracking, the Engineer should rather specify the required curing procedures, inspection, and crack repair methods. In addition, the mix proportions of the concrete should be such as to maximize the aggregate content (since it is the paste that shrinks). Shrinkage-reducing admixtures (discussed in section C5.4) also may be used.

The need to reduce paste content to reduce plastic and drying shrinkage, while increasing water cement ratio to reduce autogenous shrinkage, may be counter to the need for high strength and or low permeability of the system. "High performance" concrete is therefore likely to be at higher risk of cracking, meaning that greater care needs to be taken with detailing and workmanship.

C4.8.1 Plastic Shrinkage

Plastic shrinkage occurs when water evaporates from the surface faster than bleed water rises to the surface. Plastic shrinkage cracking therefore can be prevented by preventing the evaporation of water from the concrete. In general, high performance concrete is particularly vulnerable to plastic shrinkage cracking because it exhibits little or no bleeding. However, altering the mix proportions to encourage bleeding is not an appropriate means to limit plastic shrinkage.

1. Measures to minimize the occurrence of plastic shrinkage include the following:Have sufficient personnel, equipment, and supplies available to place and finish the concrete promptly. Cover the concrete with wet burlap, polyethylene sheeting, or building paper, or use an evaporation retardant between finishing operations to prevent drying.
2. Start curing the concrete as soon as possible.
3. Dampen the subgrade, formwork, and reinforcement before placing concrete.
4. Use fog sprays, temporary windbreaks, and sunshades as needed, especially under hot, dry, or windy conditions.
5. Place concrete in the late afternoon or at night.
6. Synthetic fibers may help to control plastic shrinkage cracking.

Figure C4.8.1-1. Typical plastic shrinkage cracks, caused by rapid loss of mix water while the concrete is still plastic (IMG12267)

C4.8.2 Autogenous Shrinkage

Autogenous shrinkage is the volume change that occurs when there is no moisture loss to the surrounding environment. It takes place because the volume of the hydration products of cement is less than that of the unhydrated cementitious material(s) and water from which they form. It is most noticeable in concrete in which the water-cementitious materials ratio is less than about 0.42[17]. Shrinkage that takes place within the first 24 hours of placement is of greatest concern because at these early ages the concrete has the lowest strain capacity and is most vulnerable to cracking.

Since no moisture loss is involved in autogenous shrinkage, efforts to prevent drying at the construction stage cannot prevent autogenous shrinkage. Concrete mix proportions and ingredients will have the most significant influence. Measures to minimize autogenous shrinkage include:

1. Minimizing the cementitious paste content (that is, maximizing the aggregate content).
2. Increasing the water-cementitious materials ratio.
3. Avoiding the use of large quantities of excessively fine cementitious materials.
4. Using cement with a lower C_3A content.

Note that some of these measures may offset those needed to meet the strength and/or durability requirements.

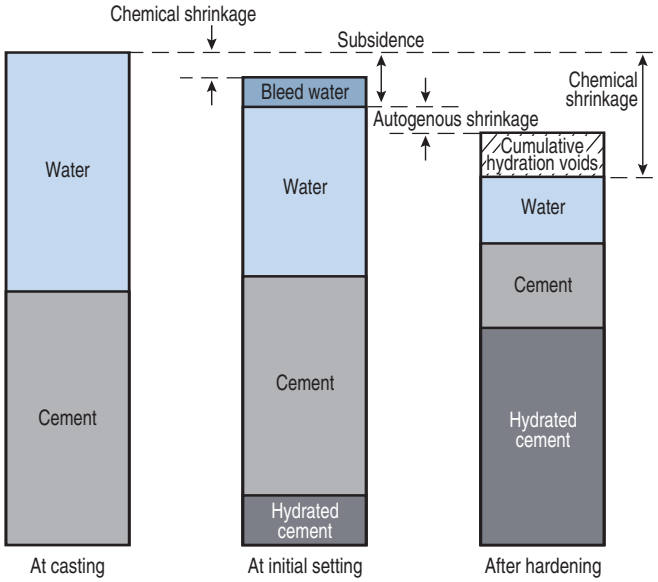


Figure C4.8.2-1. Volumetric relationship between subsidence, bleed water, chemical shrinkage, and autogenous shrinkage. Only autogenous shrinkage after initial set is shown. Not to scale.

[17] Holt, Erika E., *Early Age Autogenous Shrinkage of Concrete,* VTT Publication 446, Technical Research Center of Finland, Espoo, 2001, 194 pp. Also available through Portland Cement Association as LT257.

C4.8.3 Drying Shrinkage

Drying shrinkage occurs due to loss of moisture from the concrete after final set and continues to take place for weeks or months after placement. Concrete is particularly vulnerable to the development of drying shrinkage stresses immediately after formwork removal because its tensile strength may be low and drying can be severe, particularly if the concrete temperature is greater than the ambient temperature. It is therefore important to ensure that concrete is prevented from rapid drying after formwork is removed, by means of applying curing compounds, fog sprays, wet burlap and/or shading.

Drying shrinkage is relevant to long-term deformations. It should be noted that massive elements such as piers dry out very slowly, and that drying shrinkage therefore plays an insignificant role in their long-term deformation. In slender elements, drying affects long-term deformations or prestress losses. However, the creep test (ASTM C 512) includes the combined effects of drying and long-term loading.

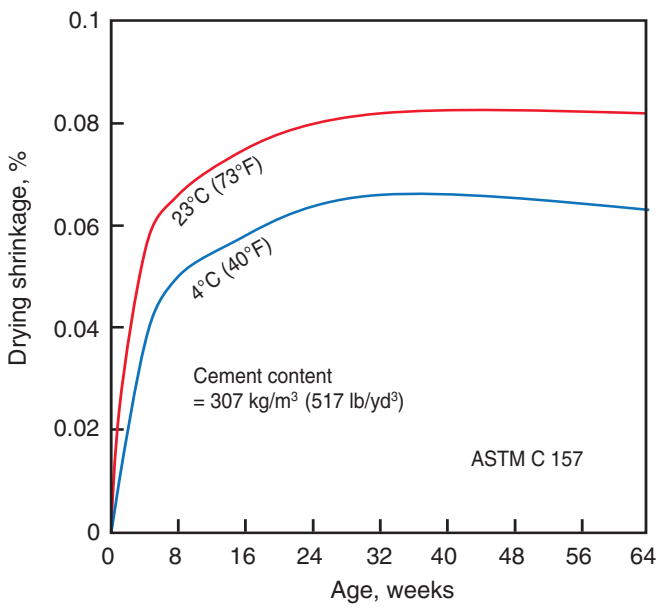


Figure C4.8.3-1. Effect of initial curing on drying shrinkage of portland cement concrete prisms. Concrete with an initial 7-day moist cure at 4°C (40°F) had less shrinkage than concrete with an initial 7-day moist cure at 23°C (73°F). Similar results were found with concretes containing 25% fly ash as part of the cementing material (Gebler and Klieger 1986).

Shrinkage reducing chemical admixtures are addressed in Section C5.4.

C4.9 Sulfate Resistance

Sulfate attack is particularly prevalent in arid regions where naturally occurring sulfate minerals present in soils and ground waters are in contact with structures. In North America, these areas are located primarily in the western United States and the prairie provinces of Canada. The necessary conditions for sulfate attack are well established and preventive measures can be taken to provide the needed service life[18].

Although the severity of sulfate attack generally is defined in terms of sulfate concentration in the soil or groundwater, the cations present have a significant effect on the severity of attack, with magnesium sulfate the most aggressive, calcium sulfate the least aggressive, and sodium sulfate of intermediate aggressiveness.

In sulfate-bearing soil or groundwater, use AASHTO M 85 (ASTM C 150) Type II or Type V cement, AASHTO M 240 (ASTM C 595) Type IS or Type IP, ASTM C 1157 Type MS (moderate sulfate resistant) or Type HS (high sulfate resistant) cement, or a combination of portland cement with sufficient Class F fly ash, slag cement, and/or silica fume to provide the degree of sulfate resistance required (ACI 201). In addition, limit the water-cementitious materials ratio to the values shown in Table C4.9-1. This is a criterion that should be applied during pre-qualification.

There are no standard test methods to assess the sulfate resistance of concrete. ASTM C 1012 and ASTM C 452 are methods used for testing cements.

[18] DePuy, "Chemical Resistance of Concrete," *Significance of Tests and Properties of Concrete and Concrete-Making Materials,* STP 169C, Paul Klieger and Joseph F. Lamond, Editors, American Society for Testing and Materials, Philadelphia, PA, 1994, pp. 263-281.

Figure C4.9.1. Concrete deterioration due to sulfate attack. (IMG16953)

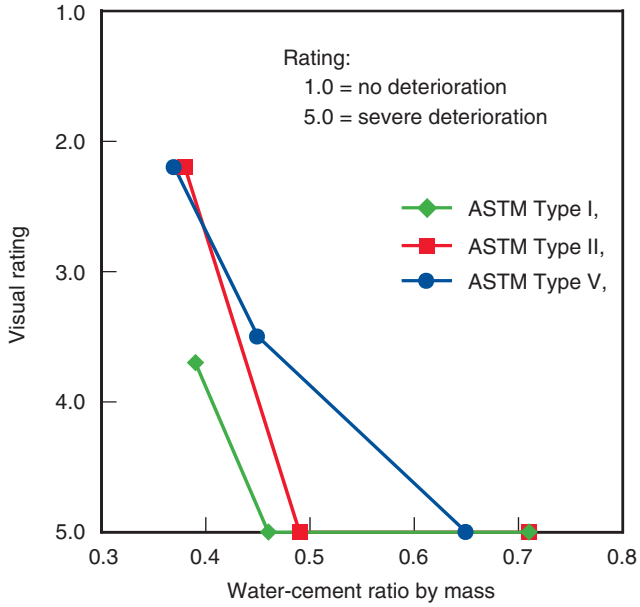


Figure C4.9.2. Average 16-yr ratings of concrete beams in sulfate soils for three portland cements at various water-cement ratios (Stark 2002).

