Durability of Concrete: Second Edition

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SECOND EDITION

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for the
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Preface

This circular is an update to *Transportation Research Circular 494: Durability of Concrete*, published in 1999, originally authored by Bryant Mather, V. Ramakrishnan, Steven H. Kosmatka, D. Stephen Lane, H. Celik Ozyildirim, and David L. Rettner. This revised publication is intended to provide the latest information for consideration by practitioners on producing durable concrete for transportation structures and pavements. Considering the number of facilities that have required repairs and reconstruction before reaching their intended service life, as well as the cost of the rehabilitation and the inconveniences to the traveling public, the importance of constructing long-lasting bridges and pavements continues to capture national attention and remains a high-priority item.

This document is divided into sections introducing each topic and discussing the production of durable concrete through materials selection, proportioning, construction practices, specifications, and testing. Also included is a section of case studies, providing examples of problems encountered in the field that involve concrete pavement and bridges, along with the proposed solutions by the authors of the studies. Special appreciation is expressed to Peter Taylor for overall editing and to Paul Tennis for his work in updating the Introduction, Materials Selection, Proportioning, and Testing sections. Sincere thanks are expressed to Karthik Obla for his work in updating the Specifications portion and to Prashant Ram for his work in updating the Construction Practices section. The work Thomas Van Dam did on the Case Studies section and that Heather Dylla did in updating the References is greatly appreciated. In addition, many committee members and friends of the committee who have an interest in the subject and experience in the field made significant contributions.

The Durability of Concrete Committee welcomes suggestions from readers and practitioners for future updating of the information.

—Tyson D. Rupnow, *Chair*
Durability of Concrete Committee

PUBLISHER'S NOTE

The contents of this Circular reflect the views of the authors of the original edition published in 1999 and of the expert volunteers who undertook the updates for this edition. These contributors are responsible for the facts and the accuracy of the data. This document does not necessarily reflect the official views or policies of the Transportation Research Board, and the contents of this Circular do not constitute a standard, specification, or regulation.

Contents

Introduction

BRYANT MATHER *(deceased), U.S. Army Corps of Engineers*
Updated by Paul Tennis, Portland Cement Association

When used in transportation, the performance of concrete is generally regarded as satisfactory if it meets the contractual requirements for composition, slump, and strength and thereafter is found to be "durable." There is a misconception that concrete has a property named "durability." This is not the case, since concrete with a given set of properties will endure without noticeable change for centuries or even millennia in one environment and be reduced to fragments in a few years or even a few months in another. Durability includes a series of properties required for the particular environment to which concrete will be exposed during its service life. *Durable concrete* is that which resists the forces in that environment that tend to cause it to deteriorate prematurely without requiring excessive effort for maintenance. Many assume that requiring a certain level of strength, a minimum cementitious materials content, and a maximum water–cementitious material ratio (w/cm) will ensure durable concretes. This can be misleading; durable concrete must possess properties appropriate for the environment.

In order to maximize the probability that concrete in a given application will be durable, it is necessary to deal not only with the direct but also with the indirect factors that can influence the ability of concrete to successfully resist a deteriorative environment. To be resistant to the effects of freezing and thawing, even if critically saturated, the concrete must have a proper air–void system (specifically spacing factor less than 0.2 mm by ASTM C457), sound aggregate [durability factor of at least 60 by AASHTO T 161 (ASTM C666) Procedure A], and moderate maturity [compressive strength of at least 31 MPa (4,000 psi)]. To be resistant to the effects of alkali–silica reaction (ASR), the concrete must be made with aggregate that is not deleteriously reactive, must not contain too high an alkali content for the reactive aggregate used, or must contain an adequate amount of pozzolan, ground granulated blast-furnace slag (GGBFS or slag cement), or a lithium compound. To be resistant to the effects of sulfate attack, the concrete must have a low permeability (w/cm ratio) and use cement resistant to sulfate attack or must contain an adequate amount of an appropriate pozzolan or GGBFS. To be resistant to excessive damage by abrasion, the concrete must have high to moderate strength with high abrasion-resistant coarse aggregate. To avoid excessive carbonation and consequent danger of steel corrosion, the steel should have an adequate cover of concrete with low permeability. These are the primary concrete characteristics that have a direct effect on durability.

There is, however, a considerable amount of literature and experience that confirms the intuitive conclusion that high-quality concrete has a beneficial effect on durability regardless of the specific nature of the deteriorative influence. In the older literature, this is often referred to as concrete containing more cement; indeed, there is anecdotal evidence that some past advisors on concrete durability problems always simply said "use more cement." In the context of the then-current state of the practice, this automatically meant concrete of lower w/cm and hence lower permeability and greater abrasion resistance. More recently there have been assertions that durability is enhanced by use of certain controls on particle size distribution of aggregate. It is usually clear that the recommended gradations could have an effect to reduce water demand, and hence would, in service, tend to give a lower w/cm at given cement content and slump. Also, in addition to the lower water content, reduction in cement content's minimizing of chemical reactions and reduction in paste content's increasing of dimensional stability are expected with better gradations of aggregates.

It is also clear that many issues not related to materials selection or proportioning can have major effects on durability of concrete. These include primarily consolidation, finishing, and curing. Failure to adequately consolidate the fresh concrete results in honeycombing of concrete. Excessive vibration of many mixtures will induce segregation and may alter the air–void system. Improper finishing, especially of high w/cm mixtures, can result in surface concrete's becoming nonresistant to freezing and thawing due to its air–void system being damaged by excessive manipulation. Proper curing is essential for quality concretes. As stated in ACI 308, sometimes nothing needs to be done to cause concrete to possess a satisfactory temperature and moisture condition during its early stages so that the desired levels of relevant properties develop. However, it is not often possible to confidently predict in advance that the environment will be so favorable. Hence, intentional activity to properly cure concrete is often required for durability.

As the foregoing suggests, many factors can affect the durability of concrete. It is the hope of the committee whose members contributed to the preparation of this circular that use of the information contained here will make nondurable concrete an even rarer occurrence in transportation.

Table 1 lists types of materials-related distress that can occur to concrete in service in transportation, along with manifestations, causes, typical times of appearance, and methods of prevention or reduction (Van Dam et al., 1998).

TABLE 1 Factors in Concrete Durability

Type of Materials-Related Defect	Surface Distress Manifestations and Locations	Cause or Mechanisms	Time of Appearance	Prevention or Reduction
Due to Physical Mechanisms				
Freezing and thawing deterioration of hardened cement paste	Scaling or map cracking, generally initiating near joints or cracks; possible internal disruption of concrete matrix.	Deterioration of saturated cement paste due to repeated cycles of freezing and thawing.	1–5 years	Addition of air-entraining agent to establish protective air–void system.
Deicer scaling and deterioration	Scaling or crazing of the slab surface.	Deicing chemicals can amplify deterioration due to freezing and thawing and may interact chemically with cement hydration products.	1–5 years	Limiting w/cm ratio to no more than 0.45, and providing a minimum 30-day drying period after curing before allowing the use of deicers.
Deterioration of aggregate due to freezing and thawing	Cracking parallel to joints and cracks and later spalling; may be accompanied by surface staining.	Freezing and thawing of susceptible coarse aggregates results in fracturing or excessive dilation of aggregate.	10–15 years	Use of nonsusceptible aggregates or reduction in maximum coarse aggregate size.

(continued)

TABLE 1 (*continued*) **Factors in Concrete Durability**

Type of Materials-Related Defect	Surface Distress Manifestations and Locations	Causes or Mechanisms	Time of Appearance	Prevention or Reduction
Due to Chemical Mechanisms				
ASR	Map cracking (rarely more than 50 mm deep) over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Reaction between alkalis in cement and reactive silica in aggregate, resulting in an expansive gel and the degradation of the aggregate particle.	5–15 years	Use of non-susceptible aggregates, addition of pozzolans, limiting of alkalis in concrete, addition of lithium salts.
Alkali–carbonate reactivity	Map cracking over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Expansive reaction between alkalis in cement and carbonates in certain aggregates containing clay fractions.	5–15 years	Avoiding susceptible aggregates, or blending susceptible aggregate with nonreactive aggregate.
External sulfate attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Expansive formation of ettringite or gypsum that occurs when external sources of sulfate (e.g., groundwater, deicing chemicals) react with aluminates in cement or fly ash.	1–5 years	Minimizing tricalcium aluminate content in cement or using blended cements, class F fly ash, or GGBFS.
Internal sulfate attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Formation of ettringite from internal sources of sulfate that results in either expansive disruption on the paste phase or fills available air voids.	1–5 years	Minimizing tricalcium aluminate content in cement, using low sulfate cement, eliminating source of slowly soluble sulfate, and use cements conforming to ASTM C150, C595, or C1157, and avoiding high curing temperatures.
Corrosion of embedded steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel.	Chloride ions penetrate concrete and corrode embedded steel.	3–10 years	Reducing the permeability of the concrete, providing adequate concrete cover, and coating steel.

Material Selection

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Selecting materials to use in producing concrete for a specific application is a crucial step in avoiding durability problems. The ultimate goal is the economical production of durable concrete. The process should begin with an examination of the environmental and service conditions to which the concrete will be subjected during production and service to identify those deterioration mechanisms that should be protected against. Identifying those mechanisms that can be disregarded will enhance project engineering since it may be uneconomical to provide protection against environmental actions to which the concrete will not be exposed.

Understanding the characteristics of the individual constituent materials available for use is a core function of the materials engineer since the material's characteristics control its response to the durability stresses to which it is exposed. Certain problems can arise when inappropriate materials choices are made, such as using a high water-to-cementitious materials (w/cm) ratio in a concrete exposed to sulfates, but they can be avoided by engineers' knowledge and awareness of the characteristics of their materials. This chapter discusses the various constituent materials that are commonly used to produce hydraulic cement concrete used in transportation structures and the durability problems that are associated with them. Sections on proportioning and test methods follow.

CEMENTITIOUS MATERIALS

Cementitious materials, combined with water to produce a paste, play a key role in concrete by initially providing the fluidity necessary for mixing, placement, consolidation, and, later, the chemical reactions to bind the components into a solid mass with the requisite physical properties. Serving as the binding matrix of concrete, the chemical and physical stability of the paste phase, as well as its microstructure, are critical to the durability of the concrete. The properties of the paste phase depend on the characteristics of the cementitious materials and chemical admixtures as well as the w/cm ratio used in producing the concrete. For the purposes of this discussion, cementitious materials are divided into two classes: hydraulic cements and supplementary cementitious materials (SCMs). Included in the SCM category are ground granulated blast-furnace slag, fly ash, silica fume, and natural pozzolans. Additional information on cementitious materials can be found in American Concrete Institute (ACI) 225R, ACI 232.1R, ACI 233R, ACI 234R, ACAA 1995, Holland (2005), Taylor et al. (2007), and Johansen et al. (2006).

Hydraulic cements are produced from a clinker consisting essentially of crystalline calcium silicates. The calcium silicates are hydraulic in nature; that is, they react with water to produce calcium silicate hydrates that serve as the primary binding phase in concrete. Slag is an amorphous (glassy) material composed of calcium silicates and calcium aluminosilicates that possess some latent (slowly acting) hydraulic behavior. Pozzolans are generally composed of amorphous silicates and aluminosilicates. Most possess little or no hydraulic behavior, but do react with alkaline solutions to produce cementing hydrates. Certain pozzolans may contain sufficient calcium or other base metals (alkalis) to provide latent hydraulic behavior. The behavior of the cementing materials is a function of their chemistry and physical properties. Table 2 outlines some of the major factors affecting their behavior.

Hydraulic Cements

The four major compounds found in cement clinker are tricalcium silicate (C_3S) responsible for early (i.e., 28-day) strength, dicalcium silicate (C_2S) which reacts more slowly and provides later-age strength gain, tricalcium aluminate (C_3A) which reacts very rapidly generating heat and must be balanced in the cement with sulfate to prevent rapid stiffening, and tetracalcium aluminoferrite (C_4AF). The clinker is produced under tightly controlled conditions and is interground, usually with small amounts of a few additional components into a fine powder.

AASHTO M85 and ASTM C150 are specifications for portland cement. AASHTO and ASTM work actively together to keep differences to a minimum. The following types of portland cement are commonly available and have differing characteristics suited to be used for different purposes:

- Type I cement is used in concrete for general purposes. These include structures and pavements for general transportation, commercial and industrial applications.
- Type II (MS) cement has a limit on the amount of C_3A, which provides moderate resistance to sulfate attack.
- Type II (MH) limits the heat of hydration through a chemical limit. Type II (MH) is intended for use in more massive elements to control the development of thermal stresses.
- Type III cement is usually more finely ground than Type I and is intended for use when early strength is required. It is often used in precast operations where production requires rapid turnover of forms and other facilities, as well as in fast-track paving and patching

TABLE 2 Some Characteristics Affecting the Behavior of Cementitious Materials

Characteristic	Lower Values	Higher Values
Fineness	Slower hydration, but higher potential for continued long-term property development.	Faster hydration, rapid property development, low potential for long-term development.
Calcium oxide–silica ratio	Slower hydration and property development, but higher potential for long-term development; lower heat of hydration; higher capacity to bind alkalis in hydrates.	Faster hydration; increased potential for beneficial use of low CaO SCMs.
Tricalcium aluminate	Lower early heat generation; higher sulfate resistance.	
Aluminates		Higher capacity to bind chloride
Alkalis	Lower potential for alkali–aggregate reaction (AAR).	Increased efficiency of some air-entraining admixtures; increased potential for beneficial use of low CaO SCMs

NOTE: For more details, refer to Taylor et al. (2007) and Johansen et al. (2006).

operations where early opening to traffic is desired. Careful use of Type III cement is needed to ensure durability, especially at elevated curing temperatures.

- Type IV cement has a very low rate and amount of heat generated. It is intended for use in massive concrete structures to control temperature rise during hydration.
- Type V cement has a more restrictive limit on the C_3A content than Type II cement to provide higher resistance to sulfate attack. It is common in areas where the soils contain high concentrations of sulfate.

AASHTO M240 and ASTM C595 are prescriptive specifications for blended cements. AASHTO and ASTM are working together to harmonize the requirements of these specifications. Classification of blended cements is based on the type (pozzolan or slag) and amount of blending component. Further classification is based on whether the cement provides special attributes or not. Resistance to sulfate attack, either moderate or high, is indicated by the labels MS or HS respectively when compliance with specified expansion limits has been shown by testing with ASTM C1012. Compliance with limitations on the heat of hydration is indicated by the labels MH (moderate) or LH (low). In addition, an optional requirement can be invoked that assures the cement will not cause deleterious alkali–silica reactions (ASRs) with aggregates as indicated by testing with borosilicate glass aggregate in C227. This is a very rigorous requirement, so while compliance provides strong assurance that the cement will avoid ASR problems, a failure to comply does not necessarily mean that the cement would not provide sufficient protection with a given aggregate.

AASHTO M 240 and ASTM C595 have a classification scheme with three basic types depending on the blending component and cement bearing an indication of the percentage of the blending material:

- Type IS (*N*%) having slag at *N*% (up to 95% by mass);
- Type IP (*N*%) having pozzolan at *N*% (up to 40% by mass);
- Type IT (*N%*) (*M%*) which includes two pozzolans or a slag and a pozzolan, with similar limits on composition, with amounts identified in the designation; and
- Type IL (*N%*) having ground limestone at *N%* (up to 15% by mass).

ASTM C1157 is a performance specification for hydraulic cements. It has no requirements regarding composition or ingredients, but its focus is clearly on hydraulic cements within the calcium–silicate–aluminate system. Performance requirements are similar to those used in the blended cement specifications, with MS and HS labels indicating moderate and high sulfate resistance, and MH and LH indicating moderate and low heat of hydration, respectively. It also carries an option (designated by adding "R" after the cement type) for low reactivity with ASR aggregates.

Supplementary Cementitious Materials

Pozzolans commonly used include fly ash, silica fume, and metakaolin. These materials are usually added to concrete as a constituent of blended cement or at the concrete batch plant as a partial replacement of hydraulic cement. Although the use of these materials (notably fly ashes) is sometimes driven primarily by economics, most can enhance various aspects of concrete durability if proper consideration is given to their characteristics and the other materials with which they are being used. The chemical and physical characteristics of individual SCMs should be used in conjunction with knowledge of the characteristics of the other concreting materials and the required concrete properties and durability to guide their use.

The use of some of these materials at high cement replacement levels may be necessary to achieve certain desired results, but can reduce the early and 28-day strengths of concrete depending on the characteristics of the specific material used and the level of use. Using SCMs in tandem to produce ternary or even quaternary blends can avoid such problems; for instance, using small amounts of silica fume or metakaolin in combination with smaller amounts of fly ash or slag than would be needed to achieve the desired protection against chloride intrusion or ASR. Adjustments in w/cm can also be used to counterbalance slow strength gain characteristics when high replacement levels are needed.

AASHTO M295 and ASTM C618 are specifications for fly ashes and natural pozzolans used in concrete. Fly ashes are produced in coal-fired power plants and are broken into two classes based on the combined silica, alumina, and ferrite (SAF) content. For Class F, the SAF content exceeds 70%, while Class C has a minimum SAF of 50% and with CaO contents generally ranging between 10% and 30%. Some Class C fly ashes may contain crystalline C_3A which can cause rapid stiffening if not controlled by sulfate additions and which tend to have a negative impact on sulfate resistance. Natural pozzolans are derived from earth materials and are categorized as either raw, requiring no pyro-processing to provide pozzolanic behavior, or calcined, where pyro-processing is needed to activate the pozzolanic nature. Opal, diatomaceous earth, and pumicite are examples of materials in which the silicates and aluminosilicates are pozzolanic in the natural state. Clays, shales, and slates are examples of materials that require calcining to transform the silicates and aluminosilicates into a pozzolanic material. Metakalolin is a common term used for calcined kaolin, relatively pure aluminosilicate clay with very low alkali content.

AASHTO M307 and ASTM C1240 are specifications for silica fume, a byproduct of the ferrosilicon metal industry. It has relative pure amorphous silica of extreme fineness and thus a highly reactive and efficient pozzolan. AASHTO M321 is a specification for high reactivity pozzolanic materials, such as metakaolin or ultra-fine fly ash that calls attention to special attributes similar to those provided by silica fume and not specifically identified in AASHTO M295 or ASTM C618.

AASHTO M302 and ASTM C989 are the specifications covering ground, granulated, blast-furnace slag (also known as slag cement) used in concrete. Slag cements are classified into three grades: 120, 100, and 80, based on reactivity as measured by strength relative to a control. The reactivity of slag from a given source is a function of its fineness and composition.

Cementitious Materials and Durability

The cementitious paste phase of concrete plays a critical role in concrete durability. Its capillary system serves as the primary conduit for the movement of water and dissolved ionic species through the concrete. Most deterioration mechanisms affecting concrete in transportations systems involve either chemical reactions that depend on the presence of water or physical processes that result from moisture movement (freezing and thawing, volume instability due to shrinking and swelling). Controlling w/cm is a primary step that requires careful attention to materials selection and concrete production and is discussed in the chapter on proportioning. Fly ash, slag cement, and ground calcined clay generally improve the workability of concretes of equal slump and strength, whereas silica fume reduces workability unless the increased water demand is met by use of water-reducing admixtures to maintain slump at the same w/cm. Incremental improvements in workability at a given w/cm can also be achieved with the use of entrained air and attention to aggregate characteristics.

