

Guide to Durable Concrete

Reported by ACI Committee 201



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Guide to Durable Concrete

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Guide to Durable Concrete

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This guide describes specific types of concrete deterioration. Each chapter contains a discussion of the mechanisms involved and the recommended requirements for individual components of concrete, quality considerations for concrete mixtures, construction procedures, and influences of the exposure environment, which are all important considerations to ensure concrete durability.

This guide was developed for conventional concrete but is generally applicable to specialty concretes; however, specialty concretes, such as roller-compacted or pervious concrete, may have unique durability-related issues that deserve further attention that are not addressed herein. Readers should consult other ACI documents for more detailed information on special concretes of interest.

Keywords: abrasion resistance; acid attack; admixture; aggregate; air entrainment; alkali-aggregate reaction; calcium chloride; carbonation; cement paste; corrosion; curing; deicer; deterioration; durability; fly ash; freezing and thawing; mixture proportion; petrography; pozzolan; reinforced concrete; salt scaling; sea water exposure; silica fume; skid resistance; spalling; strength; sulfate attack; supplementary cementitious materials; temperature; water-cementitious material ratio.

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CHAPTER 1—INTRODUCTION AND SCOPE

Concrete is one of the most widely used construction materials in the world. This fact attests to concrete's performance as a versatile building material. Durability represents one of the key characteristics of concrete that has led to its widespread use. Durability of hydraulic-cement concrete is determined by its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality, and serviceability when exposed to its environment. Properly designed, proportioned, placed, finished, tested, inspected, and cured concrete is capable of providing decades of service with little or no maintenance. Certain conditions or environments exist that will lead to concrete deterioration. Attacking mechanisms can be chemical, physical, or mechanical in nature, and originate from external or internal sources. Chemical and physical attacking mechanisms often work synergistically. Depending on the nature of attack, distress may be concentrated in the paste, aggregate, or

reinforcing components of the concrete (or a combination thereof).

The various factors influencing durability and the particular mechanism of deterioration should be considered in the context of the environmental conditions to which the concrete would be subjected. In addition, consideration should be given to the microclimate to which the specific structural element is exposed. Deterioration, or the severity of deterioration, of a given structure may be affected by its orientation to wind, precipitation, or temperature. For instance, exterior girders in a bridge structure may be exposed to a different and more aggressive environment than interior girders.

The concept of service life is increasingly used for the design of new structures. To provide durable concrete, the specific demands on the concrete in its intended use should be given careful consideration. Required service life, design requirements, and expected exposure environments (macro and micro) should be determined before defining the appropriate materials and mixture proportions necessary to produce concrete suitable for a particular application. The use of good materials and proper mixture proportioning will not necessarily ensure durable concrete. Appropriate measures of quality control, testing, inspection, placement practices, and workmanship are essential to the production of durable concrete. Properly designed testing and inspection programs that use trained and certified personnel are also important to ensure that durable concrete is produced. ACI has a number of certification programs that are applicable. This guide discusses the more important causes of concrete deterioration and gives recommendations on how to prevent such damage. Chapters on fresh concrete, freezing and thawing, alkali-aggregate reaction (AAR), aggressive chemical exposure, corrosion of metals, and abrasion are included. Fire resistance of concrete and cracking are not addressed in detail, because they are covered in ACI 216.1, 224R, and 224.1R, respectively.

Fresh or unhardened concrete can be consolidated and molded to the desired shape to serve its intended purpose. During this stage, a number of properties significantly influencing the durability of the hardened concrete are established. Pore structure development, air-void system formation, material mixing, placement and consolidation, curing, and minimizing or eliminating cracking of plastic concrete are all important to the ultimate durability of concrete.

Deterioration of concrete exposed to freezing conditions can occur when there is sufficient internal moisture present that can freeze at the given exposure conditions. Freezing-and-thawing damage is a serious problem, and is greatly accelerated by the use of deicing salts. Fortunately, concrete made with high-quality aggregates, a low water-cementitious material ratio (w/cm), a proper air-void system, and that is allowed to mature before being exposed to freezing and thawing, is highly resistant to freezing-and-thawing damage.

Although aggregate is commonly considered to be inert filler, this is not always the case in a concrete environment. Certain aggregates can react with alkali hydroxides from cement and other materials, causing expansion and deterioration.

Potential issues with AARs can often be identified by reviewing the historical performance of candidate materials under similar exposure conditions and by evaluation based on the appropriate laboratory testing techniques. Methods for mitigating AARs are: use of low-alkali cement, avoidance of reactive aggregate, use of sufficient levels of suitable supplementary cementitious materials (SCMs), use of sufficient levels of appropriate chemical admixtures, or a combination of these.