Table C4.9-1. Requirements for Concrete Exposed to Sulfates in Soil or Water

Sulfate exposure	Sulfate (SO_4) in soil, % by mass	Sulfate (SO_4) in water, ppm	Cement type*			Maximum w/cm-ratio, by mass
	ASTM C 1580		ASTM C 150	ASTM C 595	ASTM C 1157	
Negligible	Less than 0.10	Less than 150	No special type required			—
Moderate (includes seawater)	0.10 to 0.20	150 to 1500	II	IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS)	MS	0.50
Severe	0.20 to 2.00	1500 to 10,000	V		HS	0.45
Very severe	Over 2.00	Over 10,000	V		HS	0.40

* Pozzolans and slag that have been determined by test or service record to improve sulfate resistance may also be used.

Adapted from ACI 318 (2005).

C4.10 Consistency

Monitoring consistency is a valuable tool for assessing the between-batch uniformity of concrete. It may be preferred to allow the contractors to choose a slump that is suitable for their equipment, and to impose a limit on variability between batches. Consistency of normal concrete is most commonly measured using the slump test.

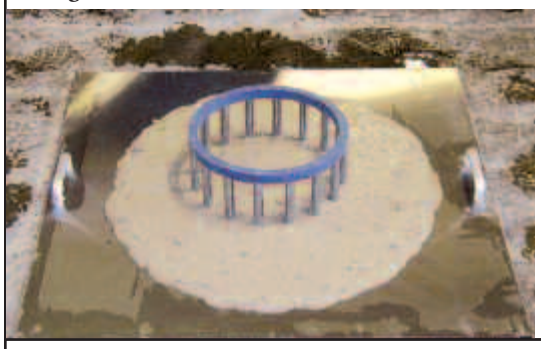
Self-consolidating concrete (SCC) is not the same as conventional flowing concrete produced using a high-range water-reducing admixture. A number of test methods are under consideration by ASTM to characterize the placement properties of SCC, including the "slump flow" test, "Column Segregation" test, and "J-ring" test[19]. The slump flow test is conducted using

Slump Flow

Test is performed similar to the conventional slump test (ASTM C 143) using the Abrams cone (use of inverted cone possible). However, instead of measuring the slumping distance vertically, the mean spread of the resulting concrete patty is measured horizontally. This number is recorded as the slump flow.

Measured characteristic: Filling ability (deformability) & stability.

J-Ring



The J-Ring consists of a ring of reinforcing bar such that it will fit around the base of a standard slump cone. The slump flow with and without J-Ring is measured, and the difference calculated.

Measured characteristic: Passing ability.

Column Segregation

Test evaluates static stability of a concrete mixture by quantifying aggregate segregation. A column is filled with concrete and allowed to sit for awhile after placement. The column is then separated into three or four pieces. Each section is removed individually and the concrete from that section is washed over a No. 4 sieve and the retained aggregate weighed. A non-segregating mix will have a consistent aggregate mass distribution in each section. A segregating mix will have higher concentrations of aggregate in the lower sections.

Measured characteristic: Stability.

Figure C4.10-1. QCQA tests for SCC. (IMG16973, IMG16972, IMG16971)

[19] Assaad, J., Khayat, K.H. and Daczko, J., "Evaluation of Static Stability of Self-Consolidating Concrete," *ACI Materials Journal,* Vol. 101, Issue No. 3, May-June 2004, pp. 207-215.

the slump cone as described in AASHTO T 119 (ASTM C 143); however, rather than measuring the vertical subsidence or slump, the average diameter of the disk that spreads out on lifting the inverted cone is measured[20]. Typical flow values for SCC are in the range of 20 to 30 in. (500 to 750 mm).

SCC mixtures usually contain a high-range water-reducing admixture either alone or in combination with a viscosity-modifying admixture. To achieve desirable placement properties, SCC mixtures may also incorporate higher cementitious materials contents, low water-cementitious materials ratios, and smaller sized aggregates. The aggregate grading and moisture content are of particular importance and must be controlled carefully for consistent results.

The performance grade for consistency (Table C4.10-1) should be specified as required to produce acceptable consolidation. In most bridge members, it would be more appropriate to specify a slump of 3 to 4 in. Grades 2 and 3 are self-consolidating concrete, which may be appropriate for precast members or (for Grade 3) in repairs. It is recommended that the Engineer not specify a consistency performance grade but allow the Contractor to propose it if he desires. The Contractor must demonstrate adequate performance of the concrete as placed in the trial batches and mockups.

			Slump flow		
			< 22" (550 mm)	22-26" (550-650 mm)	> 26" (650 mm)
Member characteristics	Reinforcement level	Low			
		Medium			
		High			
	Element shape intricacy	Low			
		Medium			
		High			
	Surface finish importance	Low			
		Medium			
		High			
	Element length	Low			
		Medium			
		High			
	Wall thickness	Low			
		Medium			
		High			
	Placement energy	Low			
		Medium			
		High			

= Not recommended; potential problem area.

= Recommended

Figure C4.10-2. Slump flow targets. Adapted from Constantiner and Daczko (2002)[21].

[20] Petersson, Orjan, et. al., *Testing SCC,* presented at First North American Conference on the Design and Use of Self-Consolidating Concrete, ACBM, Northwestern University, 2002.

[21] Constantiner, D. and Daczko, J., "Not All Applications are Created Equal; Selecting the Appropriate SCC Performance Targets," *Conference Proceedings: First North American Conference on the Design and Use of Self-Consolidating Concrete,* November 12-13, 2002.

Table C4.11-1. Test Methods for Alkali-Silica Reactivity (adapted from Farny and Kosmatka 1997[22])

Test name	Purpose	Type of test	Criteria	Comments
ASTM C 295, Petrographic examination of aggregates for concrete	To evaluate possible aggregate reactivity through petrographic examination	Visual and microscopic examination of prepared samples—sieve analysis, microscopy, scratch or acid tests	Aggregate is potentially reactive if its constituents exceed any of the following[23]: • Optically strained, microfractured, or microcrystalline quartz, 5.0%; • Chert or chalcedony, 3.0%; • Tridymite or crystobalite, 1.0% • Opal, 0.5%; • Natural volcanic glass in volcanic rocks, 3.0%	Usually includes optical microscopy. Also may include XRD analysis, differential thermal analysis, or infrared spectroscopy—see ASTM C 294 for descriptive nomenclature
ASTM C 441, Effectiveness of mineral admixtures or GBFS in preventing excessive expansion of concrete due to alkali-silica reaction	To determine effectiveness of supplementary cementing materials in controlling expansion from ASR	Mortar bars—using Pyrex glass as aggregate—stored over water at 37.8°C (100°F) and high relative humidity	Per ASTM C 989, minimum 75% reduction in expansion or 0.020% maximum expansion or per ASTM C 618, comparison against low-alkali control	Not recommended. Highly reactive artificial aggregate may not represent real aggregate conditions. Pyrex contains alkalies.
ASTM C 856, Petrographic examination of hardened concrete	To outline petrographic examination procedures for hardened concrete—useful in determining condition or performance	Visual (unmagnified) and microscopic examination of prepared samples	See measurement—this examination determines if ASR reactions have taken place and their effects upon the concrete. Used in conjunction with other tests.	Specimens can be examined with stereomicroscopes, polarizing microscopes, metallographic microscopes, and scanning electron microscope
ASTM C 1260 (AASHTO T 303), Potential alkali-reactivity of aggregates (mortar-bar method)	To test the potential for deleterious alkali-silica reaction of aggregate in mortar bars	Immersion of mortar bars in alkaline solution at 80°C (176°F)	If greater than 0.10%, go to supplementary test procedures; if greater than 0.20%, indicative of potential deleterious expansion.	Very fast alternative to C 227. Useful for slowly reacting aggregates or those that produce expansion late in the reaction.
ASTM C 1293, Determination of length change of concrete due to alkali-silica reaction (concrete prism test)	To determine the potential ASR expansion of cement-aggregate combinations	Concrete prisms stored over water at 38°C (100.4°F)	Per Appendix X1, potentially deleteriously reactive if expansion equals or exceeds 0.04% at one year	Requires long test duration for meaningful results. Use as a supplement to C 227, C 295, C 289, C 1260, and C 1567. Similar to CSA A23.2-14A.
ASTM C 1567, Potential alkali-silica reactivity of combinations of cementitious materials and aggregate (accelerated mortar-bar method)	To test the potential for deleterious alkali-silica reaction of cementitious materials and aggregate combinations in mortar bars	Immersion of mortar bars in alkaline solution at 80°C (176°F)	If greater than 0.10%, indicative of potential deleterious expansion; use C 1293 to confirm	Very fast alternative to C 1293. Useful for slowly reacting aggregates or those that produce expansion late in the reaction.
Accelerated concrete prism test (modified ASTM C 1293)	To determine the potential ASR expansion of cement-aggregate combinations	Concrete prisms stored over water at 60°C (140°F)	Potentially deleteriously reactive if expansion equals or exceeds 0.04% at 91 days	Fast alternative to C 227. Good correlation to ASTM C 227 for carbonate and sedimentary rocks.