The use of SCMs either in blended cements or as individual constituents in a concrete mixture can provide significant improvement to the paste phase both in reducing capillary porosity and diffusivity and by binding ions into their hydrates that would otherwise be free to participate in deleterious reactions. However, the improvements in reduced porosity can be negated if the w/cm is not adequately controlled. Reductions in porosity and diffusivity are extremely important in preventing corrosion-related damage by inhibiting the penetration of chloride ions to the reinforcement. Additional benefit is provided by aluminate hydrates which can bind the chlorides. The binding of alkalis into silicate hydrates is a primary means for avoiding deleterious ASR with susceptible aggregates. Several test methods are available to measure the transport properties of concrete. AASHTO T277 and ASTM C1202 provide an indirect measure of resistance to chloride penetration. AASHTO T259 and ASTM C1543 are chloride ponding tests and ASTM C1556 measures the bulk diffusion of chloride into concrete. ASTM C1585 measures the rate at which water is absorbed into the capillary pore system.

SCMs, particularly the finer ones, can improve cohesiveness thus reducing bleeding and the potential for segregation in concretes. However, reduced bleeding can make concretes more prone to plastic shrinkage cracking, thus necessitating appropriate care during placement to avoid excessive evaporation and prompt application of curing.

Sulfate Attack

Sulfate ions in the pore solution of concrete can react with certain cement hydrates, particularly the calcium aluminates, causing damage to the matrix. Usually this occurs when the concrete will be exposed in service to soils or water containing sulfate. The level of protection needed is based on the concentration level of sulfate. For moderate concentrations, specify AASHTO M85 (ASTM C150) Type II, or Type II (MH), or AASHTO M240 (ASTM C595) Types IS (MS), IP (MS), or IT (MS), or ASTM C1157 Type MS cement or equivalent (Type I cement with an appropriate amount of ground slag or pozzolan). For higher concentrations of sulfate, specify Type V, IS (HS), IP (HS), IT (HS), or HS cement or equivalent (Type II cement with an appropriate amount of ground slag or pozzolan). Highly sulfate-resistant cement types can also be used in moderate sulfate exposures.

Limits on the w/cm are also required to inhibit the penetration of the solutions into concrete. ACI recommendations and solutions for concrete that will be exposed to sulfate are given in Table 3 (ACI 201.2R).

Similar reactions involving sulfate present in the concrete can also cause damage and is referred to as internal sulfate attack. An example of this is the inadvertent use of aggregates contaminated with gypsum, a sulfate mineral. Another example, over-sulfated cement, can be detected using ASTM C1038. Early exposure to excessively high temperatures such as in steam-curing operations can also trigger an abnormal sequence of reactions involving cement sulfates that can ultimately damage concrete caused by delayed ettringite formation.

Alkali–Aggregate Reactions

Low-alkali portland cements (less than 0.60% Na_2O equivalent; Na_2O equivalent = percent Na_2O + 0.658 × percent K_2O) provide some measure of protection against deleterious AARs, although above 0.40% the protection may not be sufficient with some particularly reactive aggregates. An alternate

TABLE 3 Recommendations for Concrete Exposed to Sulfate-Containing Solutions (ACI 201.2R)

Sulfate Exposure	Water-Soluble Sulfate (SO_4) in Soil, Percent by Mass[a]	Sulfate (SO_4) in Water, ppm[a]	Cementitous Material Requirements	Maximum w/cm, by Mass[b,c]
Class 0	0.00–0.10	0–150	No special requirements for sulfate resistance	No special requirements for sulfate resistance
Class 1	> 0.10 and < 0.20	>150 and <1,500	M85 (C150): Type II or equivalent[d]	0.50[c]
Class 2	0.10–0.20	150–1,500	M240 (C595): IP (MS), IS (MS), IT (MS), C1157: MS	0.45[c]
Class 3		M85	(C150): Type V**; M240 (C595): IP (HS), IS (HS); IT (HS); C1157: HS	0.40[c]
Class 3	0.20–2.00	1,500–10,000	M85 (C150): Type V**; M240 (C595): IP (HS), IS (HS); IT (HS); C1157: HS	See ACI 201.2R–08, Section 6.4

[a] Sulfate expressed as SO_4 is related to sulfate expressed as SO_3, as given in reports of chemical analysis of portland cements as follows: $SO_3\% \times 1.2 = SO_4\%$.
[b] ACI 318, Chapter 4, includes requirements for special exposure conditions such as steel-reinforced concrete that may be exposed to chlorides. For concrete likely to be subjected to these exposure conditions, the maximum w/cm should be that specified in ACI 318, Chapter 4.
[c] Values are applicable to normal weight concrete.
[d] Or equivalent. See ACI 201.2R for details.

approach is to limit the alkali loading of the concrete (alkali content × cement content of concrete) with a maximum imposed depending on the reactivity of the aggregate used. Lower limits are required for more reactive aggregates (e.g., 3.0 kg/m^3, 2.4 kg/m^3, or 1.8 kg/m^3). Hydraulic cements meeting the optional requirement of AASHTO M240 or ASTM C595 or C1157 provide excellent protection against the possibility of deleterious ASR for many, but not all, reactive aggregates. Cements that do not meet those rigorous criteria may provide sufficient control for a given aggregate or may need additional pozzolan or slag to adequately control reactivity. The amount of a given pozzolan or slag needed to protect against deleterious reactivity is a function of its chemistry, that of the other cementitious materials with which it is used, and the reactivity of aggregate they are combined with. Class C fly ashes, particularly those with high CaO contents, typically require higher amounts in a given situation than a Class F fly ash. Ground slag, with its relatively high CaO content is typically used at levels of 35% to 50% with high alkali cements, whereas 15% to 30% would be typical for a Class F fly ash under similar conditions.

MIXING WATER FOR CONCRETE

ASTM C1602 (Table 4 and Table 5) provides guidance on the quality of mixing water and optional water requirements, respectively. Potable water from a municipal or other source is considered to be of adequate quality.

AGGREGATES

Aggregates generally make up 70% to 85% of the mass of a concrete mixture. Their grading, size, mineralogical composition, porosity, surface texture, and shape greatly influence the properties of unhardened and hardened concrete. Effects on workability are described by Tattersall (1991). Obviously, any lack of durability of aggregates has a direct and undesirable consequence on the durability of concrete.

Another aggregate-related durability problem is resistance to freezing and thawing (Mindess and Young, 2003). An aggregate particle may absorb so much water that it cannot accommodate the expansion and hydraulic pressure that occur during the freezing of water. This will lead to expansion of the aggregate and possible disintegration (D-cracking) of the concrete. If such an aggregate particle is near the surface of the concrete, it can cause a popout. The resistance of an aggregate to freezing and thawing depends on its porosity, permeability, strength, degree of saturation, and size. For many aggregate types, there may be a critical particle size below which the distress due to freezing and thawing will not occur. For most aggregates,

TABLE 4 Physical Requirements for Concrete Mixing Water (ASTM C1602)

	Limits	Test Method
Compressive strength, min. % of control at 7 days	90	AASHTO T22, T23; ASTM C39, C31
Time of setting, deviation from control, h:min	1:00 early to 1:30 later	AASHTO T197; ASTM C403

TABLE 5 Chemical Requirements for Concrete Mixing Water (ASTM C1602)

Chemical Requirements Maximum Concentration in Mixing Water, ppm	Limits	Test Method
Chlorides, as Cl, ppm		ASTM C114
Prestressed concrete or bridge decks	500	
Other	1,000	
Sulfate as SO_4, ppm	3,000	ASTM C114
Alkalis (as $Na_2O + 0.658\ K_2O$), ppm	600	
Total solids by mass, ppm	50,000	ASTM C1603

the critical size is greater than the normal sizes used in practice. However, for some sedimentary rocks, such as chert, shale, and limestone, the critical size may be smaller than the maximum aggregate size used [in the range of 12.5 to 25.0 mm (0.5 to 1 in.)]. The resistance of aggregates to freezing and thawing can be evaluated either based on their past field performances, or by using a laboratory procedure such as ASTM C1646.

The thermal coefficient of expansion of concrete is largely a function of the aggregates used. The thermal coefficient of the aggregates is in turn a function of the minerals present and their relative amount in the aggregate. Quartz, a common constituent of many natural sands and many rocks has a relatively high thermal coefficient compared to other common minerals such as calcite, dolomite, and the feldspars. The cementitious paste generally has the highest thermal coefficient of the individual concrete constituents. A concrete made with quartz gravel coarse aggregate will have a higher thermal coefficient than a similar concrete made with a limestone coarse aggregate and be subject to higher thermal stress and the potential for resulting damage.

Certain aggregates are also susceptible to moisture-related volume instability. Alternate wetting and drying may cause excessive strain to develop leading to permanent increase in the volume of the concrete and eventually its breakdown.

AARs (Farny and Kerkhoff, 2007) were first identified as a durability concern for transportation concretes in 1940 (Stanton, 1940). Some aggregates contain constituents that are chemically unstable in high-alkalinity (high pH) environments such as exist in concrete. This occurs in concrete with moderate to high alkali (sodium and potassium) cement if the alkalis remain in solution. Two distinct reactions have been recognized. In the more prevalent, aggregates with certain forms of silica react with the highly alkaline pore solution. This is commonly referred to as ASR. ASR gel formed during the reaction can dramatically increase its volume when absorbing moisture. This may cause cracks in the concrete matrix and expansion of the concrete structure. The other reaction occurs with fairly select carbonate rocks, notably fine-grained dolomites and fine-grained dolomitic limestone, usually with significant insoluble residue content. Known as alkali–carbonate reaction (ACR) it is also expansive. The reaction is more complex than ASR, involving chemical breakdown of dolomite in the stone and formation of expansive reaction products.

To minimize the occurrence of damaging AARs, an aggregate's potential for deleterious reactivity should be considered during materials selection. Field service records generally provide good information for the evaluation of aggregates if the service is in excess of 10 to 15 years and information on the cementitious materials, particularly alkali content is available. ASTM standards C1260 (or AASHTO T303) and C1293 provide guidance in identifying AARs.

The potential for damage from ASR aggregates can be minimized by the selection of cementitious materials that limit the alkalinity of the pore solution, such as imposed limits on cement or concrete alkali content or the use of blended cements or SCMs in sufficient amounts to reduce the permeability and available alkalis. The addition of lithium compounds to the mixture has also been shown to mitigate deleterious ASR (Thomas et al., 2007). ASTM C227 and C441 are used to evaluate cements and cementitious materials with a standard reactive borosilicate glass aggregate without the addition of excess alkali. C1293 and C1567 are used to evaluate specific aggregates to determine the amount of SCM needed to suppress expansion under defined high-alkali conditions; however, because acceleration of these tests is achieved using an alkaline solution or alkali added to the mixture, they are not appropriate for evaluating materials that rely on limiting alkali content for effectiveness.

For ACR, very strict limits on the alkali content of the cement or concrete, blending of reactive aggregates with nonreactive ones, and limiting the maximum size of the coarse reactive aggregate may help minimize the potential for damage (Newlon and Sherwood, 1964). Slag cement has been reported to be ineffective in controlling ACR (Rogers and Hooton, 1992).

The use of certain deicers, most notably those containing calcium magnesium acetate or magnesium chloride, is thought to exacerbate problems with ACR, while potassium acetate and similar deicers have been associated with increased potential for ASR. Research is underway to confirm these findings and provide appropriate guidance.

The selection of a proper particle shape, surface texture, and grading is also important. Since aggregates generally occupy the majority of the volume in the concrete, workability of concrete is greatly affected by gradation, size, surface texture, and shape of aggregates (Tattersall, 1991).

CHEMICAL ADMIXTURES

The durability of concrete and reinforced concrete can be significantly enhanced with the use of various chemical admixtures. These include air-entraining admixtures for freezing and thawing resistance. Water-reducing and high-range water-reducing admixtures (superplasticizers) reduce the water content, the water–cement ratio or w/cm, all of which result in lower permeability to aggressive elements. The corrosion inhibitors improve corrosion resistance in the presence of chloride ions or reduced pH, and ASR inhibitors control alkali–silica reactivity. Shrinkage-reducing admixtures (SRAs) reduce drying shrinkage cracking and ultimately lower the permeability.

The following sections discuss the various chemical admixtures that can be used to enhance durability. Air-entraining admixtures are not covered below as they are discussed elsewhere (Whiting and Nagi, 1998). A general review of most of the chemical admixtures is given in *Transportation Research Circular 365: Admixtures and Ground Slag for Concrete* (1990), and as such this document will focus on their effects related to durability.

Water-Reducing Admixtures

The primary durability benefits from water reduction arise if the reduction in water is used to lower the w/cm and thus the permeability of the concrete. This results in a reduction of the rate of ingress of potentially harmful substances such as chloride ions in marine environments or where chloride-containing deicing chemicals are used. In general, lower w/cm reduces the permeability and vice versa. Reduced carbonation and improved resistance to chemical attack are additional benefits.

Secondary benefits of water-reducing admixtures are that they allow the achievement of low w/cm values without increasing cement content that can lead to increased drying shrinkage and thermal stresses. In addition, they can improve workability at a given w/cm so that consolidation of the concrete is improved.

These products are specified and classified in AASHTO M194 (ASTM C494): Standard Specification for Chemical Admixtures. Water-reducing admixtures produce a minimum of 5% reduction in water and are classified as Type A, water-reducing; Type D, water-reducing admixtures and retarding admixtures; and Type E, water-reducing and accelerating admixtures.

The Type E water-reducing admixtures should not contain chloride if they are to be used with embedded steel. High-range water-reducing admixtures provide a 12% or greater reduction in water and are classified as Type F, high-range water-reducing admixtures, or as Type G, high-range water-reducing and retarding admixtures.

Types F and G high-range water-reducing admixtures are also covered by ASTM C1017, which is the specification for chemical admixtures for use in producing flowing concrete. Type F usually falls under the plasticizing (Type 1) classification and Type G under the plasticizing and retarding (Type 2) classification.

Typical chemical compositions of water-reducing and high-range water-reducing admixtures are given in *Transportation Research Circular 365* (1990).

A new group of high-range water-reducing admixtures has been introduced. The products in this group are based upon polycarboxylates. In general, dosage rates for the same level of water reduction are lower for these high-range, water-reducing admixtures than they are for the products discussed earlier.

When some high-fineness pozzolans are added, especially silica fume, high-range water-reducing admixtures must be used to help disperse the pozzolan and compensate for the high water demand that is incurred due to the very high surface area. If this is not done, the expected permeability reductions will not be achieved, due both to the higher water content or the decreased dispersion or both.

Corrosion Inhibitors

Corrosion inhibitors provide protection to embedded steel in concrete by reducing the corrosion rate in the presence of chloride ions. They act by limiting either the anodic or cathodic electrochemical reactions involved in the corrosion process. But they are not a substitute for good quality concrete, and guidelines for reducing chloride ingress must be followed.

In alkaline environments, such as concrete, a natural iron oxide forms on the surface of the steel. This oxide layer consists of two types of oxides: ferrous oxide and ferric oxide. Ferrous oxide, though stable in alkaline environments, reacts with chloride ions to form complexes that move away from the steel to form rust. The chloride ions are released to attack the steel again. Eventually, the entire passivating oxide layer is undermined.

It is theorized that anodic inhibitors, such as nitrites, help to promote the formation of the protective ferric oxide layer that is resistant to attack by chloride ions, thus inhibiting corrosion. Cathodic inhibitors react with the surface to interfere with the reduction of oxygen. The reduction of oxygen is the principal cathodic reaction in alkaline environments.

Corrosion-inhibiting admixtures can affect the unhardened and hardened properties of the concrete. Produce trial mixtures to determine concrete performance parameters.

Commercially available corrosion inhibitors include calcium nitrite, sodium nitrite, and a mixture of amines and esters, dimethyl ethanol amine, amines, and phosphates. Performance of inhibitors in concrete is discussed elsewhere (Berke and Weil, 1994; Nmai and Kraus, 1994). With the exception of calcium nitrite, few, if any, long-term performance data beyond 5 to 10 years are available. Short-term data are available (Berke et al., 1994; Maeder, 1996; Johnson et al., 1996; Vogelsang and Meyer, 1996). However, caution must be exercised in the evaluation of short-term results, which can be misleading (Berke et al., 1994).

The long-term benefits of calcium nitrite are well documented (Berke and Weil, 1994; Nmai and Kraus, 1994; Berke and Hicks, 1996; Berke et al., 1997; Virmani, 1990; Tomosawa et al., 1990). Based upon these results, relationships were developed to indicate the level of chloride against which a given addition of 30% calcium nitrite protects.

Performance criteria for an amine and ester commercially available inhibitor were given by Johnson et al. (1996). This inhibitor, at a dosage of 5 L/m^3, was stated to protect up to 2.4 kg/m^3 of chloride. A reduction in the chloride diffusion coefficient of 22% to 43%, depending on concrete quality, was determined, using accelerated test methods.

ASR Inhibitors

Several compounds have been investigated for use as admixtures in concrete to control ASR damage. Lithium compounds are the best-known ASR inhibitors. When ASR occurs, in the presence of lithium ions, a minimally expansive lithium-bearing ASR gel is formed. This is generally not damaging to the concrete (Farny and Kerkhoff, 2007). While many lithium compounds could be used for this purpose, lithium nitrate ($LiNO_3$) is considered more effective while being safer to handle and is therefore the more desirable additive.

The effectiveness of lithium compounds in preventing deleterious ASR depends on the lithium compound used, the addition rate, the aggregate reactivity, and the cement alkalinity. It is noted that ASTM C1260 cannot be used to assess the effectiveness of lithium compounds, or determine the appropriate addition rate (Farny and Kerkhoff, 2007), but a method for doing so is described in Thomas et al. (2008).

Shrinkage-Reducing Admixtures

SRAs significantly reduce drying shrinkage that often causes cracking in restrained concrete (Nmai et al., 1998; Shah et al., 1998). A reduction in cracking or crack size should improve durability.

SRA can affect properties of freshly mixed and hardened concrete. A recent study shows that they are compatible with other durability-enhancing admixtures, and they might have some additional benefits in slightly reducing chloride ingress (Berke et al., 1996; Berke et al., 1997).

Proportioning

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The objective of proportioning concrete mixtures is to determine the most economical and practical combination of readily available materials to produce a concrete that will satisfy the service requirements under the particular conditions of use.

MIXTURE CHARACTERISTICS

Mixture characteristics are selected based on the construction methods and intended use of the concrete, the exposure conditions, the size and shape of concrete members, and the physical properties of the concrete (such as strength) required for a particular structure. The concrete durability properties, such as resistance to freezing and thawing, or resistance to chloride penetration, should be verifiable with the appropriate test methods specified. The old practice of using a simple proportion specification in the hopes that it will meet the needs of a modern construction project in terms of placing rate, strength gain, and durability is no longer appropriate.