Sulfates and other salts in soil, groundwater, or seawater may have a deleterious effect on hardened concrete, but can be resisted by using suitable cementitious materials and proper quality control and curing of a properly proportioned concrete mixture.

Because the topic of delayed ettringite formation (DEF) remains a controversial issue and is the subject of various ongoing research projects, no definitive guidance on DEF is provided in this document. It is expected that future versions of this document will address DEF in significant detail.

Quality concrete can resist occasional exposure to mild acids, but no portland-cement concrete offers good resistance to attack by strong acids or compounds that convert to acids; special protection is necessary in these cases. Substances that are initially inert may convert to acids over time, and can also present durability issues.

Corrosion of embedded steel reinforcement produces reaction products that occupy additional volume. This can lead to internal stress within reinforced concrete and subsequent distress. The spalling of concrete due to corrosion is a particularly serious problem. Corrosion can also weaken reinforcing steel and thus reduce the structural capacity of concrete. One of the principal causes of reinforcing steel corrosion is the ingress of chloride made available from sources such as deicing salts. Ample cover over the steel and use of a low-permeability, air-entrained concrete will ensure durability in the majority of cases. More positive protection, such as epoxy-coated reinforcing steel, cathodic protection, noncorrosive reinforcement, or chemical corrosion inhibitors, is needed for severe exposures.

Abrasion can be a problem in industrial floors from traffic or process conditions. In hydraulic structures, particles of sand or gravel in flowing water can erode surfaces. The use of high-quality concrete and, in extreme cases, abrasion-resistant aggregate, will usually result in adequate durability under these exposures. The use of studded tires on vehicles has caused serious wear in concrete pavements; conventional concrete will not withstand this damage. Impact caused by operating equipment, such as fork lifts and heavy equipment, will pulverize concrete of inadequate strength, and may require the selection of special aggregates and mixture proportions.

In summary, this document addresses durability by dividing specific modes of attack into separate chapters. This approach was taken for organizational purposes and for convenience to the reader. In practice, one will find that damage to concrete is often due to multiple distress mechanisms.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation and definitions

ACI provides a comprehensive list of acceptable notations and definitions through an online resource, “ACI Concrete Terminology” (American Concrete Institute 2008). Some notations and definitions contained in this version of ACI 201.2R are not defined in “ACI Concrete Terminology” and are described as follows:

Philleo factor (F')—a distance, used as an index of the extent to which hardened cement paste is protected from the effects of freezing, so selected that only a small portion of the cement paste (usually 10%) lies farther than that distance from the perimeter of the nearest air void.

specific surface (α)—the surface area of air voids divided by their volume, expressed in compatible units so that the unit of specific surface is a reciprocal length.

spacing factor ($\bar{L}$)—an index related to the maximum distance of any point in a cement paste or in the cement paste fraction of mortar or concrete from the periphery of an air void.

CHAPTER 3—FRESH CONCRETE

3.1—Introduction

Concrete in its unhardened state (fresh concrete) is a readily deformable material consisting of hydraulic cement; aggregates; water; and often chemical admixtures, SCMs, or both. Fresh concrete can be consolidated and molded into the desired shape to serve its intended purpose. During this unhardened state, the desirable properties of the fresh concrete, such as target slumps and air contents, are determined, and the structure of the air-void system is developed (Powers 1978).

3.2—Pore structure

Both mixing and placement methods of fresh concrete contribute to determining the final arrangement of the concrete pore structure. The overall pore structure and its relation to transport mechanisms, permeability, and porosity will significantly influence the durability and degree of satisfactory performance of the structure relative to interaction with environmental conditions and internal reactions.

The transport of deleterious agents into and within hardened concrete has an important influence on the durability of concrete structures. The rate, extent, and effect of the transport are largely dependent on the pore structure of the concrete (amount, shape, size of pores, and pore-size distribution), the presence of cracks in the concrete, and on the microclimate at the concrete surface (Fig. 3.1) (Schiessl 1992). There is a wide range in the size of pores in the cement paste. The distribution of air-void pore sizes has a pronounced influence on the rate of transport into and within the concrete. In general, pore sizes may be classified as gel pores (less than 3 nm [1.2×10^{-7} in.]), capillary pores (3 to 3000 nm [1.2×10^{-7} to 1.2×10^{-4} in.]), and entrapped- and entrained-air voids (greater than 3000 nm [1.2×10^{-4} in.]) (Philleo 1986). The larger capillary pores, as well as the entrapped- and entrained-air voids, are particularly relevant to concrete durability.