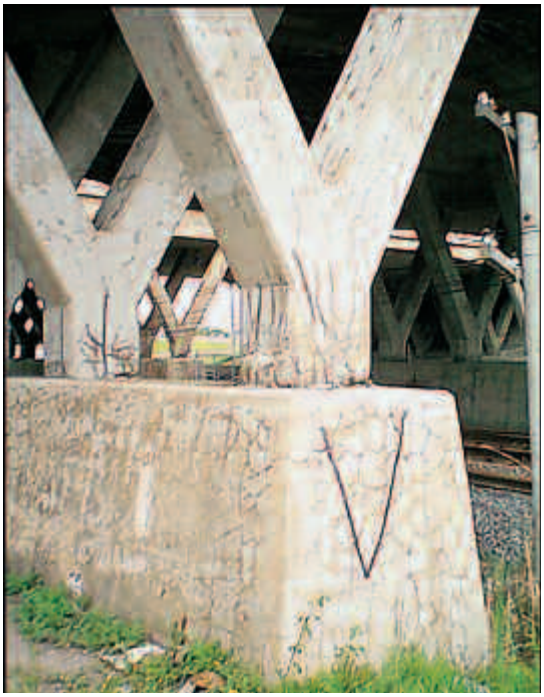
[22] Farny, James A., and Kosmatka, Steven H., *Diagnosis and Control of Alkali-Aggregate Reactions,* IS413, Portland Cement Association, 1997, 24 pages.

[23] *Guide Specification for Concrete Subjected to Alkali-Silica Reactions,* Portland Cement Association, IS415, September 1998, 8 pages.

C4.11 Alkali-Silica Reactivity

There are currently two ASTM test methods that use the actual job materials in combination. In ASTM C 1567, the aggregates are crushed and sieved to a specified grading and mortar bars made using the proposed dosages of the proposed cementitious materials. The mortar bars are submerged in a NaOH solution at 100°F (38°C) and the expansion after 14 days' exposure is the criterion for determining the effectiveness of the control measures. In ASTM C 1293, concrete prisms are made from the job materials using the aggregates sieved and recombined to the specified grading. The specimens are stored over water in a closed container at 100°F (38°C) for two years. While the duration of this test makes it impractical to use on a job in most cases, materials suppliers or state highway departments already may have data on the performance of the proposed job materials. If so, these data may be used in preference to data from ASTM C 1260. This is a prequalification criterion.

The basic premise of ASTM C 441 is that low-alkali cement provides adequate control of expansions due to alkali-silica reaction. This is not true of all aggregates. Thus, the test may be used to compare the relative effectiveness of different combinations of cementitious materials, but not to evaluate the acceptability of a given combination of cementitious materials. ASTM C 1567 is the recommended test method for that purpose.



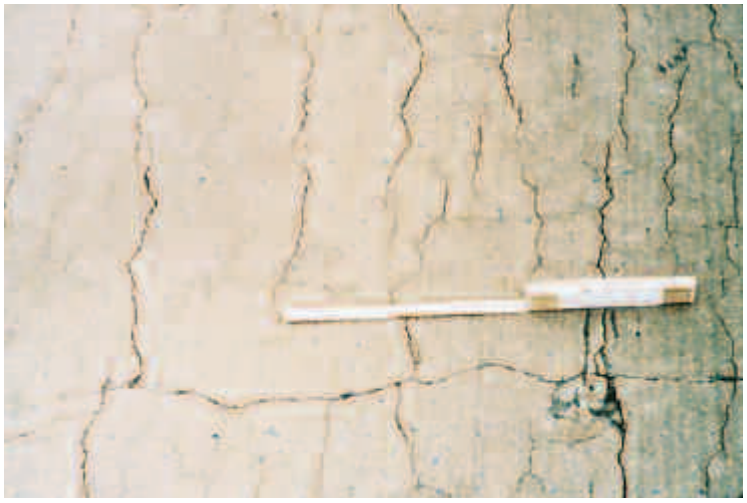


Figure C4.11-1. Cracking of concrete from alkali-silica reactivity. (IMG12421, IMG13049)

C5.0 Materials

C5.1 Cementitious Materials

Cementitious materials when used in combinations should be evaluated to determine their effect on those concrete properties which the Engineer has determined to be significant for the project, including water demand, setting time, heat development, strength development, shrinkage characteristics, and early stiffening[24].

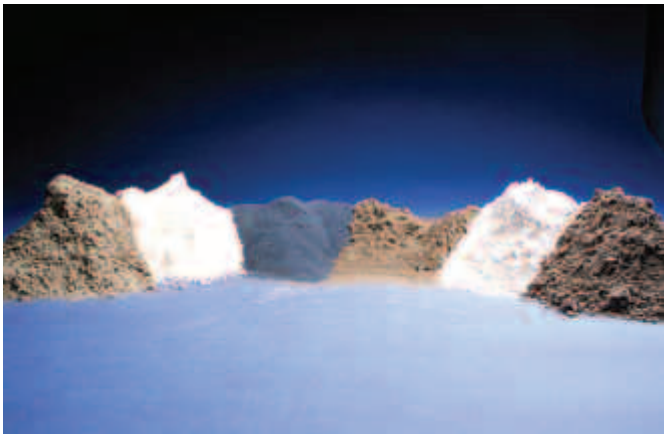


Figure C5.1-1. Supplementary cementitious materials. From left to right, fly ash (Class C), metakaolin (calcined clay), silica fume, fly ash (Class F), slag, and calcined shale. (IMG16974)

C5.2 Aggregates

C5.2.1 Grading and Impurities

Proper grading of the aggregates is key to good workability, low water demand, and efficient filling of the volume. Efficient filling of the volume by aggregates (with the consequent minimization of paste volume) is important for maximizing stiffness (modulus of elasticity), minimizing creep and shrinkage, and minimizing the generation of heat of hydration.

Particle size grading, together with the particle shape, determines the packing characteristics of the aggregate, that is, its ability to fill space. Any space not filled with aggregate must be filled with cement paste, which is typically more expensive than aggregate and is prone to shrinkage and thermal cracking. The most efficient packing is obtained with rounded, equidimensional particles of uniform grading. Sieve analyses for both coarse and fine aggregates should be performed according to AASHTO T 27 (ASTM C 136). The aggregate should meet the requirements of AASHTO M 6 and M 80 (ASTM C 33) except as noted.

Figure C5.2.1-1 provides a guide for the grading of the combined coarse and fine aggregates. Zone I mixtures tend to segregate during placement. Zone II represents the desirable range with the position of the dot and the surrounding region bounded by the parallelogram being the optimum. Zone III is an extension of Zone II of aggregates 0.5 in. (13 mm) and finer. Zone IV concrete would have too much fine mortar. Zone V concrete would be too rocky[25].

Clays and silts can increase water demand in fresh concrete, increase drying shrinkage, impair paste-to-aggregate bond, and cause disruptive swelling within the hardened concrete. Organic impurities can adversely affect setting times and strengths.

The grading of the fine aggregate fraction is also important for an additional reason: Too little fines make the concrete difficult to extrude and finish as well as more prone to bleeding, too much increases the water demand of the concrete and the required dosage of air-entraining admixture. For slip form paving, the minimum limit for the fineness modulus (calculated according to AASHTO T 27 (ASTM C 136) is 2.3. The fine aggregate should have no more than 45% passing any one sieve and retained on the next consecutive sieve.

[24] Kosmatka, Steven H.; Kerkhoff, Beatrix; and Panarese, William C., *Design and Control of Concrete Mixtures,* EB001, Portland Cement Association, 2002, 372 pages.

[25] Shilstone, James M. Sr. and Shilstone, James M. Jr., "Performance-Based Concrete Mixtures and Specifications for Today," *Concrete International,* Vol. 24, No. 2, February 2002, pp. 80-83.