Once the characteristics are selected, the mixture can be proportioned based on field or laboratory measured aggregate properties. Since the quality of the cementitious paste has a large effect on the properties of the hardened concrete, the first step in proportioning a concrete mixture is the selection of the appropriate water-to-cementitious materials ratio (w/cm) for the durability and strength needed.

Concrete mixtures should be kept as simple as possible, since an excessive number of ingredients can often make a concrete mixture difficult to control. The concrete technologist should not, however, overlook the opportunities provided by modern concrete technology.

WATER–CEMENTITIOUS MATERIALS RATIO AND STRENGTH RELATIONSHIP

Strength (compressive or flexural) is the most frequently used measure of concrete quality. While strength is an important characteristic, durability is now recognized as being equally or more important, especially when life-cycle designs of structures are considered.

For properly consolidated concrete made with sound and clean aggregates, the strength and other desirable properties of concrete under given job conditions are governed by the quantity of mixing water used per unit of cementitious materials: expressed on mass basis, this is the w/cm. Within the normal range of strengths in concrete construction, the strength is inversely related to the w/cm.

Differences in strength for a given w/cm may result from changes in the nominal maximum size of the aggregate, grading, surface texture, shape, strength, and stiffness, as well as from differences in types and sources of cementitious materials, air content, the presence of chemical admixtures, and the length of curing time.

STRENGTH

The specified compressive strength, f'_c, at 28 days, is the strength that is expected to be equaled or exceeded by the average of any set of three consecutive strength tests with a 99% probability. Flexural strength is sometimes used on paving projects instead of compressive strength. A mixture-specific relationship between compressive and flexural strength can be predetermined and the acceptance can be based on the compressive strength (Table 6).

The average strength must exceed the specified strength since the average must be selected so that only a small percentage (<1%) of all tests would fall below the specific strength. The required average strength is called f'_{cr}; it is the strength required of the selected mixture.

WATER–CEMENTITIOUS MATERIALS RATIO

The w/cm ratio is simply the mass of water divided by the mass of cementitious material. Different cementitious materials will result in different concrete characteristics. Select the w/cm ratio such that it is the value not to be exceeded that is required to meet the exposure considerations. For corrosion protection of reinforcing steel the ratio should not exceed 0.40 (with a minimum strength of 35 MPa) and, for frost resistance, 0.45 (with a minimum strength of 31 MPa). See Table 3 for recommendations for sulfate exposures. Sulfate resistance has been demonstrated by field performance to increase as the w/cm is reduced. Some state specifications require lower ratios for durability when high-performance concrete is specified.

When durability is not a controlling factor, the w/cm should be selected on the basis of concrete compressive strength. In such cases the w/cm and mixture proportions for the required strength should be based on adequate field data or trial mixtures made with actual job materials to determine the relationship between the w/cm and strength. Table 6 can be used to select a w/cm with respect to the required average strength, f'_{cr}, for trial mixtures when no other data are available [American Concrete Institute (ACI) 211.1].

TABLE 6 Typical Relationship Between w/cm and Compressive Strength of Concrete

Compressive Strength at 28 Days, MPa[a]	w/cm by Mass	
	Non-Air-Entrained Concrete	Air-Entrained Concrete
45	0.38	0.30
40	0.42	0.34
35	0.47	0.39
30	0.54	0.45
25	0.61	0.52
20	0.69	0.60

[a] This relationship assumes nominal maximum size of aggregate of about 19.0 or 25.0 mm (0.75 or 1.0 in.) (ACI 211.1).

AGGREGATES

The grading (particle size distribution) and the nature of particles (shape, porosity, surface texture) are characteristics of aggregates that have an important influence on the workability of the fresh concrete, and to some degree, the performance of the hardened concrete and the economics of concrete.

Grading is important for attaining an economical mixture because poor grading requires more water and more cementitious material. Coarse aggregates should be graded up to the largest nominal maximum size practical under job conditions. The use of large aggregates reduces the demand for paste, decreases shrinkage, and provides better aggregate interlock at pavement joints and cracks. The nominal maximum size that can be used depends on the size and shape of the concrete member to be cast, as well as on the amount and distribution of reinforcing steel in the concrete member. The maximum size of coarse aggregate should not exceed one-fifth the minimum distance between sides of forms nor three-fourths the clear space between individual reinforcing bars or wire, bundles of bars, or prestressing tendons or ducts. For unreinforced slabs on ground, the maximum size should not exceed one-third the slab thickness. The maximum aggregate size is limited to prevent bridging of aggregates leading to poor consolidation. Smaller sizes can be used when availability or economic considerations require them.

Grading influences the workability and placeability of the concrete. Sometimes midsized aggregate, around the 9.5-mm size, is lacking in an aggregate supply, resulting in a concrete with high shrinkage properties, high water demand, and poor workability and placeability, which could also affect durability. Efforts can be made to approach ideal grading through blending of aggregate sources (Shilstone, 1990). If problems develop due to poor grading, then alternative aggregates, blending, or special screening of existing aggregates should be considered. Refer to Shilstone (1990) or Taylor et al. (2007) for options on desirable aggregate gradations.

The amount of mixing water required to produce a cubic meter of concrete of a given slump is dependent on the nominal maximum size and shape and the amount of coarse aggregate. Also, rounded aggregate requires less water than crushed aggregate in concretes of equal slump.

The most desirable fine aggregate grading will depend upon the type of work, the richness of the mixture, and the size of the coarse aggregate. For leaner mixtures a fine grading (lower fineness modulus) is desirable for workability. For richer mixtures a coarse grading (higher fineness modulus) is used for greater economy.

ACI 211.1 (Table 7) provides guidance on the volume of coarse aggregate, and ASTM C33 provides guidance on the requirements for coarse aggregates.

In some parts of the country, the chemically bound chloride in aggregate may make it difficult for concrete to pass chloride limits set on concrete by ACI 318 or other codes or specifications. In such cases some or all of the chloride in the aggregate can be considered not to be available for participation in corrosion of reinforcing steel, resulting in that chloride being ignored. ACI 222R also provides guidance.

ENTRAINED AIR

Entrained air must be used in all concrete that will be exposed to freezing and thawing and deicing chemicals. It can also be used to improve workability even where not required for durability.

Air entrainment is accomplished by using an air-entraining portland cement or by adding an air-entraining admixture at the mixer. The amount of admixture should be adjusted to meet variations in concrete ingredients and job conditions. The amount recommended by the admixture manufacturer will, in most cases, produce the desired air content. Whiting and Nagi (1998) provide guidance on controlling air in concrete, including tips on adjustments to mixture proportions.

Recommended target air contents by ACI for air-entrained concrete are shown in Table 8. Note that the amount of air required for adequate resistance to freezing and thawing is dependent upon the nominal maximum size of aggregate and the level of exposure. Air is entrained in the mortar fraction of the concrete; in properly proportioned mixtures, the mortar content decreases as nominal maximum aggregate size increases, thus decreasing the required concrete air content. The levels of exposure are defined by ACI 211.1, as follows in Tables 7 and 8.

- Mild exposure. This exposure includes indoor or outdoor service in a climate where concrete will not be exposed to freezing or deicing agents. When air entrainment is desired for a beneficial effect other than durability, such as to improve workability or cohesion or in remedying the effects of low cement-content concrete, air contents lower than those needed for durability can be used.

TABLE 7 Volume of Coarse Aggregate per Unit of Volume of Concrete

Nominal Maximum Size of Aggregate, mm	Volume of Dry-Rodded Coarse Aggregate[a] per Unit Volume of Concrete for Different Fineness Moduli of Fine Aggregate			
	2.40	**2.60**	**2.80**	**3.00**
9.5	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
19	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
37.5	0.75	0.73	0.71	0.69
50	0.78	0.76	0.74	0.72
75	0.82	0.80	0.78	0.76

[a] Bulk volumes are based on aggregates in dry-rodded condition as described in AASHTO T19 (ASTM C29). These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For less workable concrete, such as required for concrete pavement construction, they may be increased by about 10%. For more workable concrete, such as may sometimes be required when placement is to be by pumping, they may be reduced by up to 10% (ACI 211.1).

TABLE 8 Approximate Air Content Requirements for Different and Nominal Maximum Sizes of Aggregate

	Air Content (percentage)							
Max. size (mm)	**9.5**	**12.5**	**19**	**25**	**37.5**	**50**	**75**	**150**
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

Source: ACI 211.1.

- Moderate exposure. This exposure includes service in a climate where freezing is expected but where the concrete will not be continually exposed to moisture or free water for long periods prior to freezing and will not be exposed to deicing or other aggressive chemicals. Examples include exterior beams, columns, walls, girders, or slabs that are not in contact with wet soil and are so located that they will not receive direct applications of deicing chemicals.
- Severe exposure. Concrete that is exposed to deicing or other aggressive chemicals or where the concrete may become highly saturated by continual contact with moisture or free water prior to freezing. Examples include pavements, bridge decks, curbs, gutters, sidewalks, canal linings, or exterior water tanks or sumps.

When mixing water is held constant, the entrainment of air will increase slump. When cement content and slump are held constant, the entrainment of air results in the need for less mixing water, particularly in leaner concrete mixtures. In batch adjustments, in order to maintain a constant slump while changing the air content, the water should be decreased by about 3 kg/m^3 for each percentage point increase in air content or increased 3 kg/m^3 for each percentage point decrease.

Specific air content cannot be readily or repeatedly achieved because of the many variables affecting air content; therefore, a permissible range of air contents around a target value must be provided. Although a range of 2% around the target value is often used in project specifications, it is sometimes an impractical limit. A more practical range is 3% (i.e., ±1.5% around the target value).

SLUMP

Slump is a measure of yield stress. Workability depends on yield stress and plastic viscosity. Concrete is to be produced with workability, consistency, and plasticity suitable for job conditions. Workability is a measure of how easy or difficult it is to place, consolidate, and finish concrete. Consistency is the ability of freshly mixed concrete to flow. Plasticity determines the concrete’s ease of molding. If more aggregate is used in a concrete mixture or if less water is added, the mixture becomes stiffer (less plastic and less workable) and difficult to mold. Neither very dry, crumbly mixtures nor very watery, fluid mixtures can be regarded as plastic.

The slump test is a measure of concrete consistency. For given proportions of cement and aggregate without admixtures, the higher the slump, the more fluid the mixture. The aggregate size, grading, and shape affect the workability. Slump is indicative of workability when similar mixtures are assessed. However, slump should not be used to compare significantly different mixtures. When used with different batches of the same mixture, a change in slump indicates a change in consistency and in the characteristics of materials, mixture proportions, or water content.

Different slumps are needed for various types of concrete construction. Slump is usually indicated in the job specifications as a range, such as 50 to 100 mm, or as a maximum value not to be exceeded. Slumps for pavements are typically 25 to 75 mm and, for structural concrete, 100 to 150 mm. For batch adjustments, the slump can be increased by about 10 mm by adding 2 kg/m^3 of water to the mixture.

WATER CONTENT

The required water content of concrete is related to a number of factors: aggregate size, grading, and shape; slump; w/cm ratio; air content; cementitious materials content; admixtures; and environmental conditions. Increase of air content and aggregate size, reduction in w/cm and slump; increase of rounded aggregates; and the use of water-reducing admixtures or fly ash reduce water demand. On the other hand, increase of temperature, cement content, slump, w/cm, aggregate angularity, and a decrease of coarse aggregate to fine aggregate ratio, increase water demand.

It should be kept in mind that changing the amount of any single ingredient in a concrete mixture can have significant effects on the proportions of other ingredients, as well as alter the properties of the mixture. For example, the addition of 2 kg/m^3 water will increase the slump by approximately 10 mm and will also increase the air content. In mixture adjustments, a decrease in air content by 1 percentage point will increase the water demand by about 3 kg/m^3 of concrete for the same slump.

CEMENTITIOUS MATERIALS CONTENT

The amount of cementitious material is usually determined from the selected w/cm and water content. The w/cm is related to the strength and durability characteristics. A minimum amount of cementitious material is sometimes included in specifications in addition to a maximum water–cement ratio or w/cm ratio. However, as was explained in the Materials Selection section of this circular, the reasons for so doing are invalid. In spite of this, some agencies have specified a minimum cementitious material content of 335 kg/m^3 of concrete in severe exposures. It is preferable to use performance-based specifications requirements. This allows the mixture proportions to optimize cementitious material combinations.

To make it economic, proportioning should minimize the amount of cementitious material required without sacrificing concrete quality. Since quality depends primarily on the w/cm, the water content should be held to a minimum to reduce the amount of cementitious material. Steps to minimize water and cementitious material requirements include use of (*a*) the stiffest practical mixture, (*b*) the largest practical nominal maximum size of aggregate, (*c*) the optimum ratio of fine-to-coarse aggregate, and (*d*) a uniform distribution of aggregate to minimize paste demand.

AASHTO M85 (ASTM C150), AASHTO M240 (ASTM C595), or ASTM C1157 detail the requirements for cements. Table 2 provides guidance on producing concrete exposed to sulfate conditions.

POZZOLANS AND SLAG

Pozzolans and ground granulated blast-furnace slag (also known as slag cement) can have varied effects on water demand and air content. The addition of fly ash will generally reduce water demand and decrease the air content if no adjustment in the amount of air-entraining admixture is made. Silica fume increases water demand and decreases air content. Slag and metakaolin have a minimal effect at normal dosages.

CHEMICAL ADMIXTURES

Air-entraining admixtures are used for resistance to cycles of freezing and thawing. Water-reducing admixtures are added to concrete to reduce the w/cm, to reduce the amount of cementitious material, or to improve the workability. Water-reducing admixtures usually will decrease water contents by 5% to 10% and several will also increase air contents by one-half to one percentage point. Retarders may also increase the air content.

High-range water-reducing admixtures, also called "superplasticizers," reduce water contents by between 12% and 30% and some can simultaneously increase the air content by up to one percentage point; others can reduce the air content or not affect it.

Calcium chloride-based accelerating admixtures may reduce water contents by about 3% and increase the air content by about one-half percentage point. When using a chloride-based admixture, the risks of reinforcing steel corrosion should be considered. ACI 318 and ACI 222R provide guidance on accelerating admixtures.

When using more than one admixture in concrete, the compatibility of intermixing admixtures should be assured by the admixture manufacturer, or the combination of admixtures should be tested in trial batches. Many admixtures contain water, which should be considered part of the mixing water if it affects the w/cm ratio by 0.01 or more. AASHTO M194 (ASTM C494) or ASTM C1017 note the requirements for admixtures.

PROPORTIONING

Proportioning methods have evolved from the arbitrary volumetric method (1:2:3—cement:sand:coarse aggregate) of the early 1900s to the present-day mass and absolute-volume methods described in ACI 211.1. Mass proportioning methods are fairly simple and quick for estimating mixture proportions using an assumed or known mass of the concrete per unit volume. A more accurate method, absolute volume, involves use of density or specific gravity values for all the ingredients to calculate the absolute volume each will occupy in a unit volume of concrete. A concrete mixture can be proportioned from statistical data on field experience or from trial mixture data.

The proportions of a presently or previously used concrete mixture can be used for a new project if strength test data and standard deviations show that the mixture is acceptable. Durability aspects previously presented need also be met. The statistical data should essentially represent the same materials, proportions, and concreting conditions to be used in the new project. The data used for proportioning should also be from a concrete with an f'_c within 7 MPa of the strength required for the proposed work.

The standard deviation is then used in Equations 1 and 2. The average compressive strength from the test record must equal or exceed the ACI 318 required average compressive strength, f'_{cr}, in order for the concrete proportions to be acceptable. The f'_{cr} for the selected mixture proportions is equal to the larger of Equations 1 and 2.

$$f'_{cr} = f'_c + 1.34S \tag{1}$$

$$f'_{cr} = f'_c + 2.33S - 3.45 \tag{2}$$

where

f'_{cr} = required average compressive strength of concrete used as the basis for selection of concrete proportions, MPa;
f'_c = specified compressive strength of concrete, MPa; and
S = standard deviation, MPa.

A field strength record, several strength test records, or tests from trial mixtures must be used for documentation, showing that the average strength of the mixture is equal to or greater than f'_{cr}.

Proportioning by Trial Mixtures

When field test records are not available or are insufficient for proportioning by field experience methods, the concrete proportions selected should be based on trial mixtures. The trial mixtures should use the materials proposed for the work. At least three mixtures with three different w/cm and cementitious material contents should be made to produce a range of strengths that encompass f'_{cr}.

Prepare trial mixtures with a slump and air content within ±20 mm and ± 0.5%, respectively, of the maximum permitted. Make and Cure three cylinders per mixture in accordance with AASHTO T126 (ASTM C192). At 28 days or the designated test age, the compressive strength of the concrete is determined by testing of the cylinders in compression. The test results are plotted to produce a strength versus w/cm curve that is used to select the mixture proportions.

A number of different methods of proportioning concrete ingredients have been used at one time or another.

The best approach is to select proportions based on past experience and reliable test data with an established relationship between strength and w/cm for the materials to be used in the concrete. The mixtures can be relatively small batches made with laboratory precision or job-size batches made during the course of normal concrete production. Use of both is often necessary to reach a satisfactory job mixture.

The following parameters must be selected first: required strength, cementitious material content or maximum w/cm, nominal maximum size of aggregate, air content, and desired slump. Trial batches are then made, varying the relative amounts of fine and coarse aggregates, as well as other ingredients. Based on considerations of workability and economy, the mixture proportions are selected.

When the quality of the concrete mixture is specified by w/cm, the trial batch procedure consists essentially of combining a paste (water, cementitious material, and, generally, an air-entraining admixture) with the necessary amounts of fine and coarse aggregates to produce the required slump and workability. Quantities per cubic meter are then calculated.

Representative samples of the cementitious materials, water, aggregates, and admixtures must be used. To simplify calculations and eliminate error caused by variations in aggregate moisture content, the aggregates should be prewetted then dried to a saturated surface dry (SSD) condition and placed in covered containers to keep them in this condition until they are used. The moisture content of the aggregates should be determined and the batched quantities corrected accordingly.

The size of the trial batch is dependent on the equipment available and on the number and size of test specimens to be made. AASHTO T126 (ASTM C192) provide guidance on mixing procedures.

Measurements and Calculations

Tests for slump, air content, and temperature should be made on the trial mixture, and the following measurements and calculations should be performed.

Mass Density and Yield

The mass density (unit weight) of freshly mixed concrete is reported in kilograms per cubic meter. The yield is the volume of fresh concrete produced in a batch, expressed in cubic meters. The yield is calculated by dividing the total mass of the materials batched by the mass density of the freshly mixed concrete [see AASHTO T121 (ASTM C138)].