Pores that are not the result of entrained or entrapped air are created from the space in the paste that was originally filled with mixing water. For plain cement pastes (without SCMs), a w/cm of approximately 0.4 is needed to hydrate all

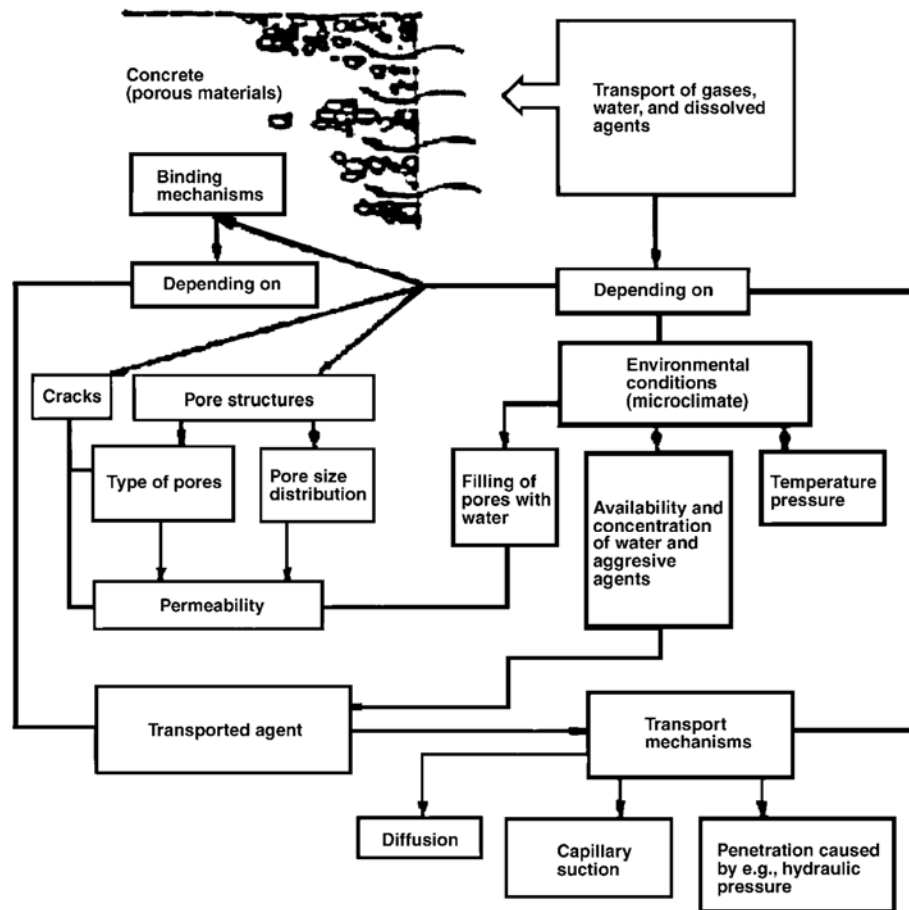


Fig. 3.1—Transport phenomena in concrete (Schiessl 1992).

of the cement; approximately one-half of this water will eventually be chemically combined, while the other half is water that is physically adsorbed on the surfaces of the gel. If the w/cm is less than 0.4, there will always be unhydrated cement left because there is insufficient original mixing water for all of the hydration products to form. If the w/cm is more than 0.4, there will be some unfilled capillary-size, original mixing-water-filled space left. The capillary pore space will fill as hydration products form. If the w/cm is 0.4, the capillary pores become discontinuous after approximately 3 days; at a w/cm of 0.8 or higher, they never become discontinuous (Powers 1959).

The air-void structure of concrete is created during mixing of the fresh concrete. If an air-entraining admixture is added or air-entraining cement is used, microscopic air voids are created that provide resistance to freezing and thawing. To be effective in relieving pressures generated as water cools and freezes, the air voids should be spaced closer than a critical maximum distance from one another within the paste component of the concrete. This critical spacing factor $\bar{L}$ is generally considered as less than 0.20 mm (0.008 in.) for most concrete (Section 4.2.3).

3.3—Mixing effects

The method selected for mixing fresh concrete can have very significant effects on important properties of the concrete such as air content, slump, and w/cm . Care should

be taken to ensure that the concrete placed has the properties needed to provide the durability required for the anticipated use and exposure of the concrete.

Extended mixing times can have a significant impact on air content, with the effect generally being more pronounced at higher air contents (Kosmatka et al. 2002). Extended mixing will also result in higher concrete temperatures. This may cause significant slump loss, especially during hot weather. Additional water added to restore slump would increase porosity and impact pore size distribution, leading to increased drying shrinkage and reduced concrete strength.