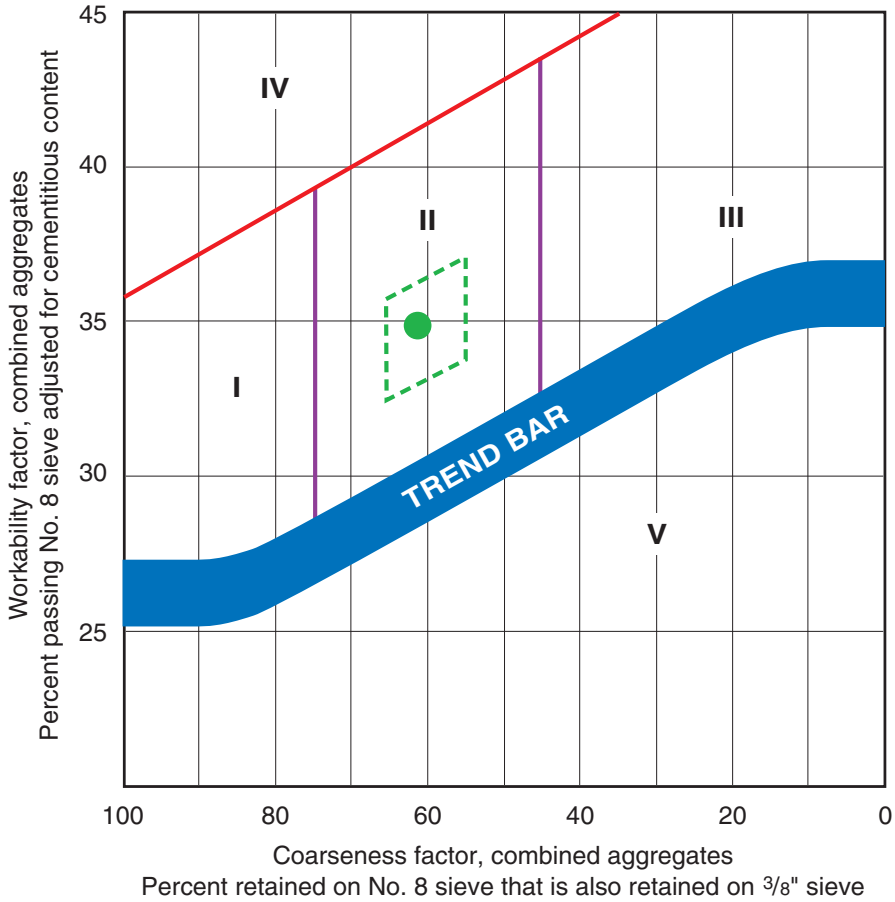


Figure C5.2.1-1. A satisfactory aggregate grading falls within Zone II on this plot.[22]

C5.2.2 Durability

C5.2.2.1 Alkali-Silica Reactivity

These guidelines are adapted from PCA IS415.

The tests for aggregate reactivity may be performed in any order. In general, ASTM C 1260 is considered conservative in that it may identify seemingly innocuous aggregates as reactive. ASTM C 1293 is considered more definitive but takes a full year to identify aggregates as reactive and two years to verify the effectiveness of mitigation measures. ASTM C 441 is not recommended because it does not represent the actual aggregate to be used and because the underlying assumption that low-alkali cement produces an acceptable result with all aggregates is not valid.

When requesting an evaluation of the aggregate by ASTM C 295, it is helpful to provide the petrographer with the list of reactive constituents and their allowable limits.

In developing a quarry, aggregate producers may have obtained ASTM C 1293 data for their aggregate with combinations of local cementitious materials. Such data would be acceptable in specifying concrete for a particular job. Some slowly reactive aggregates produce a "borderline" expansion value according to ASTM C 1293. In this situation it is helpful to plot the expansion versus time. If the slope of this line at the end of the test period indicates that the expansion is not leveling off, it is prudent to specify some mitigation measure. In the absence of these data or sufficient time to develop them, the prudent course would be to rely on ASTM C 1260 to identify the aggregate as reactive and ASTM C 1567 to select an appropriate mitigation measure. A range of combinations of cementitious materials could be tested simultaneously and all acceptable combinations included in the job specification.

Mitigation measures include the use of pozzolans and/or slag cement, either as a component of blended cement or as a separate addition at the concrete production facility. In some cases, the quantity of supplementary cementitious material in blended hydraulic cement is not sufficient to control expansions due to alkali-silica reaction. Additional supplementary cementitious material, either of the same or different kind, may be added to the blended hydraulic cement if necessary. As a guideline, Class F fly ash may require 15% to 25% by mass of total cementitious materials to meet the expansion criterion, while slag cement may require 40% to 50% and calcined clay approximately 15% to 20%. Class C fly ash is not generally recommended for this purpose, as it may actually increase expansions at some dosages. Ternary combinations (using two supplementary cementitious materials) also can be very effective.

It is essential to avoid making substitutions of one material for another on the job without testing. Fly ashes meeting the requirements of AASHTO M 295 (ASTM C 618) for Class F fly ash may be considerably different with regard to their effectiveness in controlling expansions due to alkali-silica reaction because of their contents of lime and/or alkalis, or because of their reactivity (a function of particle size and composition). Likewise, cements meeting the requirements of AASHTO M 85 (ASTM C 150) for Type I cement may require different dosages of the same fly ash to produce acceptable results with the same aggregate. If supplies are uncertain and it is anticipated that substitutions may have to be made during the course of the job, combinations of various possible job materials can be tested simultaneously to determine which ones produce acceptable results. Then all acceptable combinations may be listed in the specification and the final selection left to the Contractor or concrete producer.

In scattered areas of the Northern Great Plains, the glacial sands containing shale particles can be susceptible to popouts caused by alkali-silica reaction. Found most on hard-troweled surfaces, these popouts are much smaller and shallower than those caused by absorptive coarse aggregates. The popouts are very unusual in that they often appear a few hours after the concrete is finished and, in most cases, within the first few weeks.

The following procedures offered the best protection against the formation of popouts when reactive sands are used[26].

1. In the hot summer months, wet curing is essential. Wet curing should be initiated as early as possible.
2. Fresh concrete should be protected from drying before final finishing.
3. Hard-troweling should be avoided, if possible.

C5.2.2.2 Alkali-Carbonate Reactivity

Alkali-carbonate reactive aggregates are problematic only in limited geographical areas. The most conservative practice is to avoid the use of reactive aggregates by selective quarrying. Reactive aggregates may be identified by testing in accordance with ASTM C 1105.

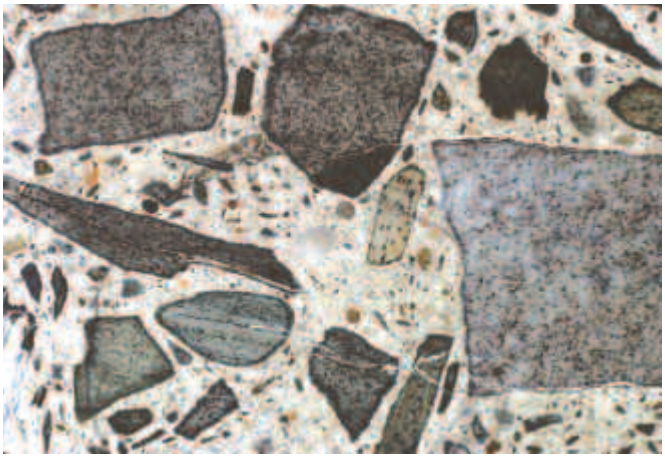


Figure C5.2.2.2-1 Micrograph showing cracks due to alkali carbonate reaction (ACR) caused by argillaceous (clay-rich) dolomitic limestone aggregates. (IMG16952)

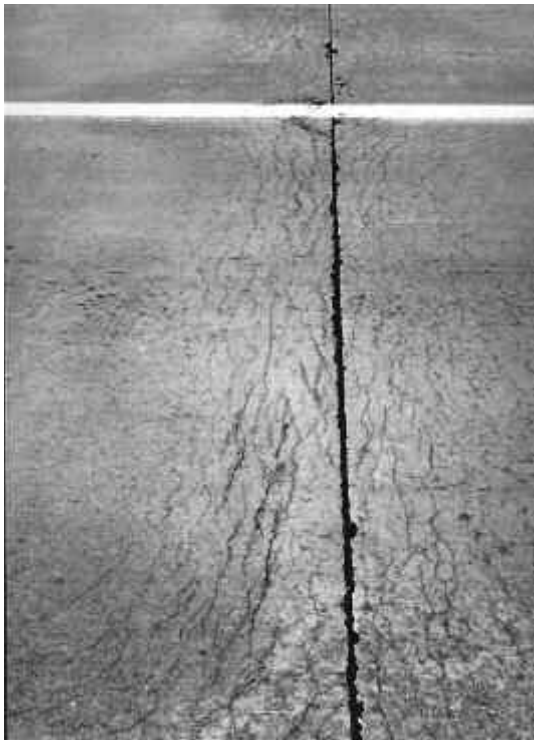
C5.2.2.3 D-Cracking

D-cracking may be a concern for concrete subjected to freezing and thawing under conditions of saturation. It may be of relevance for some bridge decks, but not for other bridge elements. Generally, the larger the particle size, the more susceptible the aggregate is to D-cracking. Aggregate susceptible to D-cracking must be either

[26] Landgren, R., and Hadley, D. W., *Surface Popouts Caused by Alkali-Aggregate Reaction,* RD121, Portland Cement Association, 2002.

rejected or beneficiated so that the particles of susceptible size are removed. The resulting reduction in the maximum size of the aggregate requires a concomitant change in the concrete mix design so that the proportion of aggregate is reduced and the proportion of cementitious material is increased. This may increase the cost of the concrete. More important, the increased paste content results in greater vulnerability to cracking due to increased thermal stresses and autogenous, plastic, and drying shrinkage. If there is an aggregate available that is not susceptible to D-cracking, it is prudent to use it.

Nearly all rock types susceptible to D-cracking are of sedimentary origin. If the performance history of a proposed aggregate is unknown and the concrete will be subjected to freezing, the aggregate must be tested. The Washington Hydraulic Fracture test[27] is the most direct method. It requires a special apparatus in which surface-sealed aggregate is placed in water. The vessel is subjected to 10 cycles of pressurization. The aggregate particles are counted to determine an index of susceptibility to D-cracking. AASHTO T 161 (ASTM C 666) tests the durability of concrete under cycles of freezing and thawing in conditions likely to saturate the concrete. Modifications for the purpose of testing aggregate for susceptibility to D-cracking include increasing the number of cycles to 350 and calculating the durability index from the expansion of the specimens. In the Iowa Pore Index Test[28,29] the aggregate is sealed into the pot of an AASHTO T 152 (ASTM C 231) air meter. Water is added to a certain level in the transparent tube at the top of the pot. Air pressure is applied to force the water into the pores of the aggregate. The decrease in the volume is called the pore index. A high pore index indicates a nondurable aggregate.