Absolute Volume

The absolute volume of granular materials such as cementitious materials or aggregates is the volume of the solid matter in the particles; it does not include the volume of the spaces between particles. The volume of freshly mixed concrete is equal to the sum of the absolute volumes of the cementitious materials, water (exclusive of that absorbed in the aggregate particles), aggregates, admixtures when applicable, and air. The absolute volume is equal to the mass (kg) of the ingredient divided by the product of its relative density (specific gravity) times the density of water (1,000 kg/m^3):

$$\text{Absolute volume (m}^3\text{)} = \text{kg of loose material/(relative density} \times 1{,}000\ \text{kg/m}^3\text{)}$$

A value of 3.15 can be used for the relative density of portland cement, and a value of 2.5 to 3.1 for blended cement. Fly ash has a relative density in the range of 1.9 to 2.8. Silica fume and slag typically have values of 2.2 and 2.9, respectively. The relative density (specific gravity) of water is 1 and the mass density (unit weight) of water is 1,000 kg/m^3. The relative density of normal weight aggregate usually is between 2.4 and 2.9. The relative density (specific gravity) of aggregate as used in mixture calculations is the bulk relative density (specific gravity) of either saturated surface-dry material or oven-dry material. Relative densities of admixtures, such as water reducers, must also be considered.

The absolute volume of air in concrete is equal to the air-content percentage divided by 100 (e.g., 7% ÷ 100) and then multiplied by the volume of the concrete batch.

The volume of concrete in the batch can be determined by either of two methods: (*a*) if the relative densities (specific gravities) of the aggregates and cementitious materials are known, these can be used to calculate concrete volume; and (*b*) if relative densities (specific gravities) are unknown or varying, the volume can be computed by dividing the total mass of materials in the mixer by the density of concrete. In some cases, both determinations are made, one serving as a check on the other.

MIXTURE PROPORTIONING EXAMPLE

The ACI 211.1 volumetric method is illustrated below.

Conditions and Specifications

Concrete is required for a pavement that will be exposed to moisture in a severe freezing and thawing environment. A specified compressive strength, f'_c, of 40 MPa is required at 28 days. Air entrainment is required. Slump should be 50 ± 25 mm. The materials available are the following:

Cement:	Type GU, ASTM C1157. Relative density of 3.0.
Coarse aggregate:	Well-graded 25.0-mm nominal maximum-size rounded gravel (ASTM C33) with an oven-dry relative density of 2.68, absorption of 0.5% (moisture content at SSD condition), and oven-dry rodded density of 1,600 kg/m^3. The laboratory sample for trial batching has a moisture content of 2%.
Fine aggregate:	Natural sand (ASTM C33) with an oven-dry relative density of 2.64 and absorption of 0.7%. The laboratory sample moisture content is 6%. The fineness modulus is 2.80.
Water:	Potable.
Air-entraining admixture:	Wood-resin type, AASHTO M154 (ASTM C260).
Water reducer:	AASHTO M194 (ASTM C494), this particular admixture is known to reduce water demand by 10% when used at a dose of 3 mL per kg of cement. Assume that the chemical admixtures have a density close to that of water, meaning that 1 mL of admixture has a mass of 1 g.

From this information, the task is to proportion a concrete mixture that will meet the above requirements.

Water–Cement Ratio and Strength

Data on previous mixtures indicate that a w/cm of 0.40 should achieve a strength exceeding 40 MPa using local materials. For an environment with moist freezing and thawing, the maximum w/cm should be 0.45. Use 0.40.

Air Content

For severe freezing and thawing exposure, ACI 211.1 recommends a target air content of 6.0%. Therefore, proportion the mixture for 6 ± 1% air and use 8% (or the maximum allowable) for batch proportions.

Water Content

Experience with these local materials indicates that a water demand of 140 kg/m^3 is required to achieve the desired slump.

Cement Content

The cement content is based on the w/cm and the water content. Therefore, 140 kg/m^3 of water divided by a w/cm of 0.40 requires a cement content of 350 kg/m^3.

Coarse Aggregate Content

The quantity of 25.0-mm nominal maximum-size coarse aggregate can be estimated from Table 6. The bulk volume of coarse aggregate recommended when using a fine aggregate with a fineness modulus of 2.80 is 0.67. Since it has a mass of 1,600 kg/m^3, the oven-dry mass of coarse aggregate for a cubic meter of concrete is

$$1{,}600 \times 0.67 = 1{,}072 \text{ kg}$$

Admixture Content

For 5% to 8% air content, the air-entraining admixture manufacturer recommends a dosage rate of 0.5 g/kg of cement. From this information, the amount of air-entraining admixture per cubic meter of concrete is

$$0.5 \times 350 = 175 \text{ g}$$

The water-reducing admixture dosage rate of 3 g/kg of cement results in

$3 \times 350 = 1{,}050$ g of water reducer per cubic meter of concrete.

Fine Aggregate Content

At this point, the amounts of all ingredients except the fine aggregate are known. In the absolute-volume method, the volume of fine aggregate is determined by subtracting the absolute volume of the known ingredients from 1 m^3. The absolute volume of the water, cement, admixtures, and coarse aggregate is calculated by dividing the known mass of each by the product of their relative density and the density of water. Volume computations are as follows:

Water	$140/(1 \times 1{,}000) = 0.140\ m^3$
Cement	$350/(3.0 \times 1{,}000) = 0.117\ m^3$
Air	$8.0/100 = 0.080\ m^3$
Coarse aggregate	$1{,}072/(2.68 \times 1{,}000) = 0.400\ m^3$
Total volume of known ingredients	$0.737\ m^3$

The liquid admixture volume is generally too insignificant to be included in these calculations. However, certain admixtures—such as some accelerating, high-range water-reducing, or corrosion-reducing admixtures—are exceptions due to their large dosage rates, and their volumes should be included.

The calculated absolute volume of fine aggregate is then

$$1\ m^3 - 0.737\ m^3 = 0.263\ m^3$$

The mass of dry fine aggregate is

$$0.263\ m^3 \times 2.64\ kg/m^3 \times 1{,}000 = 694\ kg$$

The mixture then has the following proportions for 1 m^3 of concrete:

Water	140 kg
Cement	350 kg
Coarse aggregate (dry)	1,072 kg
Fine aggregate (dry)	694 kg
Total mass	2,256 kg
Air-entraining admixture	0.175 kg
Water-reducing admixture	1.050 kg
Slump	25 to 75 mm
Air content	5% to 8%

Estimated concrete mass density using SSD aggregate is

$$140\ kg/m^3 + 350\ kg/m^3 + (1{,}072\ kg/m^3 \times 1.005) + (694\ kg/m^3 \times 1.007) + 0.175\ kg/m^3 + 1.050\ kg/m^3 = 2{,}267\ kg/m^3$$

Note that

$(0.5\%\ \text{absorption}/100) + 1 = 1.005$; and
$(0.7\%\ \text{absorption}/100) + 1 = 1.007$.

Trial Batch

At this stage, the estimated batch quantities can be checked by means of trial batches or by full-sized field batches. Enough concrete must be mixed for appropriate air and slump tests and for the three cylinders required for compressive-strength tests at 28 days, plus flexural tests if necessary. For a laboratory trial batch it is convenient to scale down the quantities to produce 0.1 m^3 of concrete.

Pozzolans and Slag Cement

Pozzolans and slag cement are sometimes added in addition to or as a partial replacement of cement to aid in workability and prevention of excessive expansion due to alkali–silica reaction. Pozzolans and slag are usually entered in the determination of the cementitious material content, using a particular dosage, such as 20% of the cementitious material. Volumes and masses are determined accordingly. If a pozzolan or slag is considered an addition to the cementitious material, it could also have been entered in the first volume calculation used in determining fine aggregate content.

REVIEW

In practice, specific procedures used in selection of concrete mixture proportions will be governed by the limits of data available on the properties of materials, the degree of control exercised over the production of concrete at the plant, and the amount of supervision at the job site. It should not be expected that field results will be an exact duplicate of laboratory trial batches. An adjustment of the selected trial mixture is usually necessary on the job.

Construction Practices

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AGGREGATE STOCKPILE

Stockpile management is the coordination of the aggregate delivery, storage, and loading into the mixing plant, which is a vital aspect of consistent, quality concrete production [American Concrete Pavement Association (ACPA), 2004a]. Locating the stockpiles is an important first consideration. A relatively flat area is preferred to facilitate unloading and stockpiling the aggregates. Placement of a pad or aggregate separation layer in the stockpile area will minimize contamination of the aggregate from the soil below as well as prevent material loss.

The goal with aggregate stockpiles is to maintain uniform gradation and moisture content and prevent aggregate contamination throughout the project. Consistent aggregate will result in the production of consistent concrete.

A few basic stockpiling practices include the following:

- Pile the material in lifts.
- Complete each lift before beginning the next.
- Do not dump material over the edges of a stockpile.
- Minimize free-fall heights of aggregates to avoid segregation.
- Only stockpile as much material as is practical.
- Minimize crushing of the aggregate by the loader.

Manage the stockpile carefully to obtain close to saturated surface dry condition. For example, thoroughly wet the aggregate then let it stand an hour before batching. Monitor the moisture content of the aggregate using probes in the stockpile. In some cases, the aggregates may be contaminated with clay or soil before arriving on the plant site. Dirty aggregates require washing or cleaning or should be rejected. In addition to causing clay ball problems in the concrete, dirty aggregates can lead to problems such as low strength. The loader operator has a key role in preventing clay or mud from being deposited into the plant's feed hoppers. The operator must control the elevation of the loader blade to prevent picking up contamination from below the aggregate stockpile.

Portable central-mix plants are usually more susceptible to producing concrete contaminated with clay balls, simply because they are temporarily placed near the project site and may have clay or loose soil underneath the stockpiles. The batch plant or concrete foreman must keep a close eye on stockpile management at portable plant sites. Stationary ready-mix plants often have a paved surface or bunkers on which the stockpiles are placed or stored and where the loader operates. This reduces the likelihood of clay being introduced into the ready-mixed concrete.

The aggregate loader operator is an important person in the production of consistent quality concrete. The primary functions of the loader operator include the following:

- Working the stockpile to provide uniform water content and gradation, while avoiding segregation;
- Minimizing contamination;
- Observing and reporting moisture variations;
- Adding material to the feed hopper appropriately; and
- Notifying the plant foreman of anticipated aggregate shortages.

BATCHING

During batching, accurate measurement of the quantity of each individual constituent of the concrete mixture must be made to ensure conformance to the selected mixture proportions. Certain features have been found to be essential elements in the design and operation of concrete production plants. Information on these elements and standards on design of systems and equipment as well as operational procedures can be found in American Concrete Institute (ACI) 304R. Additional guidance can be found in the Concrete Plant Manufacturing Bureau (CPMB) Publication No. 102, the National Ready-Mixed Concrete Association (NRMCA) Publication No. 159, and the NRMCA *Quality Control Manual*. Section 3 of the NRMCA *Quality Control Manual* provides a plant certification checklist that can be used to inspect concrete production facilities.

Facilities for storage and handling of materials should be designed to maintain the integrity and character of the individual materials. Storage facilities for the various cementitious materials should prevent the commingling of the materials and prevent confusion about the location of the different materials. Aggregate storage and handling should be accomplished in a manner to minimize segregation and contamination.

Devices that measure by mass or volume should be checked for accuracy on a frequent and regular basis to make sure they are functioning properly. Equipment should be periodically calibrated and inspected to assure that materials are properly discharged into the mixer.

The moisture content of the aggregates being batched must be determined accurately. Determining the free water carried by the aggregates enables the operator to use the proper amount of aggregate and to make the necessary adjustments in the amount of water being batched. Appropriate adjustments in the batched water are critical to assure conformance with the specified water–cementitious material ratio (w/cm). Water used to wash mixers between loads should be discharged prior to the batching of materials for subsequent loads, or provisions must be made to accurately measure the water remaining in the mixer so that appropriate adjustments can be made in the amount of water batched for the next load.

MIXING

The materials batched to produce concrete must be thoroughly mixed to achieve dispersion of the individual constituent materials into a homogenous mixture. Mixing equipment should be regularly inspected to ensure that it is in proper operating condition. The use of inspection checklists and participation in certification programs such as those available through NRMCA, CPMB, and the Truck Mixer Manufacturers Bureau are encouraged.

The most important element in concrete mixing is the blending of the constituent materials into a homogenous mixture. The mixing cycle must be sufficiently long to produce a uniform blend of materials and to develop an adequate air-void system.

Factors that affect the performance of the mixing operation are as follows:

1. Sequence of loading materials into the mixer. This is particularly important with rotating drum mixers. Certain loading sequences can result in packing of individual constituents, particularly sand or cementitious materials, into the head of the drum. Information on the effects of different loading sequences can be found in NRMCA Publication No. 148.
2. Efficiency of the mixer in blending the materials. This can be affected by excessive buildup of hardened concrete on blades and fins as well as by excessive wear or damage to these elements. The ability of the mixer to produce a concrete mixture of uniform properties within a given mixing time can be evaluated using the procedures outlined in ASTM C94. This evaluation procedure can be used to establish mixing time necessary for a given mixer to produce a uniform product.

TRANSPORTATION

Transporting the concrete from the mixer to the site of placement should be accomplished without significantly affecting the w/cm, slump, air content, homogeneity, and temperature of the concrete. This can be accomplished using a variety of equipment, depending on the distance that must be traveled. Longer distances require the use of equipment capable of agitating the concrete to maintain homogeneity of the mixture. ASTM C94 provides limits on the time to discharge and on number of drum revolutions. These limits may be exceeded if the concrete maintains the desired properties; however, caution should be exercised in extending these limits, because excessive working may have a negative impact on long-term properties.

Different drum colors depending on climatic conditions can be used to limit the impact of transport on the concrete properties. For example, in colder climates, dark drums retain solar energy, helping offset heat loss during transport; in warmer regions, light-colored drums reflect sunlight, reducing excessive heat gain.

PLACEMENT

The selection of a placement technique at a construction site will depend on the given situation. Particular effort must be made to avoid segregation of the coarse aggregate from the mortar fraction of the concrete. High temperatures during placement must be controlled in producing durable concrete (ACI 305R). Guidance on proper placement techniques is given in ACI 304R, 304.1R, 304.2R, 304.4R, and 304.5R.

ACI 304.2R covers placement of concrete by pumping. Mixtures that are to be pumped should be proportioned to have appropriate characteristics. In some cases, pumping of concrete mixtures has been found to affect the air–void system of the concrete. Pumping configurations where the concrete drops vertically some distance has been found to result in a coarsening of the air–void system with an adverse effect on the frost resistance of the concrete. Sampling of concrete for conformance to specifications should be obtained after discharge from pumping lines.

CONSOLIDATION

Concrete that has not been adequately consolidated will have excessive entrapped-void content. The presence of such voids results in lower strengths, both compressive and flexural; poor bond to reinforcement or dowels, adversely affecting load transfer; and an increase in the transport rate of fluid through the concrete (Whiting and Tayabji, 1987). The effort needed to adequately consolidate the concrete is dependent on its workability at the time of placement. Concrete that has stiffened excessively will be difficult or impossible to properly consolidate and finish and consequently will adversely affect durability.

Concrete workability is affected by the grading and proportioning of the constituent materials. To assure good concrete characteristics, these factors should be considered during the materials selection phase of the work, along with the anticipated placement and consolidation techniques. Excessive vibration of concrete should be avoided, because this may result in segregation of poorly proportioned (oversanded) mixtures and may have an adverse effect by reducing the air content of concrete intended for resistance to freezing and thawing. However, laboratory research indicates that proper consolidation by internal vibration does not adversely affect the spacing factor of air-entrained concrete (Simon et al., 1992). Guidance on the proper use of different consolidation techniques can be found in ACI 309R.

FINISHING

The objectives of the finishing operations are to produce the desired surface on the concrete with as little manipulation as possible. The specific steps involved will depend on the given situation. Overworking of the concrete should be avoided since this tends to bring excessive fines to the surface, making it prone to cracking. Overworking may also result in a reduction in the air content in the surface layer, making it susceptible to freezing and thawing or deicer damage. The concrete should not be troweled while water is present on the surface, nor should water be applied to aid finishing, as these will increase the w/cm of the surface layer and weaken it. Texturing operations such as tinning should be completed while the surface of the concrete is still plastic enough to take the texturing without disturbing the underlying mass. Saw cutting of grooves or joints should be delayed long enough for the concrete to gain sufficient strength to resist raveling of coarse aggregate. Further guidance on finishing operations can be found in ACI 304R.

CURING

Curing operations should be designed to maintain appropriate temperature and moisture conditions in the concrete, in order to facilitate the early hydration reactions of the cementitious materials. For durable concrete, it is important to prevent the development of excessive volumetric stresses resulting from thermal or drying conditions, both of which can lead to cracking.

Depending on the geometry of the element and the climatic conditions, various options can be exercised to maintain the appropriate temperature, including the use of curing blankets, appropriately colored covers, and pigmented curing compounds. For mass concrete, where heat of hydration of cementitious materials may result in unacceptable thermal stresses, appropriate action during materials selection is necessary.

Premature drying of the concrete surface must be prevented to assure durability. Precautions should be taken to prevent excessive evaporation during placement and finishing operations, and the application of curing materials should be completed as soon as possible. Concretes that contain pozzolans such as silica fume, which have very low w/cm, benefit from curing procedures that maintain a maximum of the mixing water in the system because the hydrating system can use all the original mixing water and thus minimize self-desiccation.

In steam-curing operations, care should be exercised to ensure that the temperature rise and fall of the concrete is gradual and that ambient temperature is limited to a maximum of 65°C. Recent reports indicate that certain hydraulic cements experience volume instability if exposed to moist conditions following high-temperature curing. The critical concrete temperature for this type of deterioration seems to be around 70°C, and the susceptible cements are hydraulic cements with high fineness, high alkali content, and high SO_3/Al_2O_3. The duration and time of the temperature exposure seem to be important. This phenomenon has been referred to as delayed ettringite formation and has been associated with significant expansion and cracking of the concrete. Loss of bond and loss of strength can lead to concrete destruction. An extensive review of this subject can be found in a study conducted by Day (1992).

Specifications

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Specifications are perhaps the most important means of communicating in a construction environment. Along with the contract drawings, specifications are part of the contract document; these documents instruct the contractor on his course of actions. There are two basic types of specifications commonly used today in highway construction:

- Materials and methods (M&M) specifications and
- Quality assurance (QA) specifications.

Performance-related specifications (PRS)—a third type of specification—are being developed. Several highway agencies, such as Virginia, the Port Authority of New York and New Jersey, Minnesota, Illinois, and Ontario, have had experience with projects in which PRS has been used.

M&M specifications, also called methods specifications or recipe specifications, describe exactly how the contractor must do the work: what steps to follow, what equipment to use, what materials to use and in what proportions (TRB, 1996). Although prevalent in highway construction until the 1970s, these types of specifications present some disadvantages. One major disadvantage is they prevent contractors from improving processes, exercising flexibility, and using innovation to meet the needs of the changing environment and materials resources. They also have been blamed for apparent confusion and inconsistencies in the handling of nonconforming material. This can occur when the contractor has been faithfully following M&M instructions; then, with the construction completed, one or more acceptance test results (e.g., 28-day compressive strength) fail to meet requirements. Assuming the agency has given even tacit approval for the contractor to proceed during the construction, it is now placed in a position where it has little recourse but to accept the work.