3.4—Placement and consolidation

The placement method used can significantly impact the amount of entrained air in the concrete. A summary of laboratory test results provided by Yingling et al. (1992) showed that inappropriate placement techniques can lead to a significant reduction in the air content of the concrete. It was shown that the impact of concrete freely falling through air or vertical pipe sections can reduce air content, which may result in concrete less resistant to freezing and thawing. Excessive freefall of concrete through reinforcing bar cages may impede proper consolidation and, thus, durability. Refer to ACI 304R for specific guidelines. Pumping concrete may change the air-void system of the concrete so that the resistance to freezing and thawing is impaired (Section 4.2.4).

When consolidating concrete, the duration of vibration can have a significant effect on the air content of concrete, with the effect being influenced by the initial slump of the concrete (Kosmatka et al. 2002). Increasing the time of vibration leads to a reduction in the overall air content. Consolidation, however, is normally necessary to obtain the strength and durability properties required over the service life of the concrete structure. Provided that the concrete is proportioned and vibrated properly, internal vibration will mainly remove the larger air voids, so that the air-void spacing factor will not be affected adversely. Placement and consolidation methods should be carefully evaluated to ensure that the desired fresh concrete properties are attained (ACI 309R).

Good consolidation is a prerequisite for obtaining low permeability, which is critical for making concrete resistant to weathering and most agents of deterioration. Low permeability can be achieved more readily with concrete mixtures designed for good workability and placeability. ACI 211.1 and 211.2 provide guidance for selecting mixture proportions, and ACI 309R provides guidance for consolidating concrete.

3.5—Bleeding

Once concrete is in place, but before the time of initial setting, particles denser than water (such as cement and aggregates) may settle due to the force of gravity, displacing part of the mixing water which may reach the top surface as bleed water. This condition is more pronounced in mixtures with excessive water content. Also, non-air-entrained concrete mixtures will exhibit more bleeding than air-entrained mixtures of similar composition. It is the difference in density between water and other components in the mixture that causes bleeding to occur. Just as entrapped air moves to the surface (Lankard 1995) during vibration, water will be displaced due to settling of the solid components of the concrete.

Defective hardened concrete surfaces are frequently the result of finishing fresh concrete while bleed water is present on the surface. If bleed water is worked into the concrete near the surface by finishing operations, the surface concrete can suffer from increased w/cm and decreased air content. Premature finishing of the concrete surface while bleeding is still active may result in an accumulation of bleed water below the sealed surface that can result in subsequent spalling, scaling, or delamination. Problems such as craze cracks and dusting can occur, and abrasion resistance can be reduced. Scaling of the surface from freezing and thawing is a common result of reduced air content in the surface concrete.

3.6—Cracking of fresh concrete

Cracks can form in fresh concrete as a result of plastic shrinkage, settlement, or temperature differential between the exposed surface and the interior. Insulating blankets can prevent temperature differential-induced cracking.

Plastic shrinkage cracking is characterized by a series of short, discontinuous cracks that form before initial setting of the concrete. Plastic shrinkage cracks may occur approximately parallel to each other, but will typically occur in a random, irregular pattern. Plastic shrinkage cracks may be only

superficial, or they may extend more deeply into a slab, even to its full depth.

Plastic shrinkage cracking is caused by rapid evaporation of moisture from the surface. If the loss of moisture at the surface is greater than the rate that moisture can be replenished by bleed water or capillary action, tensile stress will develop in the surface region. This phenomenon is related to the evaporation rate, which in turn is influenced by the combined effects of humidity, wind, and concrete and air temperatures. Detailed discussions of the mechanism and preventative measures can be found in ACI 305R, 308R, and 302.1R.

Settlement cracks typically occur directly above reinforcing bars and extend to the bar. Settlement cracking is caused by subsidence of fresh concrete, which can occur due to improper consolidation or, more commonly, from bleeding.

Because cracks act as an avenue for the entry of water, deicing chemicals, carbon dioxide, and aggressive chemicals, their presence may lead to a reduction in the durability of the concrete. The extent of the influence on durability will be dependent on the width and, more importantly, the depth, of the cracks. Cracks less than 5 mm (0.2 in.) deep may have a negligible effect. Cracks of greater depth, however, increase the penetration of carbon dioxide, water, and deicing chemicals, increasing the likelihood of reinforcement corrosion, damage due to freezing and thawing, and AAR. For nonreinforced concrete not exposed to freezing and thawing or aggressive chemicals, plastic shrinkage cracks may be of little significance, other than for aesthetic considerations.