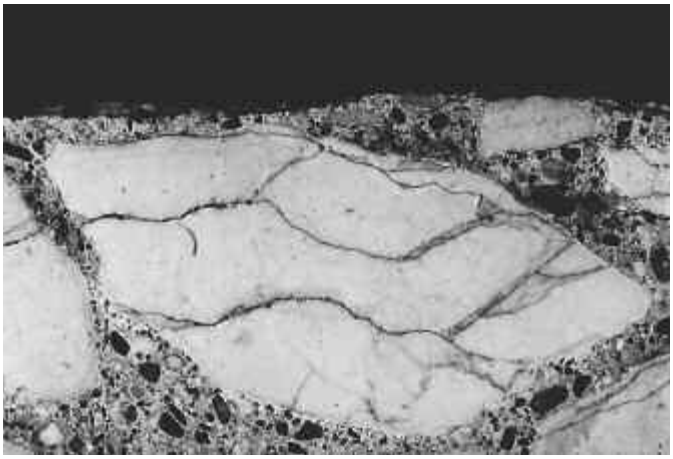


Figure C5.2.2.3-1. D-cracking along a transverse joint caused by failure of carbonate coarse aggregate. (IMG12314, IMG12315)

[27] Janssen, Donald J. and Snyder, Mark B., *Resistance of Concrete to Freezing and Thawing,* SHRP-C-391, Washington, DC: Strategic Highway Research Program, National Research Council, 1994, 301 pp.

[28] Traylor, M.L., "Efforts to Eliminate D-Cracking in Illinois," *Transportation Research Record,* No. 853, 1982, pp. 9-14.

[29] Marks, V.J. and Dubberke, W., "Durability of Concrete and the Iowa Pore Index Test," *Transportation Research Record,* No. 853, 1982, pp. 25-31.

C5.3 Water

Refer to PCA EB001, *Design and Control of Concrete Mixtures,* for a complete discussion of water for use in concrete. In general, water suitable for drinking and with no perceptible taste or odor is suitable for making concrete. Some non-potable waters including recycled wash waters, are also acceptable for use in concrete but must be tested to verify that no harmful effects will result. AASHTO M 157 (ASTM C 1602) also provides guidance on the acceptability and testing of water for use in concrete.

Figure C5.3-1. Water that is safe to drink is safe to use in concrete. (IMG12312)

C5.4 Chemical Admixtures

It is advisable to purchase all of the chemical admixtures to be used in the concrete from a single manufacturer. Certain chemical admixtures are incompatible with one another, or with certain cementitious materials. Generally, manufacturers test their own admixtures in combinations with one another using the available cementitious materials and can advise the user of potential interactions.

Early stiffening may result when water-reducing admixtures containing lignosulfonate and/or triethanolamine (TEA) are used in combination with some cements and Class C fly ashes, particularly in hot weather. Trial batches should be conducted at working temperatures to assess the likelihood of incompatibility. Tests conducted on the trial batches should include monitoring slump loss, time of set, and the temperature of the mixture with time.

ACI 201.2 reports that if chloride ions in an admixture are less than 0.01% by mass of cementitious material, such contribution represents an insignificant amount and may be considered innocuous.

Figure C5.3-2. Microwave Water Content Test (AASHTO T 318). Freshly mixed concrete wrapped in a fiberglass cloth in a heat-resistant glass tray (Pyrex) is dried in a microwave oven using a minimum of three drying intervals. The water content of the test specimen is calculated based on loss in mass of the test specimen. The w/c ratio can then be determined by dividing the amount of water from the microwave tests by the amount of cement indicated on the batch ticket. (left: IMG17613, right: IMG17614).

Figure C5.4-1. Liquid admixtures, from left to right: antiwashout admixture, shrinkage reducer, water reducer, foaming agent, corrosion inhibitor, and air-entraining admixture. (IMG12188)

The surface tension of the water in partially filled pores in concrete pulls inward on the walls of the pores, resulting in shrinkage of the concrete. Shrinkage-reducing admixtures reduce the surface tension of the pore water, reducing both the shrinkage and the susceptibility to cracking under restrained conditions. Shrinkage-reducing admixtures are used conventionally in applications where a notable reduction in drying shrinkage is desired and also may benefit concrete mixtures susceptible to autogenous shrinkage[30]. Shrinkage-reducing admixtures may affect strength, resistance to chloride ion ingress, freeze/thaw durability, modulus of elasticity, creep, and long-term shrinkage. They should never be used in lieu of proper curing.

Hydration-stabilizing admixtures may be useful in situations where a controlled extension of set time is desired, such as extended hauls and during large continuous placements. Unlike conventional set retarding admixtures, hydration-stabilizing admixtures are formulated to provide extended set time control. Depending on the dosage used, set time extensions can range from a few hours to over a day.

Corrosion inhibiting admixtures may be added to concrete to reduce the risk of corrosion of steel embedded in concrete. These products must be used in conjunction with and not in lieu of good concrete materials and practice.

C6.0 Submission and Design Requirements

C6.1 Concrete Mixture Proportioning

In times of high demand for concrete-making materials, it is recommended that alternate mix designs using alternate materials be submitted for approval simultaneously. If a material then becomes unavailable during the course of a job, the concrete producer may substitute another material and the appropriate mix design without delay. The concrete producer must inform the purchaser when substitutions are to be made, even when the materials and mix designs already have been approved.

For mixture proportioning, refer to "Design and Control[31]," or ACI 211.1 for normal density concrete, ACI 211.2 for lightweight concrete, and ACI 211.4 for high-strength concrete containing fly ash. Recommendations for proportioning high-strength concrete also are covered in Chapter 3 of ACI 363R.

Figure C6.1-1. Testing concrete mixes in the lab is often more convenient and economical than having to batch large quantities at a concrete plant. It is important to recognize that project conditions are vastly different than the controlled environment of a laboratory. Production variability and testing variability need to be considered and understood when lab tests results are interpreted. (IMG16954)

[30] Tazawa, Ei-ichi, Ed., "Autogenous Shrinkage of Concrete," *Proceedings of the International Workshop Organized by the Japan Concrete Institute,* E&FN Spon, 1999, p. 14.

[31] Kosmatka, Steven H.; Kerkhoff, Beatrix; and Panarese, William C., *Design and Control of Concrete Mixtures,* EB001, Portland Cement Association, 2002, 372 pages.

Note that the end product of the ACI mix proportioning methods is not a prescriptive "recipe" for a concrete mix design, but a starting point for laboratory trial batches. Such characteristics as the packing efficiency and water demand of the aggregates will affect the actual quantity of cement paste required for the desired workability. The dosages of water reducing and air-entraining admixtures also must be determined by trial batches using the manufacturer's recommended dosages as the starting point. In addition, all properties of the fresh and hardened concrete determined by the Engineer to be important to the project must be tested to ensure that the mix design meets the project requirements.

C6.2 Concrete Production Facility Certification

The NRMCA truck and plant certification programs follow a checklist inspection process for components and systems in concrete plants and verify that they conform to the requirements of AASHTO M 157 (ASTM C 94) and pertinent standards of concrete production equipment.

C6.3 Concrete Materials

When producing high-performance concrete, it is advisable for the concrete producer to retain traceable grab samples of the constituent concrete-making materials from each day's production. The retained samples allow for later investigation of any problems with the concrete. A 5-gallon (20 liter) pail of each aggregate and cementitious material, representative of each day's production with that ingredient, is sufficient for this purpose. Samples should be retained for six months.

C6.4 Temperature Control Methods

Recommended practices for controlling the placement temperature and in-place temperature of concrete placed during hot weather and cold weather are detailed in ACI 305R and 306R, respectively, and in PCI MNL-116 and TM-103.

C6.5 Crack Control Methods

Cracks may be caused by any combination of stresses arising from restraint of autogenous, plastic, or drying shrinkage; thermal gradients; imposed loads; and stress concentrations such that the tensile stress exceeds the tensile strength of the concrete.

The Engineer and Contractor should work together to select the combination of crack control measures that best meets the requirements of the project at a reasonable cost.

Crack control measures related to the selection of constituent ingredients and proportioning of the concrete mixture include:

1. Selection of a suitable cement.
2. Replacement of some of the cement with a low-calcium (Class F) fly ash, or use a blended cement containing Class F fly ash.
3. Minimization of the water content. This may be done by employing a water reducer and/or by selecting a favorable aggregate size and grading. The use of a compatible fly ash also may reduce water demand.
4. Selection of an aggregate with a low coefficient of thermal expansion and a rough surface texture. Maximize the aggregate content by specifying the largest possible maximum size and a favorable particle size grading.
5. Use of a shrinkage-reducing admixture, possibly in combination with carbon or steel fibers.