Under QA specifications, the contractor is responsible for quality control (QC) (i.e., process control), and the agency is responsible for QA, evaluating the acceptability of the product (TRB, 1996). Whereas M&M specifications describe the methods that should yield acceptable-quality construction, QA specifications directly describe the quality level the agency desires. Typically, QA specifications are statistically based specifications that recognize materials and construction variability and use random sampling and lot-by-lot testing to make acceptance decisions. They also contain pay adjustment schedules for use when the desired quality level has not been met or has been exceeded.

While the above distinctions between M&M and QA specifications have commonly been accepted and might seem to be clear, a word needs to be said about the actual practice of classifying agency specifications. In classifying specifications, one must bear in mind that a set of specifications contains numerous individual specifications or requirements. Many of today's so-called QA specifications are actually a combination of M&M and QA requirements. While the desired quality level is described in statistical terms for certain quality characteristics (e.g., a performance requirement on strength), the contractor is to various degrees also given instructions regarding procedures, equipment, and component materials (e.g., M&M requirement on curing).

Generally speaking, in writing specifications, there are at least three ways to tell the contractor what to do:

- Specify the procedures, equipment, and materials that can be assumed to result in the desired quality level (i.e., M&M specifications);
- Specify the desired quality level (i.e., QA specifications); and
- Specify the desired performance or serviceability level (i.e., performance specifications).

With QA specifications, the state-of-the-art is such that the first two are required. With recently proposed performance related specifications the state-of-the-art is such that only the last two are required.

The trend in highway construction specifications has been twofold: replace M&M specifications with QA specifications and continue improving QA specifications (Scott, 1977). Most current QA specifications are based on engineering intuition to define the important quality characteristics that correlate with performance and to establish pay adjustment schedules for each of these quality characteristics. In these specifications, each characteristic is weighed according to its perceived importance in providing the needed service. Ideally, performance specifications should be based on sound and strong relationships between material properties and performance. Thus, the approach toward improving QA specifications has been one of developing QA specifications that not only assure the desired level of construction quality, but also, insofar as possible, lead to the desired level of performance. This approach has resulted in PRS.

PRSs are improved QA specifications that can predict long-term performance from acceptance test results on key materials and construction quality characteristics. In PRS, acceptance testing is emphasized. PRSs are based on quantifiable mathematical models that allow the comparison of actual (as-constructed) properties to target (as-designed) properties and result in predictions of life-cycle costs. They thus provide the basis for rational acceptance and pay adjustment decisions or both (FHWA, 1997). Some of the advantages of PRS are (*a*) it allows the contractor and producer to optimize concrete mixtures thus leading to cost savings and more sustainable mixtures; (*b*) it provides incentives for the contractor to be more knowledgeable about their materials in order to meet a certain performance level as opposed to merely responding to an M&M specification; and (*c*) it provides incentives for producers and contractors to invest in quality control and technological developments.

An early PRS prototype, developed for concrete paving construction, includes QA requirements to specify the desired level of quality with respect to the following quality characteristics: strength, thickness, air content, and rideability (initial smoothness) (Darter et al., 1996). Under the contractor's control, these quality characteristics influence such measures of performance as fatigue cracking, joint faulting, joint spalling, and pavement serviceability rating. Several other quality characteristics are also under the contractor's control (e.g., concrete consolidation level); thus, they influence additional measures of performance (as well as some of those above). Such quality characteristics can be added as performance-related acceptance quality characteristics if they are amenable to acceptance testing during or immediately after construction and if models are available or can be developed to show the influence of the quality characteristic on performance (FHWA, 1997).

It is important that the specifications address all aspects of performance. Those aspects of performance that cannot be addressed through QA can be addressed through M&M, through producer certification of materials, through product warranties, or through some other such means. Curing is a prime example for this.

Concrete durability is an example of a key aspect (measure) of performance that must be addressed in all specifications. Under QA specifications, strength is specified more for load-carrying capacity (i.e., structural performance) than for durability. Also, it cannot be expected that durability will be achieved if simply the proper amount of air is entrained. Just as there are many aspects of performance, there are many aspects of durability; much more than strength and air content requirements typically appear in the specifications. The quality of aggregate, cement, and type and dosage of supplementary cementitious materials play a very important part, as does the mixture proportioning (particularly water–cementitious material ratio). The specifications should restrict the use of unsound or reactive aggregates and types of cements that could lead to the premature deterioration of concrete due to environmental exposure. PRS provides incentives to optimize mixture proportions.

Durable concretes must resist freezing and thawing when they are saturated, but they must also have low permeability when exposed to harmful solutions such as chlorides. The effect of entrained air voids on permeability is minimal since these voids are isolated. However, air voids resulting from poor consolidation or extra water would affect adversely the permeability of concretes. Permeability is considered important; by measuring electrical conductance in coulombs, some agencies have established limits on concretes in order to indirectly control permeability (Ozyildirim, 1998). The coulomb value is obtained using the AASHTO T277 or ASTM C1202 rapid chloride permeability test (RCPT). In this test, the charge (in coulombs) passed through a saturated 50-mm thick and 100-mm-diameter specimen subjected to 60 Vdc in a 6-h period is determined. Virginia and some other states have used the RCPT to calculate pay factors.

The thermal and shrinkage properties of concretes are also important, since volume changes cause cracking that facilitates the intrusion of water or other potentially harmful solutions into concrete. While such factors as the quality of aggregates and the amount of water in the mixture strongly influence shrinkage and drying properties, specifications should call for tests on concrete specimens to determine whether the potential volumetric changes are within acceptable limits established to ensure longevity.

In addition to materials properties, another important area with respect to performance that requires attention in the specifications is the construction practices. Pavement smoothness, slab thickness, consolidation, cover depth over reinforcing steel, dowel bar alignment, timing of joint sawing and depth of sawcut, curing effectiveness, and skid resistance are some of the construction parameters that affect the longevity of concrete structures and pavements. Most of these parameters can be (some already are) used in PRS as acceptance quality characteristics that enable pay adjustments that better reflect the expected performance.

In summary, the PRS approach, when properly complemented by M&M and other traditional specification requirements, can lead to significant benefits. In the long term, contractors and producers will benefit since they will have a better understanding of their product and more flexibility in making it. They will know when to place more importance on certain elements of quality control so as to increase the likelihood of achieving good performance, and they will have proper incentive to achieve performance, through a fair and rational pay-adjustment system that rewards high-quality work. Agencies should also benefit in the long term because they will be able to specify that quality level which results in lowest life-cycle costs. With both contractors and agencies having the common goal of minimizing life-cycle costs, concrete construction should be more cost-effective.

Testing

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Quality control and acceptance testing are indispensable parts of the construction process. Tests of concrete to evaluate the performance of available materials, to establish mixture proportions, and to control concrete quality in the field include slump, air content, density, and strength. Slump, air content, and strength tests are usually required in project specifications for concrete quality control, whereas density is used more in mixture proportioning. Some special testing due to the environmental conditions may be required.

Following is a discussion of testing frequency and descriptions of the major control tests to ensure uniformity of materials, desired properties of freshly mixed concrete, and required strength of hardened concrete. Special tests are also described.

TESTING FREQUENCY

The frequency of testing aggregates and concrete for typical batch-plant procedures depends largely upon the uniformity of materials. Initially it is advisable to conduct tests several times a day, but as work progresses the frequency often can be reduced.

Usually, aggregate moisture tests are conducted once or twice a day. The first batch of fine aggregate in the morning is often overly wet since free moisture will migrate overnight to the bottom of the storage bin. As fine aggregate is drawn from the bottom, the moisture content should stabilize at a lower level and the first moisture test can be made. After a few tests, changes in moisture content can be judged to a fairly accurate degree by sight and feel. Subsequent tests are usually necessary only when a change is readily apparent.

Slump tests should be performed for the first batch of concrete each day, as well as whenever consistency of concrete appears to vary and whenever cylinders are made at the job site.

Air content tests should be performed often enough at the point of delivery to ensure proper air content, particularly if temperature and aggregate gradation change. An air content test is desirable for each sample of concrete from which cylinders are made; a record of the temperature of each sample of concrete should also be kept.

The number of strength tests performed will depend on the job specifications and the occurrence of variations. Strength tests of each class of concrete placed each day should be taken not less than once a day. The average strength of two cylinders is required for each test. Additional specimens may be required when high-strength concrete is involved or where structural requirements are critical. The specimens should be laboratory cured. Specifications may require that additional specimens be made and field cured, as nearly as practical in the same manner as the concrete in the structure. Two 7-day test cylinders, along with two 28-day test cylinders, are often made and tested to provide an early indication of strength development. As a rule of thumb, the 7-day strength is about 75% to 80% of the 28-day strength, depending upon the type and amount of cement, water–cementitious material ratio, curing temperature, and other variables.

TESTING AGGREGATES

Sampling Aggregates

Methods for obtaining representative samples of aggregates are given in AASHTO T2 (ASTM D75). Accurate sampling is important. Reducing large field samples to small quantities for individual tests must be done with care so that the final samples will be properly representative. For coarse aggregate, this is usually done by the quartering method. The sample, thoroughly mixed, is spread on a piece of canvas in an even layer 75 to 100 mm thick. It is divided into four equal parts. Two opposite parts are then discarded, and the remaining material is remixed. This process is repeated until the desired size of sample remains. A similar procedure is sometimes used for moist fine aggregate. Sample splitters (mechanical devices for dividing a sample into smaller sizes) are desirable for dry fine aggregate.

Organic Impurities

Organic impurities in fine aggregate are determined in accordance with AASHTO T21 (ASTM C40). A sample of fine aggregate is placed in a sodium hydroxide solution and shaken. The following day the color of the solution is compared with a standard color solution. If the color is darker than the standard, the fine aggregate should not be used for important work without further investigation. Some fine aggregates contain small quantities of coal or lignite that give the liquid a dark color. In such cases, mortar strength tests [AASHTO T71 (ASTM C87)] using the fine aggregate in question will indicate the effects of the impurities present. Note that appreciable quantities of coal or lignite in aggregates can cause popouts and staining of the concrete and can reduce durability when concrete is exposed to weathering. The quantity of organic impurities may be insufficient to reduce the strength of the concrete appreciably and the fine aggregate may be acceptable. Local experience is often the best indication of the durability of concrete made with such aggregates.

Objectionable Fine Material

Large amounts of clay and silt in aggregates can adversely affect durability, increase water requirements, and increase shrinkage. Specifications usually limit the amount of material passing the 75-µm (No. 200) sieve to 2% or 3% in fine aggregate and to 1% or less in coarse aggregate. Guidance for testing for material finer than that which passes through the 75-µm sieve is provided by AASHTO T11 (ASTM C117). AASHTO T112 (ASTM C142) provides guidance for testing for clay lumps.

Grading

Grading of aggregates significantly affects concrete mixture proportioning and workability. Hence, grading tests are an important element in the evaluation of concrete quality. The grading of an aggregate is determined by an analysis in which the particles are sorted into their various sizes by standard sieves. Guidance for conducting the analysis is provided by AASHTO T27 (ASTM C136).

Results of sieve analyses are used in three ways: (*a*) to determine whether or not the materials meet specifications; (*b*) to select the most suitable material if several aggregates are available; and (*c*) to detect variations in grading that are sufficient to warrant blending selected sizes or an adjustment of concrete mixture proportions.

The grading requirements for concrete aggregate are shown in ASTM C33. Materials containing too much or too little of any one size should be avoided. Some specifications require that mixture proportions be adjusted if the average fineness modulus of fine aggregate changes by more than 0.20. Other specifications require an adjustment in mixture proportions if the amount retained on any two consecutive sieves changes by more than 10% by mass of the total fine aggregate sample. A small quantity of clean particles that pass a 150-µm (No. 100) sieve but are retained by a 75-µm (No. 200) sieve is desirable for workability. For this reason most specifications permit up to 10% of this material in fine aggregate.

Moisture Content of Aggregates

Several methods can be used for determining the amount of moisture in aggregate samples. The total moisture content for fine or coarse aggregate can be tested in accordance with AASHTO T255 (ASTM C566). In this method a sample of known mass of damp aggregate is dried either in an oven, on a hot plate, or in a microwave oven. From the values of mass before and after drying, the total and surface (free) moisture contents can be calculated. The total moisture content can be calculated as follows:

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

where

P = moisture content of sample, %;
W = mass of original sample, g; and
D = mass of dried sample, g.

The surface moisture content is equal to the total moisture content minus the absorption. AASHTO T85 (ASTM C127) and AASHTO T84 (ASTM C128) are test methods for determining the moisture contents of coarse and fine aggregate, respectively. Only the surface moisture, not the absorbed moisture, becomes part of the mixing water in concrete.

ASTM C70 provides the method for determining the surface (free) moisture in fine aggregate. The same procedure can be used for coarse aggregate with appropriate changes in the size of sample and dimensions of the container. This test depends on displacement of water by a known mass of moist aggregate; therefore, the relative density (specific gravity) of the aggregate must be known accurately.

Electric moisture meters are used in many concrete batching plants to check the moisture content of fine aggregates. They operate on the principle that the electrical resistance of damp fine aggregate decreases as moisture content increases, within the range of dampness normally encountered. The meters measure the electrical resistance of the fine aggregate between electrodes protruding into the batch hopper or bin. Such meters require periodic calibration and must be maintained properly. They measure moisture content accurately and rapidly, but only at the level of the electrodes.

TESTING FRESHLY MIXED CONCRETE

Sampling Freshly Mixed Concrete

Testing should be conducted by qualified personnel. The importance of obtaining properly representative samples of freshly mixed concrete for control tests must be emphasized. Unless the sample is representative, test results will be misleading. Guidance on obtaining and handling samples can be found in AASHTO T141 (ASTM C172). The sample should be at least 28 L, used within 15 min of the time it was taken, and protected from sunlight, wind, and other sources of rapid evaporation during this period. The sample should not be taken from the first or last 10% of the batch discharge.

Consistency

The slump test, AASHTO T119 (ASTM C143), is the most generally accepted method used to measure the consistency of concrete. The test equipment consists of a slump cone (a metal conical mold 305 mm high, with an 203-mm diameter base and 102-mm diameter top) and a steel rod (16 mm in diameter, 600 mm long) with a hemispherically shaped tip. The dampened slump cone, placed upright on a flat, solid surface, should be filled in three layers of approximately equal volume. Therefore, the cone should be filled to a depth of about 65 mm (after rodding) for the first layer, to about 150 mm for the second layer, and until overfilled for the third layer. Each layer is rodded 25 times. Following rodding, the last layer is struck off and the cone is slowly and vertically removed as the concrete subsides or settles to a new height. The empty slump cone is then placed next to the settled concrete. The slump is the vertical distance the concrete settles, measured to the nearest 5 mm from the top of the slump cone (mold) to the displaced original center of the subsided concrete.

A high slump value is indicative of a wet or fluid concrete. The slump test should be started within 5 min after the sample has been obtained, and the test should be completed in less than 2½ min, as concrete loses slump with time.

Temperature Measurement

Because of the important influence concrete temperature has on the properties of freshly mixed and hardened concrete, many specifications place limits on the temperature of fresh concrete. Glass or armored thermometers are available. The thermometer should be accurate to ±0.5°C, and should remain in a representative sample of concrete for a minimum of 2 min or until the reading stabilizes. A minimum of 75 mm of concrete should surround the sensing portion of the thermometer. Electronic temperature meters using thermocouples with digital readouts are also available. Complete the temperature measurement (ASTM C1064) within 5 min after obtaining the sample.

Mass Density and Yield

AASHTO T121 (ASTM C138) are used to determine the mass density and yield of freshly mixed concrete. The results can be sufficiently accurate to determine the quantity of concrete produced per batch. The test also can give indications of air content provided the densities of the ingredients are known. A balance or scale sensitive to 0.3% of the test load is required. The size of the container varies with the size of aggregate. Care is needed to consolidate the concrete adequately and strike off the surface so that the container is filled properly. The container should be calibrated periodically. The mass density is expressed in kilograms per cubic meter and the yield (volume of the batch) is expressed in cubic meters.

A nuclear method (ASTM C1040) can also be used to determine the density of unhardened concrete.

Air Content

A number of methods for measuring air content of freshly mixed concrete can be used. AASHTO standards include the pressure method [T152 (ASTM C231)], the volumetric method [T196 (ASTM C173)], and the gravimetric method [T121 (ASTM C138)]. Variations of the first two methods can also be used. Other innovative methods are being developed. One such method is the air void analyzer, which is a portable device that measures the entrained air void structure of fresh concrete in about 30 min. However, this process is not yet standardized and so is not discussed here.

The pressure method is based on Boyle's law, which relates pressure to volume. Many commercial air meters of this type are calibrated to read air content directly when a predetermined pressure is applied. The applied pressure compresses the air within the concrete sample, including the air in the pores of aggregates. For this reason, tests by this method are not suitable for determining the air content of concretes made with some lightweight aggregates or other very porous materials unless they have been vacuum-saturated. Correction factors for normal-weight aggregates are relatively constant and, though small, should be applied to obtain the correct amount of entrained air. Some meters use change in pressure of a known volume of air and are not affected by changes in elevation. Pressure meters are widely used because the mixture proportions and densities of the materials need not be known. Also, a test can be conducted in less time than is required for other methods.

The volumetric method requires removal of air from a known volume of concrete by agitating the concrete in an excess of water. This method can be used for concrete containing any type of aggregate, including lightweight or porous materials. The test is not affected by atmospheric pressure, and densities of the materials need not be known. Care must be taken to sufficiently agitate the sample to remove all air.

The gravimetric method uses the same test equipment as that for mass density of concrete. The measured mass density of concrete is subtracted from the theoretical mass density as determined from the absolute volumes of the ingredients, assuming no air is present. This difference, expressed as a percentage of the theoretical mass density, is the air content. Mixture proportions and densities of the ingredients must be accurately known, otherwise results may be in error. Consequently, this method is suitable only where laboratory-type control is exercised. Significant changes in density can be a convenient way to detect variability in air content.

With any of the above methods, air content tests should be started within 5 min after the sample has been obtained.

Strength Specimens

Guidance for producing and curing specimens for strength tests can be found in AASHTO T23 (ASTM C31) (field specimens) or AASHTO T126 (ASTM C192) (laboratory specimens). Molding of strength specimens should be started within 15 min after the sample is obtained.

The standard test specimen for compressive strength of concrete with a nominal maximum aggregate size of 50 mm or smaller is a cylinder 152 mm in diameter by 305 mm in height. While rigid metal molds are preferred, plastic or other types of disposable molds conforming to AASHTO M205 (ASTM C470) can be used. They should be placed on a smooth, level surface and filled carefully to avoid distortion of their shape.

It is also common for 100-mm diameter by 200-mm high cylinder molds to be used with concrete containing up to 25-mm nominal maximum size aggregate. This smaller cylinder is easier to cast, requires less sample, weighs considerably less than a 152-×-305-mm concrete cylinder, and is therefore easier to handle and requires less moist-curing storage space.