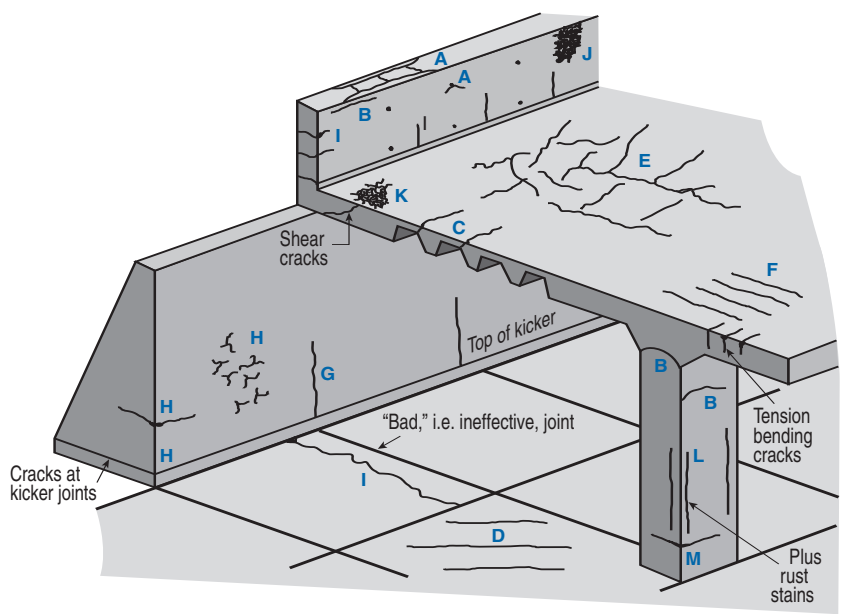


Figure C6.5-1. An illustration of nonstructural cracks that may occur in a hypothetical concrete structure (Concrete Society, 1982).
A to C—plastic settlement, D to F—plastic shrinkage, G to H—thermal effects, I—drying shrinkage, J to K—crazing, L to M—reinforcement corrosion, N—alkali silica reaction.

6. Reducing the modulus of elasticity (stiffness) by the entrainment of 4% to 6% air in the concrete even if it is not necessary for frost resistance.

7. Use of an aggregate with an absorption of less than 1%, or ensuring that the aggregate moisture condition is always at or above saturated surface dry.

Crack control measures related to workmanship include:

1. Control of the placement temperature. The concrete should not be much cooler than the ambient temperature, however. In winter, it may be advantageous for the concrete to be somewhat warmer than the ambient temperature. Depending on the importance of controlling cracking, a detailed analysis of thermal stresses may be necessary.

2. Use of established procedures for hot-weather and cold-weather concreting to control concrete quality.

3. Appropriate construction management to have sufficient personnel at the site to place, consolidate, and finish the concrete promptly.

4. Provision of fog sprays and windbreaks as necessary to prevent the surface of the concrete from drying.

5. Curing the concrete as soon as possible.

6. Control of the temperature after placement. Allowing controlled evaporation of water from absorptive blankets on the surface is an effective means of both cooling and moist curing.

7. Adherence to established procedures for good concreting practice. Consult the documents of American Concrete Institute Committees 201, 207, 224, 232, 233, 234, 304, 305, 306, 308, 309, 345, and/or 363, and PCI MNL–116 as appropriate.

Crack control measures related to structural design and detailing include:

1. Minimization of strains likely to occur in structural elements. For example, to minimize cracking in a bridge deck it may be necessary to limit the deflection of the supporting girders.

2. Limiting the maximum dimensions of any structural element by providing construction joints.

3. Not specifying a higher strength than necessary, or if possible, specifying a 56- or 90-day strength rather than an earlier age strength.

4. Minimization of the restraint to which the concrete is subjected.

5. Use of reinforcing steel to develop a greater number of small cracks rather than a few large cracks.

6. Use of a large number of small-diameter reinforcing bars at close spacing rather than a few large-diameter bars. (Note that the bar spacing must permit adequate consolidation of the concrete.)

In evaluating the literature on the relationship between corrosion of uncoated reinforcement and cracks perpendicular to it, Oesterle[32] concluded that for crack widths less than 0.016 in. (0.4 mm), crack width was of minor importance. The quality of the concrete and the depth of cover over the reinforcement are the primary factors determining the service life of cracked concrete. He recommended limiting crack widths to a maximum of 0.016 in. (0.4 mm) for corrosion protection.

Mailvaganam et. al.[33] recommended the following limits:

- Maximum of 0.004 in. (0.1 mm) for the most severe exposure (industrial or marine environment where watertightness is essential)
- Maximum of 0.008 in. (0.2 mm) for normal exterior exposures or interior exposures of structural members in humid or aggressive atmosphere
- Maximum of 0.012 in. (0.3 mm) for internal and protected members

[32] Oesterle, R.G., *The Role of Concrete Cover in Crack Control Criteria and Corrosion Protection,* Portland Cement Association R.& D. Serial No. 2054, 1997.

[33] Mailvaganam, Noel P.; Grattan-Bellew, P.E.; and Perinca, Gerry, "Deterioration of Concrete: Symptoms, Causes, and Investigation," *National Research Council,* Canada, Institute for Research in Construction, Ottawa, Canada, 2000.

C6.6 Curing

Curing methods, materials and monitoring procedures are detailed in ACI 308R and in FHWA FP-03 for cast-in-place concrete and PCI MNL-116 for precast concrete. Where it is practical, moist curing is preferable to the application of a curing compound. For precast concrete, use curing described in PCI MNL-116.

The objective of curing is to maintain moisture and temperature conditions for sufficient time to allow for the hydration of cementitious materials and pozzolans. Good curing is essential for the concrete to develop the desired durability and strength, and to minimize cracking. Depending on the properties desired, HPC may have lower water-cementitious materials ratio, higher cementitious

Figure C6.6-1. Curing methods that maintain the presence of mixing water in the concrete during the early hardening period include ponding or immersion, spraying or fogging (top left photo), and saturated wet coverings (top right photo). Methods that reduce the loss of mixing water from the surface of the concrete include covering the concrete with impervious paper or plastic sheets, or by applying membrane-forming curing compounds (bottom photos). Curing often involves a series of different procedures used at a particular time as the concrete ages. For example, fog spraying or plastic covered wet burlap can precede application of a curing compound. (IMG16978, IMG16979, IMG16981, IMG16980)

materials content, and/or reduced bleeding as compared with conventional concretes. The higher the cement content, the greater attention should be given to curing methods to control the development of internal temperature differentials which could lead to cracking. In the case of aggressively evaporative environments, i.e. low humidity and/or windy conditions, use of fogging, sunshades, windscreens, or enclosures may be necessary to prevent excessive surface drying. Curing must be initiated before the concrete starts to dry.

If water curing is employed, it should be done on a continuous basis throughout the specified curing period. Intermittent water curing that allows concrete to undergo cycles of wetting and drying can be more detrimental than no curing at all.

If steam curing is employed, care should be taken to prevent the concrete temperature exceeding the temperature above which the risk of delayed ettringite formation may occur in mixtures containing materials prone to the problem. In general, 158°F (70°C) is accepted as a reasonable upper limit. Higher temperatures may be acceptable for certain materials if proven by field performance or test.

Both maturity and temperature-matched curing are excellent methods of assessing the development of strength or other properties of the concrete at early ages. Decisions such as when to re–move forms, release prestressing, or post-tension can be made more reliably with information from either of these methods. Both methods require the use of thermocouples to measure the temperature history of the concrete.

The maturity method predicts the in-situ strength (or other properties) of the concrete at any time based on data developed from the trial batches. Thus it requires an investment in testing the development of the desired properties using trial batches subjected to a range of curing conditions representing those anticipated on site. ASTM C 1074 generates a maturity index based on the temperature history.

The maturity method normally is used to assess the current strength of the concrete; however, it also can be used to model different hypothetical situations, such as the length of time formwork should remain in place or whether it would be effective to use insulated forms or heated enclosures during cool weather to accelerate the construction schedule.

Temperature-matched curing uses data from the thermocouples to control the curing temperature of companion cylinders. The compressive strength (or other property) of the cylinders is determined directly and closely matches that of the concrete in the structure at the same time. Temperature-matched curing is thus simpler to implement than the maturity method and provides more direct information about the current properties of the in-situ concrete, but does not predict properties.

C7.0 Quality Management

The guide specification has been based on the presumption that a given concrete mixture will be pre-qualified by specification and testing appropriate for the given element and environment. Quality assurance and control are then based on confirming that every load of concrete used at the site is comparable to the pre-qualified mixture. This means that it is not necessarily required to test every property of every load, but to confirm that the materials and mix proportions are the same as those in the pre-qualified mix, and that selected indicators of variability (such as air content and consistency) remain within suitable bounds.

C7.1 Quality Assurance

Refer to ACI 121R, PCI MNL-116, AASHTO Guide to Quality Control/Quality Assurance, and NRMCA Publication 190 for more detail on quality assurance systems.

C7.2 Quality Control

Recommended practices related to quality control and testing of high-strength concrete are detailed in ACI 363.2, "Guide to Quality Control and Testing of High-Strength Concrete," PCI MNL-116, NRMCA Quality Control Manual, and NRMCA Publication 190.

In principle, sufficient testing is required to ensure that the requirements of the specification are being complied with, and that a uniform product is being produced. The tests should be related to the parameters deemed important in the specification. Some tests may be conducted at pre-qualification stage to assure that the mix design provides an adequate concrete, while production acceptance testing may be done to demonstrate that the concrete being delivered from batch to batch is equivalent to the qualified mixture.

C8.0 Production of Concrete

C8.1 General

C8.2 Equipment

In situations where discharge of the concrete may not be completed within the allowable time limitations, such as extended hauls, hydration-stabilizing admixtures may be beneficial. Hydration-stabilizing admixtures are discussed in Section C5.4.

C8.3 Measurement of Materials

Consult the admixture manufacturer's literature for guidance as to the order in which admixtures should be added to the concrete, as this will affect their performance. In general, admixtures should be introduced separately to the batch.

Materials quantities in freshly mixed or hardened concrete can be approximated using a number of techniques. The water content of fresh concrete can be determined using the Microwave test (AASHTO T 318,) except that allowance has to be made for the moisture state of the aggregate at the time of mixing. Cement content may be estimated using ASTM C 1084 for systems that do not contain supplementary cementitious materials. Monitoring unit weight will provide a means of flagging changes in mixture proportions.

C8.4 Mixing

It is essential to ensure thorough mixing, both for uniform distribution of the concrete ingredients throughout the batch and to entrain an adequate air-void system. However, overmixing may remove entrained air from the concrete. In addition, with some synthetic air-entraining admixtures, retempering and extended mixing can result in excessive air and/or clustering of air voids. Trial batches and mockups should be used to verify that the ingredients and procedures used would result in satisfactory air-void systems in the concrete as placed.

If silica fume is used, particular attention must be paid to the batching sequence and mixing procedure to ensure uniform mixing. The use of a blended cement containing silica fume guarantees uniform mixing. The use of pulpable bags of silica fume is not advisable, as it may be difficult to achieve adequate mixing. The specifier may wish to consult the silica fume supplier for recommendations.

C8.5 Temperature Control

Temperature of concrete is important for the development of properties related to strength and durability. So long as the concrete is protected from freezing, low temperatures result in the development of favorable properties, but at a significantly slower rate. Early-age freezing of concrete may disrupt the paste microstructure and permanently damage the concrete. Elevated temperatures result in accelerated setting and high early strengths but reduced ultimate strengths, as well as higher permeability and greater potential for delayed ettringite formation. The exact temperature at which the concrete becomes vulnerable to delayed ettringite formation varies with the cementitious material(s) employed.

In general, 158°F (70°C) is accepted as a reasonable upper limit. Higher temperatures may be acceptable for certain materials if proven by field performance or test. The maximum temperature of 158°F (70°C) is specified for two reasons:

1. Delayed ettringite formation is possible under some circumstances when the temperature exceeds this value.
2. The higher the curing temperature, the more permeable the concrete.

Appropriate use of supplementary cementitious materials will reduce or eliminate the possibility of delayed ettringite formation[34].

Some concrete ingredients and mix proportions are better suited than others for extreme weather conditions. In hot weather, supplementary cementitious materials such as Class F fly ash and slag cement, which generate less heat of hydration than cement, help keep the heat development of the concrete in the appropriate range, thus minimizing the likelihood of thermal cracking. They also can make the concrete less susceptible to premature stiffening.

[34] Miller, F.M., and Conway, T., "Use of Ground Granulated Blast Furnace Slag for Reduction of Expansion Due to Delayed Ettringite Formation," *Cement, Concrete and Aggregates,* Vol. 25, Issue. 2, 2003, pp. 59-68.

C8.5.1 Cold Weather

The primary concerns relating to cold weather are slow setting and slow strength gain, permanent damage to the concrete due to early freezing, and excessive thermal gradients that may lead to cracking.

When concrete is to be placed in cold weather, or at a time of year when cold weather is likely, plans to maintain the concrete at the appropriate temperature should be made well before the temperature is expected to drop below freezing.

Concrete mix designs developed for placement at cooler temperatures normally have somewhat higher cement contents than those for hot weather. The use of slag cement and fly ash may need to be reduced or eliminated unless they are required to control expansions due to alkali-silica reaction or to increase the resistance to sulfate attack or to reduce permeability. In that case, the total cementitious materials content may need to be increased, or Type III or Type HE cement may be used instead of Type I/II. The required dosage of air-entraining admixture will be lower than at normal temperatures. The longer setting time of concrete in cold weather increases the window of vulnerability to plastic shrinkage cracking. If the concrete is much warmer than the ambient air or the wind is blowing, the local reduction in relative humidity also can contribute to plastic shrinkage cracking. Concrete surfaces must be protected from drying with windbreaks, application of curing compound, etc. An accelerating admixture conforming to AASHTO M 194 (ASTM C 494) Type C or E may be used provided its performance previously has been verified by trial batch.

Use of admixtures containing chlorides is not recommended and in prestressed or post-tensioned concrete is strictly prohibited.

Ideally, concrete should not be placed when the temperatures of the air at the site or the surfaces on which the concrete is to be placed are less than 40°F (5°C). If circumstances require that concrete be placed at these temperatures, special provisions are required as detailed in ACI 306R and PCI MNL-116. Covering and/or other means of protecting the concrete should be available on site before starting placement. The concrete temperature should be maintained at 50°F (10°C) or above for at least 72 hours after placement and at a temperature above freezing for the remainder of the curing period. If the concrete is to be heated, it should be by a method that does not expose the concrete to CO_2 gas. Note that corners and edges are the most vulnerable to freezing. Concrete damaged by pre-mature freezing must be completely removed and replaced.

Figure C8.5.1-1. Concrete being covered with a tarpaulin to retain heat of hydration. (IMG15215)

Note that in colder temperatures, concrete gains strength more slowly. This effect is more pronounced for concrete containing supplementary cementitious materials. Before removing formwork or post-tensioning structural elements, the adequacy of the in-place compressive strength of the concrete must be verified by the maturity method, temperature-matched curing, nondestructive testing, or tests of cores.

Guidance and further details on cold weather concreting practices are given in ACI 306R and PCI MNL-116.

C8.5.2 Hot Weather

The primary concerns relating to hot-weather concreting include increased water demand, premature stiffening, loss of workability, increased rate of setting, loss of entrained air, plastic shrinkage cracking, decreased later-age strength, excessive hydration temperatures, and excessive thermal gradients leading to cracking. High performance concrete may experience little or no bleeding; thus it is particularly sensitive to plastic shrinkage cracking.

Figure C8.5.2-1. Liquid nitrogen added directly into a truck mixer to reduce the concrete temperature. (IMG12357)

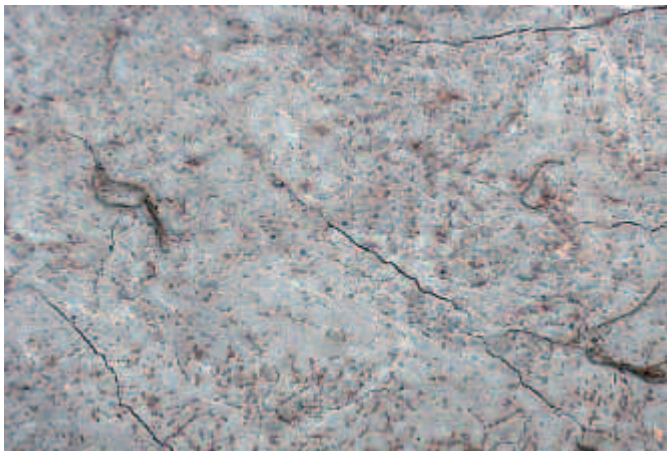
Calcium sulfate in the form of gypsum and anhydrite is added to cement to control the hydration of aluminates, preventing early stiffening. Elevated temperatures accelerate the dissolution of the aluminates and retard the dissolution of the sulfates. Class C fly ashes with high alumina contents can be problematic in hot weather if they contribute more aluminates than soluble sulfates to the concrete. The use of some water-reducing admixtures also can contribute to early stiffening. This effect is more pronounced in hot weather because of the increased water demand of the concrete (thus the tendency to use higher dosages of water-reducing admixture).

The concrete mix design used for hot weather should have been previously verified as appropriate using trial batches mixed and cast at temperatures representative of typical hot weather conditions at the site. The use of slag cement, Class F fly ash, and/or natural pozzolans in substitution for part of the cement is recommended. All of these materials hydrate more slowly and generate lower heats of hydration than cement, thus reducing problems with slump loss, premature stiffening, and thermal cracking. Class C fly ashes with high contents of Al_2O_3 may contribute to premature stiffening.

Reductions in air contents due to hot weather can be corrected by increasing the dosage of air-entraining admixture and/or by retempering with water-reducing admixture or water to restore the slump. Do not exceed the maximum allowable water-cementitious materials ratio or manufacturer's maximum recommended dosage for any of the admixtures.

Retarding admixtures may be used if their performance previously has been verified by trial batches.

Thermal cracking may be prevented by ensuring that the temperature of concrete at the time of placement is as low as practical, and in no case should it exceed 90°F (32°C) except in precast concrete plants that have demonstrated successful use of a maximum temperature of 95°F (35°C). When possible, store aggregates out of direct sunlight. Aggregates also may be cooled and



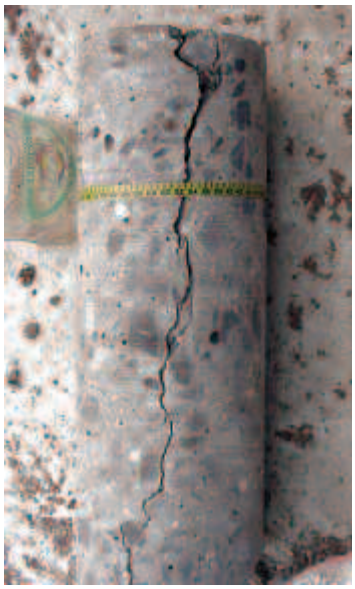


Figure C8.5.2-2. Mass concrete footing where the insulation was removed much too soon and a large thermal gradient developed through the concrete by thermally shocking the surface. The footing was approximately 5 ft (1.5 m) thick, and the insulation was removed after ~2 days. The concrete cracked shortly thereafter. (left: IMG17801, right: IMG17802)

moistened by sprinkling with water. If possible, avoid the use of hot cement or fly ash. Mixing water may be chilled, or chipped ice (batched by mass) may be used in substitution for some of the water. Be sure that all of the ice melts during mixing. Mixing and transporting equipment may be painted white or a light color to minimize the heat absorbed from the sun. Depending on the heat characteristics of the concrete, placements may be scheduled for late afternoon or nighttime to reduce thermal gradients. Delays in placement should be avoided. The use of a white curing compound will help reflect the sun's heat.