Beams for the flexural strength tests should be 152 × 152 mm in cross section for aggregates up to 50 mm in nominal maximum size. The length of beams should be at least three times the depth of the beam plus 50 mm.

Test cylinders to be rodded should be filled in three approximately equal layers with each layer rodded 25 times for 152-mm-diameter cylinders; beam specimens up to 200 mm deep should be filled in two equal layers with each layer rodded once with a 16-mm rod for each 14 cm^2 of the specimen's top surface area. If the rodding leaves holes, the sides of the mold should be lightly tapped with a mallet or open hand. Concrete with a slump in excess of 75 mm should be rodded; concrete with a slump less than 25 mm should be vibrated; 25- to 75-mm slump concrete can be rodded or vibrated. Immediately after casting, the tops of the specimens should be (*a*) covered with an oiled glass or steel plate, (*b*) sealed with a plastic bag, or (*c*) sealed with a plastic cap.

The strength of a test specimen can be greatly affected by jostling, changes in temperature, and exposure to drying, particularly within the first 24 h after casting. Thus, test specimens should be cast in locations where subsequent movement is unnecessary and where protection is possible. Cylinders and test beams should be protected from rough handling at all ages.

Standard testing procedures require that specimens be cured under controlled conditions, either in the laboratory or in the field. Controlled laboratory curing in a moist room or in limewater provides a standard curing condition, allowing comparison between tests. Specimens cured in the field in the same manner as the structure they represent may give a more accurate indication of the actual strength of concrete in the structure at the time of testing, but they give little indication of whether a deficiency is due to the quality of the concrete as delivered or to improper handling and curing. On some jobs, field-cured specimens are made in addition to those given controlled laboratory curing, especially when the weather is unfavorable, to determine when forms can be removed or when the structure can be put into use.

The above tests are commonly performed on all concretes. The following tests are performed on fresh concrete for special conditions or to provide additional evaluation and tighter quality control.

Time of Setting

AASHTO T197 (ASTM C403) outline the method of determining the time of set or rate of hardening.

Maturity Testing

In-place concrete strength development can also be evaluated by maturity testing [American Concrete Institute (ACI) 306R (Section 6.4) and ASTM C1074]. This procedure is conducted to determine times for joint sawing, opening to traffic, form removal, and prestress release. It may also be used for in situ strength evaluation for structural loading.

Accelerated Curing Tests

Accelerated strength tests can be used to expedite quality control of concrete in the production process and for the acceptance of structural concrete where adequate data correlated with the standard 28-day compressive strength test are available. Warm water (35 ± 3°C), boiling water, and autogenous accelerated curing methods used for such purposes are described in ASTM C684.

Cement, Water Content, and Water–Cement Ratio

Test methods are available for determining the cement and water content of freshly mixed concrete. These test results can assist in an estimate of strength and durability potential prior to the setting and hardening of the concrete and can affirm that the desired cement and water contents have been obtained. ASTM test methods C1078 and C1079, based on the Kelly–Vail method, determine cement content and water content, respectively.

Supplementary Cementitious Materials Content

Standard test methods are not available for determining the supplementary cementitious materials (SCMs) content of unhardened concrete. However, the presence of certain SCMs such as fly ash can be determined by washing a sample of the concrete's mortar over a 45-µm (No. 325) sieve and viewing the residue retained with an optical microscope at magnification of about 200 times. Fly ash particles, for example, would appear as spheres of various colors. Sieving the mortar through the 150-µm or 75-µm sieve is helpful in removing sand grains.

Bleeding of Concrete

The bleeding properties of fresh concrete can be determined by either of two methods described in AASHTO T158 (ASTM C232). One method consolidates the specimen by tamping without further disturbance; the other method consolidates the specimen by vibration, after which the specimen is vibrated intermittently throughout the test. The amount of bleed water at the surface is expressed as the volume of bleed water per unit area of exposed concrete, or as a percentage of the net mixing water in the test specimen. The bleeding test is rarely used in the field.

TESTING HARDENED CONCRETE

Molded specimens [AASHTO T23 (ASTM C31), AASHTO T126 (ASTM C192), or ASTM C873] or hardened concrete samples obtained from construction [AASHTO T24 (ASTM C42), or ASTM C823] can be used in tests on hardened concrete. Separate specimens should be obtained for different tests as specimen preconditioning for certain tests can make the specimen unusable for other tests. Of the following tests, only the strength tests are commonly used for quality control of concrete. The other tests are used to verify certain properties before or after construction.

Strength Tests of Hardened Concrete

Strength tests of hardened concrete can be performed on (*a*) cured specimens molded from samples of freshly mixed concrete [AASHTO T23 or T126 (ASTM C31 or C192)]; (*b*) specimens cored or sawed from the hardened concrete in accordance with AASHTO T24 (ASTM C42); or (*c*) specimens made from cast-in-place cylinder molds (ASTM C873). Cast-in-place cylinders can be used in concrete that is 125 to 300 mm in depth. For all methods, cylindrical samples should have a diameter at least three times the maximum size of the coarse aggregate in the concrete and a length as close to twice the diameter as possible.

Cores should not be taken until the concrete can be sampled without disturbing the bond between the mortar and the coarse aggregate. For horizontal surfaces, cores should be taken vertically, and not near formed joints or edges. For vertical or sloped faces, cores should be taken perpendicular to the central portion of the concrete element. Coring through reinforcing steel should be avoided when possible. A pachometer (electromagnetic device) can be used to locate steel. Cores taken from structures that are normally wet or moist in service should be moist conditioned and tested moist, as described in AASHTO T24 (ASTM C42). Those from structures normally dry in service should be conditioned in an atmosphere approximating their service conditions and tested dry.

Test results are greatly influenced by the condition of the specimen. Grind or cap the ends of cylinders and cores for compression testing in accordance with the requirements of AASHTO T231 (ASTM C617). Various commercially available materials can be used to cap compressive test specimens. Sulfur materials can be used if the caps are allowed to harden at least 2 h before the specimens are tested. Caps should be made as thin as is practical. Reusable unbonded caps (neoprene pads) may be used in accordance with ASTM C1231.

Conduct the testing of specimens in accordance with (*a*) AASHTO T32 (ASTM C39) for compressive strength, (*b*) AASHTO T97 (ASTM C78) for flexural strength using third-point loading, (*c*) AASHTO T177 (ASTM C293) for flexural strength using center point loading, and (*d*) ASTM C496 for splitting tensile strength.

For both pavement thickness design and pavement mixture proportioning, the modulus of rupture (flexural strength) should be determined by the third-point loading test. However, compressive strength or modulus of rupture by center point loading [AASHTO T177 (ASTM C293)] or cantilever loading can be used for job control if preferred by the project engineer.

The amount of variation in compressive-strength testing is far less than for flexural-strength testing. To avoid the extreme care needed in field flexural-strength testing to offset this disadvantage, compressive-strength tests should be used to monitor concrete quality after a laboratory-determined empirical relationship has been developed between the compressive and flexural strength of the concrete used.

The moisture content of the specimen has considerable effect on the resultant strength. A saturated specimen will show lower compressive strength and higher flexural strength than those for companion specimens tested dry. This is important to consider when cores taken from hardened concrete in service are compared with molded specimens tested as they are taken from the moist-curing room.

Air Content

ASTM C457 outlines a method for determining the air–void system parameters of hardened concrete, including air content. This test is performed to assure that the air–void system is appropriate for a particular environment. The test is also used to determine the effects different admixtures and methods of consolidation and placement have on the air–void system. The test can be performed on premolded specimens or samples removed from the structure. Using a ground section of a concrete sample, the air–void system is viewed through a microscope. The information obtained from this test may include the volume of entrained air, the sample's specific surface, and the spacing factor. Automated systems that are capable of measuring the parameters stated above are available. Use of automated process helps faster data acquisition. However, as with any automated system, limitations should be considered, if any, before implementation.

Density, Relative Density, Absorption, and Voids

The procedures outlined in ASTM C642 can be used to determine the density, relative density, absorption, and voids content of hardened concrete. The boiling procedure of the method can render the specimens useless for certain additional tests, especially strength tests. The density can be obtained by multiplying the relative density by the mass density of water 1,000 kg/m^3.

SSD density is often required for specimens to be used in other tests. In this case, the density can be determined by soaking the specimen in water for 48 h and then determining its mass in air (when SSD) and immersed in water. The SSD density is then calculated as follows:

$$D_{\text{SSD}} = \frac{W_1 \rho}{W_1 - W_2}$$

where

D_{SSD} = density in the SSD condition, kg/m^3;
W_1 = the SSD mass in air in kilograms;
W_2 = the mass immersed in water in kilograms; and
ρ = the density of water, 1,000 kg/m^3

The SSD density provides a close indication of the freshly mixed mass density of concrete. A nuclear method (ASTM C1040) can also be used to determine the density of unhardened concrete.

Cement Content

ASTM C1084 outlines the test method for determining the cement content of hardened concrete. Although not frequently performed, the cement content tests are valuable in determining the cause of lack of strength gain or poor durability of concrete. Aggregate content can also be determined by these tests. The user of these test methods should be aware of certain admixtures and aggregate types that can alter test results. The presence of finely divided SCMs would be reflected in the test results.

Supplementary Cementitious Materials and Chemical–Admixture Content

Petrographic techniques outlined in ASTM C856 can be used to determine the presence and amount of certain SCMs such as fly ash. A sample of the SCMs used in the concrete is usually necessary as a reference to determine the type and amount of the SCMs present. The presence and possibly the amount of chemical admixtures (such as water reducers) can be determined by infrared spectrophotometry. The presence of calcium chloride as a chemical admixture can be determined as described below.

CHLORIDE CONTENT

The chloride content of concrete and its ingredients should be checked to make sure it is below the limit necessary to avoid corrosion of reinforcing steel. Refer to ASTM C1152 for acid-soluble chloride and ASTM C1218 for water-soluble chloride test methods. ACI 318 and ACI 222R provide chloride limits.

Petrographic Examination

Petrographic examination uses microscopic and other techniques described in ASTM C856 to determine the constituents of concrete, concrete quality, and causes of inferior performance, distress, or deterioration. Estimating future performance and structural safety of concrete elements can be facilitated. Some of the items that can be revealed by a petrographic examination include paste, aggregate, SCM, and air content; frost and sulfate attack; alkali–aggregate reactivity; degree of hydration and carbonation; water–cement ratio; bleeding characteristics; fire damage; scaling; popouts; effect of admixture; and several other aspects.

Volume and Length Change

Volume- or length-change limits are sometimes specified for certain concrete applications. Volume change is also of concern when a new ingredient is added to concrete, because mix designers must make sure there are no significant adverse effects. Length change due to drying shrinkage, chemical reactivity, and forces other than intentionally applied forces and temperature changes can be determined by AASHTO T160 (ASTM C157) (water and air storage methods). Determination of early volume change of concrete before hardening can be performed using ASTM C827. Creep can be determined in accordance with ASTM C512. The static modulus of elasticity and Poisson's ratio of concrete in compression can be determined by methods of ASTM C469, and dynamic values of these parameters can be determined by ASTM C215.

Durability

Durability refers to the ability of concrete to resist deterioration from the environment or service in which it is placed. Properly proportioned concrete that is properly placed, finished, and cured should endure without significant distress throughout its service life. Various tests can be performed to meet project requirements, ensure or check durability, or determine the effects of certain ingredients or concreting procedures on durability. Resistance to freezing and thawing can be determined in accordance with AASHTO T161 (ASTM C666), ASTM C671, and ASTM C682. Deicer-scaling resistance can be determined by ASTM C672. Corrosion protection and determining corrosion activity of reinforcing steel can be tested by ASTM C876. Alkali–aggregate reactivity can be analyzed by ASTM C227 (alkali–silica reaction), C289, C441 [effectiveness of SCMs to inhibit of alkali–silica reaction (ASR)], and C586 [alkali–carbonate reaction (ACR) rock cylinder test], C1260 (rapid mortar bar) and C1567 (evaluation of materials combinations to mitigate ASR), C1293 (concrete prism), and C1105 (ACR concrete prism). Sulfate resistance can be evaluated by ASTM C452 and C1012. Abrasion resistance can be determined by ASTM C418 (sandblasting), C779 (revolving disk, dressing wheel, and ball-bearing methods), C944 (rotating cutter), and C1138 (underwater abrasion).

Permeability

Various test methods are available for determining the permeability of concrete to various substances. Both direct and indirect methods are used. Resistance to chloride–ion penetration, for example, can be determined by ponding chloride solution on a concrete surface and, at a later age, determining the chloride content of the concrete at particular depths (AASHTO T259). The rapid chloride permeability (electrical resistance) test (AASHTO T277, ASTM C1202) can be correlated with permeability and resistance to chloride–ion penetration of concrete. The test procedure cautions the possibility of interferences. Any ingredient, such as calcium nitrite, that affects the electrical conductance would affect the test result. Various absorption methods are also used. Direct water permeability data can be obtained by using U.S. Army Corps of Engineers method CRD-C163-92.

Nondestructive Test Methods

Various nondestructive tests can be used to evaluate the relative strength of hardened concrete. The most widely used are the rebound (ASTM C805), penetration (ASTM C803), pullout (ASTM C900), break-off (C1150), and dynamic or vibration (ASTM C597) tests. Each method has limitations, and caution should be exercised against acceptance of nondestructive test results as having a constant correlation to the traditional compression test; that is, empirical correlations must be developed prior to use.

Gamma radiography equipment can be used in the field to determine the location of reinforcement, density, and perhaps honeycombing in structural concrete units. ASTM C1040 procedures use gamma radiation to determine the density of unhardened and hardened concrete in place.

Battery-operated magnetic detection devices like the pachometer or covermeter are available to measure the depth of reinforcement in concrete and to detect the position of reinforcing steel. Electrical-resistivity equipment is being developed to estimate the thickness of concrete pavement slabs.

A microwave-absorption method has been developed to determine the moisture content of porous building materials such as concrete. Acoustic-emission techniques show promise for studying load levels in structures and locating the origin of cracking.

Additional information on methods for testing fresh and hardened concrete can be obtained in Kosmatka et al. (2002), as well as Lamond and Pielert (2006).

Case Studies

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The intention of this chapter is to recognize the occurrence and severity of the major problems affecting the field performance of concrete in transportation structures and emphasize the critical role of all facets of materials selection, design, construction, and maintenance in attaining durable concrete.

CASE I: MINNESOTA PAVEMENT

A study was commissioned by the Aggregate and Ready Mix Association of Minnesota (ARM) to investigate the causes of an unusually high number of exterior concrete problems observed after the winter of 1996–1997 (Snyder, 1998). The problems were generally related to the scaling and spalling of exterior flatwork (i.e., driveways, sidewalks, patios, and floors) placed during the summer of 1996. Other types of distress such as popouts were also evident but less common. Some of the problems were also observed in concrete that had been placed as much as 3 years earlier.

ARM solicited its members to submit samples of distressed concrete. These samples were then evaluated to identify the factors contributing to the field performance problems. In addition, a panel of concrete durability and quality experts was assembled to perform the examination and tests of the hardened concrete samples, to analyze the resulting data, and to develop recommendations for improving the quality of future concrete construction in Minnesota.

The study evaluated a total of 33 projects. The most predominant defects found in the concrete samples submitted were:

Scaling:	85%
Popouts:	30%
Mortar flaking:	12%

Evaluation of the core samples submitted from the projects showed the following controllable factors affecting performance:

Air entrainment problems:	(69% total)
Low air at surface only:	48%
Low air content throughout:	15%
No air entrainment:	6%
Finishing problems:	61%
Inadequate curing:	61%
Low cementitious material content (<333 kg/m^3):	55%
Long transit time (>45 min):	42%
High water–cementitious material ratio (w/cm) (>0.45):	39%
Nondurable aggregate:	30%
Early exposure to deicing chemicals:	6%
Improper joint spacing or sawing time:	3%
Cement or alkali–silica reaction problem:	3%

Finishing problems were defined as concrete that had an adequate entrained air void system, except near the surface, or with a high w/cm at the surface. It was determined that these problems were most likely caused by premature finishing or over finishing.

Two additional factors were identified that most likely contributed to the high occurrence of surface deterioration: late-season paving and severe winter weather. Five of the projects evaluated were paved in late October or early November 1996. It is unlikely that concrete placed that late in the year would have had time to develop sufficient strength and durability prior to freezing unless extraordinary curing conditions were provided.

Additionally, the month of November 1996 had more than three times the normal rainfall, and monthly average temperatures 4°C below normal. The winter of 1996–1997 had 77 cycles of freezing and thawing through 0°C and 81 cycles through –4°C. It was hypothesized that the large amount of available moisture saturated the concrete and that the extreme cold and high number of cycles of freezing and thawing, coupled with the lack of near-surface air entrainment in a large number of the projects, caused the surface scaling.

The report makes several recommendations for reduction of the problems identified, including the following actions:

- Making adjustments to concrete mixture proportions and admixture dosage rates when materials change;
- Improving materials selection to improve workability, to reduce water demand, and to reduce the need for retempering at the job site;
- Improving the selection of aggregates to reduce the amount of nondurable material present;
- Improving mixture proportioning to provide adequate durability for the materials' intended use and exposure conditions, and designing mixtures for a balance between strength, durability, workability, and finishability;
- Not allowing the final evaporation of bleed water to take place until just before the application of the curing medium;
- Minimizing haul times either through shrink mixing or on-site mixing when necessary;
- Improving curing practices to improve strength and durability; and
- Providing a training course in total quality management for all personnel involved in concrete production, placement, and finishing.

CASE II: NANTICOKE RIVER BRIDGE, MARYLAND

Maryland Department of Transportation (DOT) experienced significant cracking in the decks of the Nanticoke River Bridge and in the Route 50 Bridge over Route 331 and the DP&L Railroad (DP&LRR), both constructed in 1990 (Healy and Laurie, 1998). The initial deck placements on both bridges exhibited significant cracking over 100% of their surface. Cracking propagated in both the longitudinal and transverse directions and approximately 70% of the cracks exceeded 0.18 mm in width, which is the normal American Concrete Institute level of acceptance.

Vibration analysis of the two bridges showed that the Nanticoke River Bridge was less critical than the Route 331 and DP&LRR Bridge, but both bridges exhibited similar cracking. The similarity in cracking led to an evaluation of the bridge deck curing procedures. At the time

Maryland State Highway Administration specifications required that bridge deck slabs be cured by spraying with a liquid membrane-forming curing compound immediately after concrete finishing and then be covered with burlap, polyethylene, or cotton mats for 7 days. This procedure was used on the cracked areas of both bridges.

A series of alternate curing methods and procedures was then tried. The addition of plastic fibers was also tried, as were different placement times and temperature requirements. The intent was to develop a process that would produce a more crack-free deck, but the process was not a scientifically based study to compare different methods.