Plastic shrinkage cracking results from loss of moisture from concrete before it has set. Aggregates should be batched as close to a saturated condition as possible to avoid absorbing mix water. The concrete should be protected from loss of moisture during mixing and placement. Protection measures may include fog spraying and shelter from wind. Absorbent forms should be dampened before placement. The concrete should be placed and finished as rapidly as possible and curing compound (if used) applied as soon as possible. If there is any delay in applying the curing compound, use a fog spray to keep the surface from drying out. When the rate of evaporation is predicted from Figure C8.5.2-3 to be above 0.1 lb/ft^2/hr (0.5 kg/m^2/hr), provide wind screens and fog spraying as appropriate or stop placing concrete. Note that high performance concrete is particularly vulnerable to plastic shrinkage cracking because it has little or no bleeding. Take extra precautions to prevent evaporation when placing silica fume concrete in hot weather. If plastic shrinkage cracking is observed, the Contractor should provide wind screens and more fog spraying as needed. If these measures are not effective, operations should stop until weather conditions improve.[35]

Guidance and further details on hot-weather concreting practices are given in ACI 305R and PCI MNL-116.

C8.5.3 Control of Temperatures

Traditionally, mass concrete members were considered to be those with dimensions of 3 ft. (1 m) or more. However, high performance concrete may be more susceptible to cracking due to its higher cementitious materials content and/or increased modulus of elasticity. Thus special precautions may be required even for thinner HPC members to minimize cracking.

The primary objective with mass concrete is to control the temperature gradient between the internal temperature and the surface. This can be accomplished by the following measures:

1. Minimizing the heat of hydration by appropriate selection of cementitious materials and limitation of the cement content. Select the largest practicable maximum size of aggregate and use supplementary cementitious materials known to reduce the heat.
2. Minimizing the placement temperature of the concrete, for example by cooling the individual ingredients, using ice for part of the mixing water, and/or injecting liquid nitrogen into the mixer.
3. Cooling the concrete after placement by use of embedded cooling coils or pipes.
4. Managing the construction procedures and scheduling to protect the concrete from excessive temperature differentials. For example, concreting may take place at night to prevent solar radiation from heating the surface, or the surface may be insulated to minimize the temperature difference between the surface and the interior.

Consult the publications of ACI 207.1R, 207.2R, and 207.4R for further information and recommended practices.

[35] ACI 224R, *Control of Cracking in Concrete Structures.*

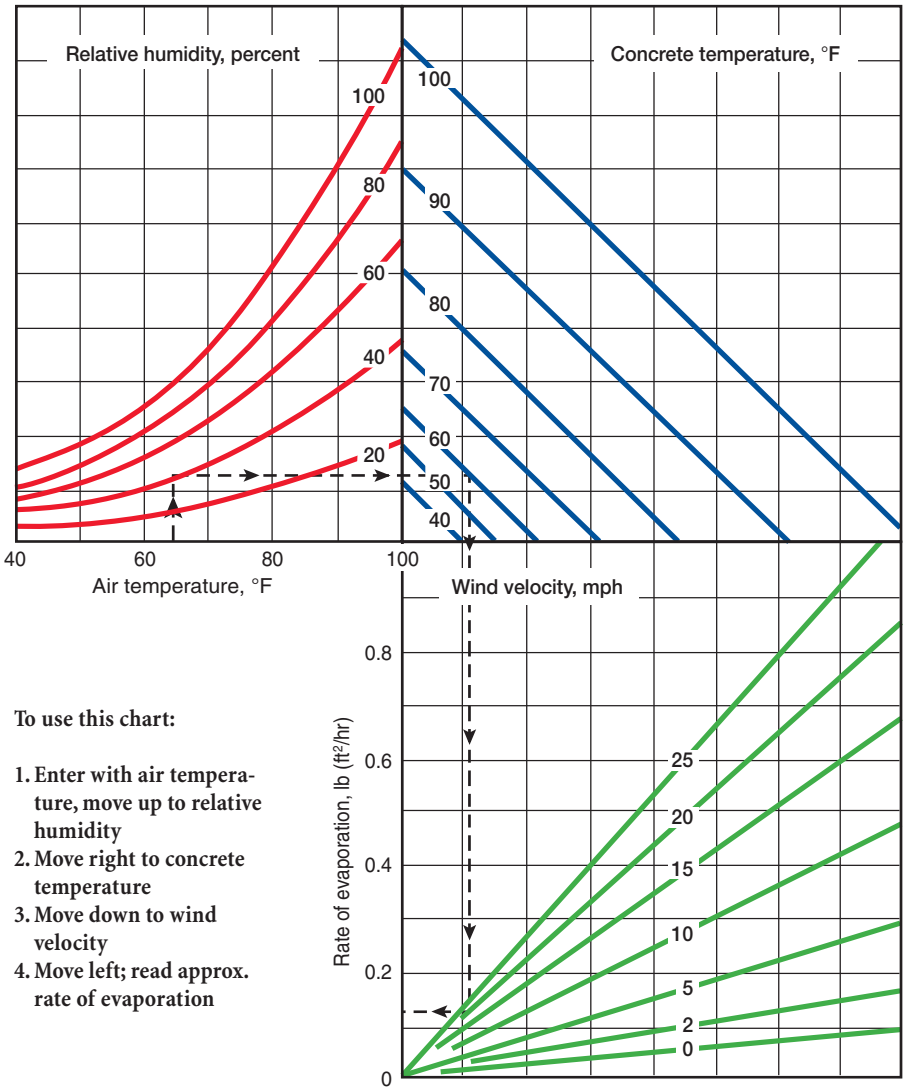


Figure C8.5.2-3. Effect of concrete and air temperatures, relative humidity, and wind velocity on rate of evaporation of surface moisture from concrete. Wind speed is the average horizontal air or wind speed in mph (km/h) measured at 20 in. (500 mm) above the evaporating surface. Air temperature and relative humidity should be measured at a level approximately 4 to 6 ft (1.2 to 1.8 m) above the evaporating surface and on the windward side shielded from the sun's rays (Menzel 1954)[36].

C8.6 Trial Batches and Mockups

Trial batches are essential to verify the performance characteristics of the concrete. Laboratory trial batches can be used to calibrate field quality control measurements, for example slump retention and the volume of air corresponding with a satisfactory air-void system in the hardened concrete. Trial batches or mockups can be used to correlate early-age or accelerated-cure strengths with the corresponding specified design strength.

If a maturity method is to be used to monitor the strength or other properties at early ages, the necessary data should be developed using trial batches.

If the job is scheduled for a time of year when hot or cold weather is anticipated, trial batches should be cured

[36] Menzel, Carl A., "Causes and Prevention of Crack Development in Plastic Concrete," *Proceedings of the Portland Cement Association,* 1954, pp. 130 to 136.

at the anticipated job temperatures in addition to a standard curing temperature of 23°C (73°F). The strength gain characteristics and effects on the development of the other specified properties could be determined.

Trial batches and mockups always should use concrete-making materials representative of those to be used through the course of the work. If material is to be supplied in bulk form, bagged materials should not be used in the trial batches or mockups.

Because of the time required to conduct trial batches and perform the required testing, it is advisable to conduct trial batches on a suite of concrete mix designs using the various materials under consideration for use. Then if substitution of a material is necessary during the course of the job, mix design already has been verified and the substitution can be made without delay.

Field trial batches and mockups verify that the batching, mixing, transport, placement, and finishing techniques to be used in the field with full-scale batches will produce satisfactory concrete. Pay special attention to any performance characteristics specified above Grade 1. For example, if resistance to freezing and thawing and/or deicer scaling is specified, the air-void system of the concrete as placed is critical. Mixing, pumping (if used), consolidation, and finishing procedures all affect the air-void system. Cores should be taken from the mockup and examined according to ASTM C 457 to ensure that the air-void spacing factor of the concrete near the top surface is acceptably low.

Permission may be granted to forgo some trial batch testing if the supplier has supplied a similar material within the last 12 months. A shorter period may be selected, depending on the local conditions of material availability and variability, construction practices, and contractor turnover.

C8.7 Site Addition of Materials

For high-performance concrete, the addition of water at the site should be avoided, particularly when high strengths or low chloride penetration values are specified. Late addition of water will compromise the microstructure and ultimate performance of the concrete, even such addition is to replace water apparently lost through evaporation or aggregate absorption. It is best to prohibit the addition of water at the site and permit the addition of a water reducing or high-range water-reducing admixture at the site to achieve the required slump.

Figure C8.7-1. Admixture being added to concrete on-site. (IMG17641)

C8.8 Delivery Tickets

Delivery tickets are an important form of quality monitoring, because when things go wrong the tickets are the first source of information on what went into the batch, and where in the structure the batch went. Such information can significantly reduce the amount of effort required in some forensic investigations.

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