The results showed that all the procedures tried resulted in some deck cracking, but the use of plastic fibers and a combination of curing compound followed by the application of wet burlap significantly reduced the amount of cracking. Both procedures also added significant cost to the deck construction, so it was decided that a careful application of the moistened burlap curing procedure would be used on the remainder of the pours that produced results comparable to the other two concepts.

Concurrent with the above project, the State of Maryland had been investigating deck cracking in general. Inadequate curing was again deemed to be the most likely cause of cracking. As a result of this overall investigation, the standard specifications have been changed to provide for better curing conditions and to specifically require the use of wet burlap curing with continuous wetting as the method for curing bridge decks.

CASE III: BISSELL BRIDGE, CONNECTICUT

The Bissell Bridge, constructed in 1957, crossed the Connecticut River in Windsor, Connecticut, with 14 simple spans of 37 m (Schupack and Stark, 1998). The bridge was demolished in 1992–1993 to make room for a wider Interstate highway. The demolition allowed the opportunity to evaluate various performance and durability aspects of the bridge superstructure. The original durability study was conducted between December 1991 and August 1993 and was focused on web longitudinal cracking. During the study it was realized that the deck slab performance was exceptional, and samples were salvaged by the study contractor, Schupack, Suarez Engineers, Inc. (SSE), for possible future study.

The Portland Cement Association contracted with SSE and Construction Technology Laboratories in January 1994 to perform a limited study to try to explain the exceptional performance of the Bissell Bridge slab.

The bridge was constructed as a monolithic T-beam structure with no cold joints between the deck slab and the web. This was done to permit rapid casting of each span and the reuse of forms and falsework. The monolithic casting was achieved by retarding the concrete until the entire T-beam superstructure was placed and all deflections had occurred in the falsework. The entire concrete mass was then revibrated. It is believed that this construction procedure eliminated cracking due to falsework successive deflection, the lack of a cold joint between the deck and the web, and lack of concrete settlement cracks at the junction of the web and slab.

The concrete (from construction records) consisted of an eight-bag mixture (446 kg/m^3), with a water–cement ratio (w/c) of 0.35, poorly graded 38-mm maximum size coarse aggregate, and well-graded fine aggregate. The concrete gained sufficient strength in 2 to 3 days (higher than 21 MPa) to allow early posttensioning. The posttensioning introduced longitudinal compressive stresses in the slab of approximately 2 MPa that probably helped control drying shrinkage cracks. The concrete compressive strength exceeded 35 MPa at 28 days. Impact hammer readings taken prior to demolition indicated concrete compressive strength ranging from 28 to 46 MPa, with an average of 35 MPa. Two cores broken after demolition averaged 48 MPa compressive strength.

The entire bridge deck was covered by a bituminous wearing course (BWC) with a planned thickness of 50 mm. Cores showed the actual thickness varied from 38 to 102 mm. Original plans had called for a tack coat prior to overlay, but whether it had been placed could not be determined. There is no record of any performance problems associated with the bridge deck, and records indicate only localized repairs in 1976 and 1977. It is unknown if the original BWC was ever replaced in its entirety.

Prior to bridge demolition the entire BWC was removed by mechanical means. The deck was then visually inspected. Sounding of the deck revealed no delaminations, cracks, or reinforcement corrosion. Two incidents of localized deterioration were observed on a cantilever portion of the slab that supported either a catwalk or the median. These may have been caused by poor local concrete quality or chloride contamination.

The bridge deck was exposed to the normal deicing practices of Connecticut DOT. The bridge elements exposed to runoff generally had high total chloride–ion contents. The bridge slab generally had chloride contents well below the threshold at which corrosion would be expected. The chloride–ion content of the upper 19 mm of the slab provides evidence that the slab was exposed to significant levels of chloride. The concrete was sufficiently resistant to chloride diffusion that 35 years of exposure had not caused chloride–ion concentrations to reach corrosion levels. An analysis of the chloride–ion gradient between the slab and the web shows that the concrete appears to be as resistant to chloride–ion migration as is a 7% silica–fume concrete. It is unknown whether the BWC or tack coat had some special feature that prevented chloride-ion ingress into the bridge slab.

The study concludes that even though the Bissell Bridge was exposed to deicing salts, it provided 35 years of excellent performance due to the following:

- Very low permeability concrete due to a high cement factor, well-graded fine aggregate, and a low w/c of 0.35. A high dosage of a water-reducing and retarding admixture and revibration of the concrete also probably helped reduce the permeability.
- Partial longitudinal post-tensioning about 3 days after placement prevented transverse shrinkage cracking and reduced the possibility of chloride ingress.
- Stiffness of the prestressed superstructure minimized transverse load distribution stresses in the slab and reduced the chance of cracking.
- The BWC supplied some level of chloride–ion shielding.

CASE IV: IOWA PAVEMENT

The vibratory consolidation practices used for portland cement concrete (PCC) pavement became a concern to the Iowa DOT when overvibration was identified as a contributing factor to the premature deterioration of US-20 in Webster and Hamilton counties (Tymkowicz and Steffes, 1997). First noticed in 1990, the deterioration was noteworthy because the pavement was only 3 years old at the time. The pavement exhibited surface distress characteristics similar to the staining and cracking associated with D-cracking.

While the primary source of the cracking was thought to be chemical reactions, a second cracking pattern that emerged was attributed to freezing and thawing. Longitudinal cracking, evenly spaced at about 0.6 m, started to appear on the pavement surface. This spacing is consistent with the vibrator spacing used on the slipform paver for the project. Cores evaluated during the initial investigation into the deterioration showed many instances where the hardened concrete had air contents below 3%.

A similar cracking pattern was noticed on I-80 in Dallas County at approximately the same time. This pavement was also 3 years old when longitudinal cracking was first identified. Again, the spacing between the cracks approximated the transverse spacing of the vibrators on a slipform paver. Cores taken from the cracked areas showed air contents of 3% in the top half of the core and 6% in the bottom half.

Longitudinal trails have also been observed on the surface of PCC pavements in other areas of the state. These trails also run parallel to each other, in an approximate pattern of the vibrator spacing of a slipform paver. It was believed that these trails are formed by excessive vibration in the plastic concrete during the paving operation. The overvibration causes localized areas of high paste content, which allows the transverse tinning forks to penetrate into the concrete and result in visible trails in the pavement. These trails are also apparent in areas where the surface has been diamond ground, and the high paste areas are easily contrasted against areas where coarse aggregate is present.

The Iowa DOT conducted a research study in 1995 to determine the effect of vibrator frequency, paver speed, and transverse location in the pavement on air content. Test sections were paved on three paving projects, using a slipform paver operating at two different speeds (0.8 and 1.5 m/min) and three vibrator frequencies [5,000, 6,500, and 8,000 vibrations per minute (vpm)]. The vibrator frequencies were chosen to conform to the Iowa DOT specification that vibrators must operate between 5,000 and 8,000 vpm. Only one consecutive pair of vibrators had its speed controlled. The other vibrators on the paver were kept at the contractor's settings (which were supposed to conform to Iowa DOT specifications), and their frequencies were measured and recorded.

The pavements placed during the research project were all 300 mm thick, and 7.9 m wide, and used the same Iowa DOT mixture C-3WR-C20.

The study had the following results (these results pertain to the specific mixture proportions used in Iowa):

- Vibration frequencies varied by as much as 3,000 vpm between vibrators on a single paver even though all vibrators were set at the same speeds. In most cases the vibrators that were not part of the controlled study were outside of the 5,000 to 8,000 vpm range, usually above it, and in one case as high as 12,000 vpm.
- The contractors usually positioned the vibrators parallel to the pavement surface. However, there were variations of as much as 125 mm between the highest and lowest vertical positions of the vibrators on a single paver.
- Positioning of vibrators at the pavement surface may result in less consistent air content throughout the pavement, when compared with positioning of vibrators 100 mm below the surface.
- The radius of effective consolidation for a vibrator may be considerably smaller than originally thought. Cores showed significant entrapped air voids within 100 mm of the vibrator location.

- Vibrators operating at high frequencies (12,000 vpm in this study) will significantly lower the air content of the concrete immediately adjacent to the vibrator.
- Vibrators operating between the 5,000 and 8,000 specification limit do not negatively impact the air content of the concrete if normal (1.5 m/min) paver speed is maintained.
- If the paver speed is reduced to 0.7 m/min, the vibrators operating at 5,000 vpm did not negatively affect the hardened air content, but those operating at 8,000 vpm did.
- The paver hydraulic control valve settings should not be considered accurate, and frequent checks with a tachometer are recommended to ensure proper and consistent vibrator speed.

CASE V: PC I-BEAM END DETERIORATION, MICHIGAN

The focus of this research study was the evaluation and abatement of girder end deterioration in prestressed concrete I-beam bridges (Ahlborn et al., 2002). The study had three components: field inspection, experimental study, and analytical modeling. The purpose of the field inspection was to document the beam end distress states and collect data in order to understand the causes of girder end distress. The experimental study dealt with evaluating shallow and deep patches for beam end repair. The analytical study looked into the influence of prestressing actions and additional live and intrinsic loads on girder end distress.

The initial task was the documentation of observed girder end condition by inspecting 20 prestressed concrete I-beam bridges constructed in Michigan between 1961 and 1998. Upon documenting the prestressed concrete I-beam end condition in Michigan, a survey of state DOTs was conducted to document the observations in other states. A survey return rate of 40% was achieved with 20 states responding. All respondents indicated they do not gather specific inspection data on prestressed concrete beam end conditions. While most states have not repaired prestressed I-beams for end deterioration, roughly 50% percent of the respondents indicated that their state DOT specifications would be used in the rehabilitation of prestressed concrete I-beam ends. The only responding state to indicate that I-beam end repair has been attempted was Michigan.

In reviewing the observations and data obtained during the inspections, there are three inspection items of importance that are related to I-beam end deterioration. These items are presence of beam end cracking, bearing condition and beam end restraints, and drainage and expansion joint condition. Categories of condition of deteriorated prestressed concrete beam ends have been developed to define a distress level and an associated preventative maintenance or repair technique. Condition states for a prestressed concrete I-beam also describe the progression of distress at the beam end with time.

The study identified four major families of preventive maintenance approaches that can be applied to beam ends. These techniques were structure modification, surface insulating methods, electrical control methods, and environment modifying methods. The study developed three analysis tools for successfully executing a beam end repair project. The tools are testing procedures and distress severity criteria for PCC I-beam end deterioration, cause–evidence relationships for beam end distress, and an example performance matrix for preventive maintenance techniques.

The finite element (FE) modeling of a PCC I-girder was performed to evaluate the causes of observed beam end distress. The discrete beam analysis identified the effects of prestressing loads, and design changes with respect to tendon geometry and arrangement. Three types of prestressed concrete I-beams were modeled from existing bridges to determine the causes for initial bursting cracks at the end zones. The first model was a beam with straight tendons, the second beam was a Wisconsin type with and without bond breakers, and the third beam was with draped tendons. The stress formation at the end zones and cracking potential were studied. The analysis results indicated that there is a tendency for beam-end cracking with all the tendon configurations due to high shear and tension stresses upon release of tendons.

The structural interactions between the bridge members and the load-transfer mechanism to the beam ends were analyzed on a full bridge model. The structural behavior of the bridge under several service loading stages was analyzed. The impact of the diaphragms on beam ends was investigated by describing diaphragms with different geometry and cross sections and having different material properties. It was seen that changes in the stresses at the beam ends were insignificant with the use of different diaphragms.

Forms of distress at the beam ends include concrete spalling, delamination, cracking, and corrosion of reinforcement. The loss of concrete permits accelerated deterioration of reinforcing and prestressing steels, allows detensioning of prestressing steel, and increases the stress demand (bearing, shear, flexural) on the remaining section. Properly functioning repairs can restore cover to reinforcing and prestressing steels and reestablish the original intended cross section of the concrete. The two of the most important properties of concrete repair are crack resistance and substrate adhesion (bond). Crack resistance is needed to prohibit ingress of contaminants that can adversely affect the performance of the repair. Adhesion is required to assist the parent member in carrying loads as well as protecting the parent member (or repair) steel reinforcement from corrosion. A performance evaluation of vertical repair material was conducted and focused on evaluating crack development and repair bond tensile strength at the conclusion of a thermal cycling period. The performance measure for maximum repair crack width for this study was 0.15 mm. Visual observations of the repair condition at the conclusion of the post-curing period revealed cracking within the repair material itself and at the repair–substrate joints (i.e., top and bottom repair joints). For bond tensile strength, two sets of performance measures were observed. First, repairs cannot delaminate from the substrate and, second, a bond tensile strength of 2.75 MPa was required. Over one-third of the repair specimens did not meet the delamination performance criteria and none of the specimens were able to develop a bond tensile strength of greater than 2.75 MPa.

The outcome of the study is summarized below.

1. The prestressed concrete I-beam ends are often cracked. The cracking with the presence of moisture accelerates the girder end deterioration primarily by accelerating the chloride ingress process and corrosion initiation of shear reinforcement and prestressing tendons.
2. The recent deck design using the continuous live load system eliminates the expansion joint and consequently provides a roof over the beam end. Moisture access to the beam end and the ingress of chlorides are subsequently reduced. However, spray from traffic below and new diaphragm details, which encase the beam end and traps moisture, still make beam ends a vulnerable portion of the I-beam bridges. The diaphragm in this configuration also conceals the beam end, which makes visual inspection impossible. The primary approach for improving beam durability should be the elimination or reduction of beam-end cracking. In all existing bridges, beam ends with cracks of any width should be sealed.

3. Analytical models showed that the cracking potential is very high on straight and draped prestressed strand girders. The cracking potential is lower but still exists in sheathed or debonded girders, which affect the more recent manufacturing process. Prestressed concrete beam-end cracking is due to the transverse and shear stresses generated by axial load change along transfer length. These loads cannot be eliminated, but cracking can be minimized with the use of confinement steel near beam ends. Further study is needed to determine the exact arrangement and size of confinement steel.

4. Conclusions related to full bridge analysis include the effect of diaphragms and bearings on the stresses at the beam ends. The purpose of diaphragms is for girder stability during erection and transfer of shear between girders under live loads. It is seen in the analyses that the diaphragm geometry and material properties do not generate significant influence on the beam end stresses. The recent diaphragm design and material properties may require changes. Steel X-bracings may be a proper alternative, which provide ventilation for the beam ends. An efficient detail with steel X-braced girders and beam ends free of diaphragm should be further investigated.

5. The beam-end stresses are amplified due to nonfunctional neoprene bearings. Analytical studies show that beam end vulnerability is a concern for bridge safety for two reasons. First, the deteriorated portion of the girder end is often within the path of live-load transfer to the bearings. Second, the loss of bond near the ends reduces the prestressing force affecting the moment capacity. Load path is established under dead and service loads to assess when the deteriorated portions of beam end intrude into the load path.

6. Shallow and deep patch repairs on delaminated girder ends can be a way of restoring the cross section and preventing further progression of tendon corrosion. All patching materials, however, are not equal and may show significant differences in expected performance. The three repair materials evaluated herein showed unacceptable cracking, and none met the minimum adhesion criteria through bond tensile strength testing.

7. The bridge scoping, assessment, maintenance, and rehabilitation are often currently performed on roadway corridors. The scoping inspections are performed within a designated corridor. Beam ends at various condition states can be encountered. Utilizing the inspection data and further studies using the Pontis database, common beam end distress can be categorized into 12 condition states. The first six of these conditions can be dealt with using preventive maintenance.

CASE VI: EARLY-AGE CONCRETE BRIDGE DECK CRACKING, MICHIGAN

The need for this research was based on an observed deck deterioration mechanism that is accelerated by the existence of cracks (Attanayaka et al., 2003). The primary objective of this research was to identify the major parameters influencing early-age reinforced concrete (RC) deck cracking. The second objective was to develop recommendations for the modification of these parameters that are within the control of the bridge designer, the materials engineer, the contractor, or the maintenance engineer. The project tasks consisted of literature review, nationwide survey on the subject of RC deck cracking, field inspection and data collection of existing RC bridge decks of age 5 years or less, construction monitoring of new decks, laboratory and field testing, and data analysis and synthesis.

Synthesis of the literature indicated that the restrained thermal and shrinkage effects coupled with construction practices are the main parameters influencing deck cracking. The information extracted from the nationwide survey regarding the experience of other states with the problem of early-age deck cracking was also compared; specifically, with the states of the Central Northeast Region (Illinois, Minnesota, New York, Pennsylvania, and Wisconsin) with a similar climatic exposure to that of Michigan. All of these states indicated that they observe early-age cracking on concrete bridge decks and the prevalent type is transverse cracking. In order to control deck cracking, the most popular measures taken by these states are use of mineral admixtures, and changes to mix design and curing procedure. Illinois, New York, and Pennsylvania started adding fly ash (FA), silica fume, and ground granulated blast-furnace slag (GGBS) as mineral additives in their mix design, whereas Wisconsin has been adding FA and ground granulated blast furnace slag. New York, Minnesota, and Wisconsin are using retarder, air entrainer, and mid-range water reducer admixtures, but Illinois and Pennsylvania are using only air entrainer in their concrete mix design. The most often specified deck thickness among the states is 200 mm, but Minnesota utilizes a 225-mm deck. The common curing practice among these states is continuous wet curing with the exception of Illinois. The top three causes of cracking identified by the respondents from these states are substandard curing, construction practice, and mix design. These top three causes of cracking match with Michigan's responses.

The field inspection data analysis of 20 bridges indicated that crack density is higher on continuous bridges than on simple span bridges. Bridges with pavement condition index girders show minimum longitudinal crack density compared with other bridge girder types (i.e., steel, side-by-side box beam, and spread box beam). However, there is no clear relationship between deck crack density and bridge skew, deck thickness, span length, or average daily truck traffic.

During construction monitoring, observed curing procedures were often in conflict with the Michigan DOT *Standard Specifications for Construction*. In two of the five deck placement projects, curing compound was applied upon the placement of the full deck. Again, according to the specifications, wet curing should commence within 2 h of concrete placement. In all five of the deck replacement projects monitored, the burlap was placed after 12 h following concrete placement. Laboratory tests on concrete samples taken during construction monitoring indicate that out of the five bridges monitored, three had a deck concrete 28-day compressive strength in excess of 41 MPa. Concrete with a 28-day compressive strength greater than 41 MPa is classified as high-strength concrete and needs to comply with special construction and curing procedures. The laboratory tests also showed that the gain in compressive strength and elasticity modulus from 3 to 7 days were rapid, indicating high early-strength concrete properties. Concrete with such properties generates high thermal loads during hydration and high drying shrinkage during early ages. Consequently, increased deck cracking should be expected.

The synthesis of all the data collected revealed that the tensile stresses due to early-age thermal load alone could cause deck cracking. Volume change of concrete under thermal and shrinkage effects occur simultaneously. An increase in drying shrinkage from delays in wet curing will increase tensile stresses. Drying shrinkage, upon curing, will increase crack width that have formed under thermal loads. For a fixed-mix design, the ambient temperature at the time of concrete placement governs the early-age concrete thermal properties. Concrete mix parameters controlling the thermal load are the cement type, content, and fineness, and the time of inception of curing. The temperature difference between the peak temperature during hydration and the ambient temperature establishes the thermal load on the deck concrete. The thermal load controls the magnitude of the tensile stresses that develops in the deck. Use of retarders in the concrete mix delays the hydration process and may be an advantage or a drawback depending on the ambient temperature at the time of peak hydration temperature.

The first conclusion of this study is related to current practice. If the curing related stipulations of the Michigan DOT *Standard Specifications for Construction* are strictly adhered to, the density of transverse deck cracks will be reduced. This research established that an approximate thermal gradient of 11°C initiates deck cracking. Second, in order to reduce transverse cracking, the primary recommendation is to develop and optimize project specific mix design for the minimization of thermal load. As an incentive for developing a project specific mix design, peak concrete temperature during hydration may be defined as a performance parameter. The limits to the hydration temperature may be specified in the Standard Specifications. Inclusion of concrete hydration temperature in the specifications is feasible since it is measurable and with certain limitations (cement mill properties) is within the control of the contractor by the concrete mix design and curing.

CASE VII: CONCRETE BARRIER CRACKING, MICHIGAN

This research need was established in a report by Staton and Knauff (1999), *Evaluation of Michigan's Concrete Barriers*. The report described that many of the current generation barriers used by the Michigan DOT are deteriorating at a rate greater than expected. The objectives of this project were to investigate the causes of concrete bridge barrier deterioration with the goal of developing strategies for corrective action (Attanayake and Aktan, 2003). The project was designed in eight tasks including literature review, a nationwide survey of State Highway Agencies, inspection of existing barriers, monitoring barrier construction, laboratory testing, and data synthesis.

Field inspection was conducted to document the condition of barriers of a total of 21 bridges. According to the inspection data, the vertical (or transverse) cracking is the leading cause of most of other distress types. The number of vertical cracks is important in establishing the deterioration rate. Horizontal cracks can be classified as either local or continuous. The local horizontal cracks are often on the barrier's vertical face. Continuous horizontal cracks are also mostly observed on the vertical face, about the level of top longitudinal reinforcement. In barriers with horizontal cracking, significant section loss is often observed around the barrier's top portion.

Utilizing findings of field inspection and literature review, a nationwide survey of state highway agencies was conducted. The survey responses provided information with regard to the early-age cracking problem with barriers and, materials and construction practices used for concrete barriers (especially New Jersey type). All the survey respondents indicated that premature distress is observed on concrete bridge barriers. Those observed distresses are of the same types observed in Michigan. Though all the respondents identified premature distress, only Illinois, New Mexico, Vermont, and Virginia acknowledged that they have experienced an overall durability problem with the bridge barriers. Nationally, both form cast and slip-formed barriers are commonly used. Precast New Jersey-type barriers are also used in several states. Sprayed curing compound on the slip-formed barrier surfaces is the most often cited method of curing. Illinois, Minnesota, New Hampshire, New Mexico, New York, and Washington use a different concrete mix design for the barriers than for the deck. All the respondents specified the use of GGBS and FA in the mix design for the goal of reduction of the concrete permeability.

States of Massachusetts, New York, and Virginia also use silica fume along with GGBS and FA. Most of the respondents emphasized changing the mix design and curing procedure for improved durability of New Jersey-type barriers.

The construction of four bridge superstructure replacement projects with New Jersey-type barriers was monitored. In two of the projects, the barriers were slip formed. In the other two projects, barriers were form cast using metal forms on the traffic bearing side and wooden forms on the fascia. In the slip-formed barriers, concrete was not sufficiently consolidated. Though it is difficult to conclusively evaluate the barrier interior without taking well-distributed core samples, while the joints were being cut, honeycombing and large cavities were observed. The curing compound was sprayed using the Michigan DOT–recommended procedure. However, the spray was not uniform over the barrier surface as specified in the Michigan DOT *Standard Specifications for Construction*. Two days after placement, inspection revealed map cracking on most of the portion of the barrier surface as well as few full-length vertical cracking. In form-cast barriers, the top surface of the concrete is not covered or protected from direct atmospheric exposure. Forms were removed approximately 18 h following construction (this is in compliance with the specification requirements). Visual inspection upon form removal did not reveal any visible cracking. The laboratory test results indicated that two of the bridge barriers were cast with 28-day concrete strength in excess of 41 MPa. Several core samples were obtained from existing barriers. Core samples taken from the zones of distress displayed excessive leakage. The existence of cracks and large voids triggered the leakage.

The primary factor affecting durability of concrete barriers in Michigan was determined as the formation of multiple full- or partial-depth vertical cracks. The causes of cracking were identified as the internal restraint stresses resulting from thermal and shrinkage loads during cement hydration. It was also established that other distress types often emanate from vertical cracking. The reduction or control of vertical cracking will most certainly improve barrier service life. Inspection data from existing barriers and newly slip-formed barriers (2 days following placement) showed that the average full-length vertical crack spacing is twice the barrier height. On the other hand, FE analysis of a barrier segment with full base restraint showed that the full-length vertical crack spacing is equal to the barrier height. Vertical crack spacing increases with reduced base restraint conditions. The difference between observed crack spacing in the field and the results of the analysis is due to reduced barrier base restraint.

According to the findings of this study, the early barrier deterioration is initiated by the vertical cracking and accelerated by the presence of voids, cavities, and the overall concrete quality of the barrier. Recommendations are made by emphasizing the fact that early-age crack control, or in more general terms, crack management, is the key to durable barriers. It is recommended that crack arrestors be used by placing a trim inside the forms at approximately three feet intervals. The crack arrestors, some with cracks formed at full length should be sealed with a durable silicone-based flexible material during the first scheduled maintenance cycle. Additional recommendations include substitution of mineral admixtures, shifting barrier casting process to evening or night, protecting the top surface of form-cast barrier with curing compound or a wet burlap, and delaying form removal to 5 or even 7 days after concrete placement.

CASE VIII: SIDE-BY-SIDE BOX-BEAM BRIDGE PERFORMANCE, MICHIGAN

Side-by-side box-beam bridge is the bridge of choice for short (20 to 60 ft) and short- to medium-span bridges (60 to 110 ft) (Ahlborn et al., 2005). Another significant advantage of this bridge type is that construction is rapid and traffic can be maintained below the bridge without interruptions. Compared to other bridge structural systems, side-by-side box-beam bridges require shorter duration of construction. Side-by-side box-beam bridges are popular because of these advantages. This type of bridge is typically constructed by placing precast–prestressed box beams adjacent to each other, grouting shear keys, and applying transverse posttension using tie rods or prestressing tendons, and with a wearing surface or a cast-in-place reinforced concrete deck. The resulting superstructure behaves as a simply supported plate. The integrity of the plate behavior becomes compromised when longitudinal reflective deck cracks form along the shear keys allowing surface water to penetrate and become trapped between the box beams. Water saturated with deicing salts eventually penetrates along the full length of the beams and initiates corrosion of prestressing tendons. Available documentation shows that the reflective deck cracking is the leading cause of the premature deterioration process of the bridge. Though there are significant advantages, this premature deterioration mechanism forced the highway agencies to reconsider the use of this particular bridge type. Currently, there is a growing concern of revising the design procedures in order to alleviate reflective deck cracking and promote the use of this bridge type. With this premise, Michigan DOT initiated this multiphase project. As the first phase of the project, the following objectives were identified:

- Identify common types of deterioration in Michigan box beam bridges and develop inspection techniques for early identification of cracking and strand corrosion of the beams.
- Develop guidelines to assist inspectors in determining when section loss may reduce structural capacity.
- Provide guidelines for the load capacity assessment of bridges with distressed beams based on FE modeling.
- Identify effective maintenance or repair techniques for deteriorated regions of box beam bridges considered to be in good or fair condition.
- Develop recommendations for changes or modifications to the design of side-by-side box-beam bridges.

Criteria were established for the selection of 15 side-by-side prestressed concrete box-beam bridges for inspection. Bridge selection was based on manageable accessibility of the structure. The pool of bridges selected for inspection was separated into two groups; those built before 1974 and after 1987. Bridges built between 1974 and 1987 were neglected due to ephemeral design procedures and lack of in-service bridges. It was essential that bridges over a range of ages be incorporated into the inspection to extract the progression of distress and the mechanisms which cause it. Eight and seven bridges were chosen from the bridges that were built before 1974 and after 1987, respectively.

Maintenance and inspection of Michigan's prestressed concrete box-beam bridges is imperative to the preservation of the state's infrastructure. Inspection of these bridges is different from other bridge types for two reasons. First, the designed interaction between beams makes inspection of the grouted keyways and transverse posttensioning system important, and second, due to the placement of beams, many beams may only be inspected along the bottom flange and other indicators must be reviewed for indication of problems in the beams. Deterioration of individual box beams comes in many forms. Thirteen kinds of deterioration are identified in the inspection handbook developed during this project. The types of distress are ranked according to their level of structural significance by a condition rating specific to this project. The grades of deterioration provided in this handbook are specific to prestressed box beams and therefore contain more box-beam–specific detail than the National Bridge Inspection condition ratings. Flowcharts were created which may be used by the design engineer to assess the proper repairs for distress identified by the field inspector. These flowcharts provide means to first identify the type of distress, whether structural or material. Failure to differentiate between material and mechanical deteriorations may result in failure of the repair. The intent of these flowcharts is to determine the proper repair for the identified distress. Many of the repairs may be made following the Michigan DOT *Standard Specifications for Construction*. A design engineer may be required to design repairs for material related or severe forms of distress. There may also be unique site or project specific conditions for which an engineer with experience in distress related repair should be consulted. Use of the inspection handbook for early identification of common forms of distress in conjunction with the inspection report forms and repair flowcharts are beneficial in achieving increased service life of side-by-side box-beam bridges.

The objective of the FE modeling was to determine the impact of distress on shear and flexural capacities of the box beams. Two FE models were developed for flexure critical and shear critical beam lengths. The physical condition of the bridge and the girders were documented through the field inspection and the inspection data analysis indicated the most common distress types and levels of structural significance. Distresses were incorporated into the FE models and load rating was performed. The flexure critical and shear critical beam analysis was performed with the same truck type and found that the flexural capacity governs the beam failure.

As per the project (Phase I) objectives, common types of deterioration of side-by-side box-beam bridges associated with longitudinal reflective deck cracking are identified as corrosion of reinforcement and prestressing tendons, concrete delamination and spall along the edges of box-beam bottom flange, and broken tendons. Potential use of some nondestructive testing techniques for early detection of box-beam deterioration were identified as rivet gun chipper, corrosion rate measurements, surface measurements methods, linear polarization, infrared thermography, and gammagraphy. Guidelines were developed for structural capacity assessment of box beams with various distress types. Maintenance or repair techniques were identified and flowcharts were developed to identify proper repair for the identified distress types. During field inspection and data analysis it was identified that the prestressing tendons close to the boundary of the box-beam cavity were exposed to more severe weather conditions than the outmost layer of strands, and hence it is recommended to review the cover of prestressing tendons near the top of the bottom flange.

Standards and References

Updated by Heather Dylla, Louisiana State University

AASHTO (ASTM) Standards

M – Standard Specification
T – Standard Test Method
* – Specification or method does not exactly match ASTM counterpart.

AASHTO	ASTM	Title
M 85	C 150	Portland Cement
M 154	C 260	Air-Entraining Admixtures for Concrete
M 194*	C 494	Chemical Admixtures for Concrete
M 205	C 470	Molds for Forming Concrete Test Cylinders Vertically
M 240	C 595	Blended Hydraulic Cements
M 295	C 618	Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
M 302	C 989	Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars
M 307	C 1240	Standard Specification for Silica Fume Used in Cementitious Mixtures
M 321		Standard Specification for High-Reactivity Pozzolans for Use in Hydraulic-Cement Concrete, Mortar, and Grout
T 2	D 75	Practice for Sampling Aggregates
T 11*	C 117	Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing
T 19	C 29	Unit Weight and Voids in Aggregate
T 21*	C 40	Organic Impurities in Fine Aggregates for Concrete
T 22	C 39	Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens
T 23*	C 31	Making and Curing Concrete Test Specimens in the Field
T 24	C 42	Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
T 27*	C 136	Sieve Analysis of Fine and Coarse Aggregates
T 32*	C 39	Compressive Strength of Cylindrical Concrete Specimens
T 71*	C 87	Effect of Organic Impurities in Fine Aggregate on Strength of Mortar
T 84*	C 128	Specific Gravity and Absorption of Fine Aggregate
T 85*	C 127	Specific Gravity and Absorption of Coarse Aggregate
T 97	C 78	Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
T 112*	C 142	Clay Lumps and Friable Particles in Aggregates
T 119	C 143	Slump of Hydraulic Cement Concrete
T 121	C 138	Mass per Cubic Meter (Cubic Foot), Yield, and Air Content (Gravimetric) of Concrete
T 126	C 192	Making and Curing Concrete Test Specimens in the Laboratory
T 141*	C 172	Sampling Freshly Mixed Concrete
T 152	C 231	Air Content of Freshly Mixed Concrete by the Pressure Method
T 158	C 232	Bleeding of Concrete
T 160	C 157	Length Change of Hardened Hydraulic-Cement Mortar and Concrete
T 161	C 666	Resistance of Concrete to Rapid Freezing and Thawing

T 161	C 666	Resistance of Concrete to Rapid Freezing and Thawing
T 177	C 293	Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading)
T 196	C 173	Air Content of Freshly Mixed Concrete by the Volumetric Method
T 197	C 403	Time of Setting of Concrete Mixtures by Penetration Resistance
T 198	C 496	Splitting Tensile Strength of Cylindrical Concrete Specimens
T 231	C 617	Capping Cylindrical Concrete Specimens
T 255	C 566	Total Moisture Content of Aggregate by Drying
T 277	C 1202	Electrical Indication of Concrete's Ability to Resist Chloride-Ion Penetration
T 259	C 1543	Resistance of Concrete to Chloride Ion Penetration
T 303	C 1260	Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)
	C 33	Specification for Concrete Aggregates
	C 70	Test Method for Surface Moisture in Fine Aggregate
	C 94	Specification for Ready-Mixed Concrete
	C 114	Test Methods for Chemical Analysis of Hydraulic Cement
	C 215	Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens
	C 227	Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)
	C 289	Test Method for Potential Reactivity of Aggregates (Chemical Method)
	C 418	Test Method for Abrasion Resistance of Concrete by Sandblasting
	C 441	Test Method for Effectiveness of Mineral Admixtures in Preventing Excessive Expansion of Concrete Due to Alkali-Aggregate Reaction
	C 452	Test Method for Potential Expansion of Portland Cement Mortars Exposed to Sulfate
	C 457	Practice for Microscopical Determination of Air-Void Content and Parameters of the Air-Void System in Hardened Concrete
	C 469	Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression
	C 496	Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
	C 512	Test Method for Creep of Concrete in Compression
	C 586	Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)
	C 597	Test Method for Pulse Velocity Through Concrete
	C 642	Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete
	C 671	Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing
	C 672	Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
	C 682	Practice for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures
	C 684	Method of Making, Accelerated Curing, and Testing of Concrete Compression Test Specimens
	C 779	Test Method for Abrasion Resistance of Horizontal Concrete Surfaces
	C 803	Test Method for Penetration Resistance of Hardened Concrete
	C 805	Test Method for Rebound Number of Hardened Concrete

C 823	Practice for Examination and Sampling of Hardened Concrete in Constructions
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C 856	Practice for Petrographic Examination of Hardened Concrete
C 873	Test Method for Compressive Strength of Concrete Cylinders Cast in Place in Cylindrical Molds
C 876	Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete
C 900	Test Method for Pullout Strength of Hardened Concrete
C 944	Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating Cutter Method
C 1012	Test Method for Length Change of Hydraulic-Cement Mortars Exposed to Sulfate Solution
C 1017	Specification for Chemical Admixtures for Use in Producing Flowing Concrete
C 1038	Test Method for Expansion of Hydraulic Cement Mortar Bars Stored in Water
C 1040	Test Methods for Density of Unhardened and Hardened Concrete In Place by Nuclear Methods
C 1062	Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete
C 1064	Test Method for Temperature of Freshly Mixed Portland Cement Concrete
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C 1078	Test Methods for Determining Cement Content of Freshly Mixed Concrete
C 1079	Test Methods for Determining Water Content of Freshly Mixed Concrete
C 1084	Test Method for Portland Cement Content of Hardened Hydraulic-Cement Concrete
C 1105	Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction
C 1138	Test Method for Abrasion Resistance of Concrete (Underwater Method)
C 1150	Test Method for the Break-Off Number of Concrete
C 1152	Test Method for Acid-Soluble Chloride in Mortar and Concrete
C 1157	Performance Specification for Hydraulic Cements
C 1218	Test Method for Water-Soluble Chloride in Mortar and Concrete
C 1231	Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Concrete Cylinders
C 1252	Test Methods for Uncompacted Void Content of Fine Aggregate (as Influenced by Particle Shape, Surface Texture, and Grading)
C 1293	Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali–Silica Reaction
C 1543	Test Method for Determining the Penetration of Chloride Ion into Concrete by Ponding
C 1556	Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion
C 1567	Test Method for Determining the Potential Alkali–Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)

C 1585	Test Method for Measurement of Rate of Absorption of Water by Hydraulic-Cement Concretes
C 1603	Test Method for Measurement of Solids in Water
C 1646	Standard Practice for Making and Curing Test Specimens for Evaluating Resistance of Coarse Aggregate to Freezing and Thawing in Air-Entrained Concrete

ACI Standards and Reports (American Concrete Institute, Farmington Hills, Michigan)

201.2R	Guide to Durable Concrete
211.1	Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
211.2	Standard Practice for Selecting Proportions for Structural Lightweight Concrete
211.3	Guide for Selecting Proportions for No-Slump Concrete
211.4	Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash
211.5	Guide for Submittal of Concrete Proportions
222R	Corrosion of Metals in Concrete
225R	Guide to the Selection and Use of Hydraulic Cements
232.1R	Use of Raw or Processed Natural Pozzolans in Concrete
233R	Ground Granulated Blast-Furnace Slag as a Cementitious Constituent in Concrete
234R	Guide for the Use of Silica Fume in Concrete
304R	Guide for Measuring, Mixing, Transporting, and Placing Concrete
304.1R	Guide for the Use of Preplaced-Aggregate Concrete for Structural and Mass Concrete Applications
304.2R	Placing Concrete by Pumping Methods
304.4R	Placing Concrete with Belt Conveyors
304.5R	Batching, Mixing, and Job Control of Lightweight Concrete
305R	Hot Weather Concreting
306R	Cold Weather Concreting
308	Standard Practice for Curing Concrete
309R	Guide for Consolidation of Concrete
318	Building Code Requirements for Structural Concrete

U.S. Army Corps of Engineers Test Method

CRD-C163-92 Test Method for Water Permeability of Concrete Using Triaxial Cell

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Publication 102	Recommended Guide Specifications for Batching Equipment and Control Systems in Concrete Batch Plants, Concrete Plant Manufacturers Bureau
Publication 148	Mixing Concrete in a Truck Mixer
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