Cracks in fresh concrete can appear to be closed during the finishing operations. The closure, however, may be only a thin skin over the surface of the crack, and the crack may subsequently reopen. If such cracking occurs, actions should be taken to prevent the continued occurrence of cracks forming in the fresh concrete.

3.7—Summary

The strength and durability of concrete can be significantly impaired by the use of improper placement, finishing, and curing techniques while concrete is in the unhardened state. Appropriately proportioned concrete mixtures that are properly placed, consolidated, finished, cured, tested, and inspected will help to ensure that the desired durability characteristics are achieved.

CHAPTER 4—FREEZING AND THAWING OF CONCRETE

4.1—Introduction

Deterioration of concrete exposed to freezing conditions can occur when there is sufficient internal moisture present that can freeze at the given exposure conditions. The source of moisture can be either internal (water already in the pores of concrete that is redistributed by thermodynamic conditions to provide a high enough degree of saturation at the point of freezing to cause damage) or external (water entering the concrete from an external source, such as rainfall). Dry concrete (generally below approximately 75 to 80% internal relative humidity) is normally immune to damage from freezing.

Fresh concrete, in an immature state, can be damaged by a single freeze (Section 4.2.2). Mature concrete may be able to withstand repeated cycles of freezing and thawing without damage. Thus, concrete that is properly cured and reaches sufficient maturity before being exposed to freezing, such as concrete for columns or floor slabs, may tolerate freezing from exposure to a single winter season before concrete is protected from the elements. Similar concrete that is not properly cured and is exposed to freezing conditions at an early age, such as sidewalks and exposed foundation walls, may show deterioration after a few years of exposure to repeated cycles of freezing and thawing.

The severity of concrete exposure should be quantified by a combination of freezing conditions (number of annual cycles of freezing plus average low temperature reached during each cycle) plus moisture condition before each cycle of freezing. This is covered in more detail in [Section 4.2.6.4](#).

4.1.1 Concrete made with durable aggregate—Concrete made with frost-resistant aggregate that is resistant to freezing and thawing is primarily protected from damage through the use of entrained air in the concrete mixture, with secondary protection from steps taken to limit the fractional volume of freezable water in the concrete. A description of damage due to freezing and thawing of concrete made with durable aggregates, along with protection methods, is given in Section 4.2.

4.1.2 Concrete made with frost-susceptible aggregate—A properly proportioned concrete mixture that has received adequate curing can suffer damage from freezing and thawing if the aggregate (generally, the coarse aggregate) used is susceptible to damage from freezing and thawing. A description of damage in concrete made with aggregates susceptible to freezing and thawing, along with a description of aggregate identification procedures, is given in [Section 4.3](#).

4.2—Frost attack of concrete made with durable aggregates

4.2.1 Description of frost damage

4.2.1.1 Damage at early ages—Concrete in the early stages of hydration ordinarily contains a considerable amount of freezable water, along with little or no tensile strength to resist pressures developed during freezing. Concrete in this condition, which is allowed to freeze, will develop ice lenses approximately parallel to the surface exposed to freezing. Additional ice lenses can develop under coarse aggregates. When the concrete thaws and the hydration continues, the space occupied by the ice lenses will become weak planes that are susceptible to delamination or surface scaling.

4.2.1.2 Damage in cured concrete

4.2.1.2.1 Surface scaling—The most common form of damage from freezing and thawing in hardened concrete is surface scaling. Surface scaling is the loss of paste and mortar from the surface of the concrete. Generally, layers less than 1 mm (0.04 in.) in thickness are lost, but repeated cycles of freezing and thawing can lead to removal of additional material. Scaling is considerably accelerated by the presence of deicing salts. Vehicle traffic or other surface contact can also accelerate scaling by aiding in the removal of loosened material. Consequences of scaling include: change in

appearance, change in surface smoothness and, in severe cases, the loss of concrete cover over reinforcing steel.

4.2.1.2.2 Internal deterioration—Though less common, internal deterioration can have more severe consequences than surface scaling due the loss of structural integrity of the concrete. Internal deterioration manifests itself as a loss of strength in the mortar portion of the concrete. Modern concrete practice has practically eliminated this form of damage from freezing and thawing by requiring a proper air-void system and adequate maturity.

4.2.2 Preventing frost damage in new concrete

4.2.2.1 Protection from early freezing—Concrete should be protected from early freezing by following the procedures and maintaining the minimum temperatures recommended in ACI 306R. After consolidation and finishing, the concrete should be protected from cooling too rapidly by the use of insulated forms, curing blankets, and other procedures described in ACI 306R. Allowing the concrete to cool too rapidly could not only result in early freezing, it could also result in thermal cracking in the concrete (ACI 306R, 308R).

4.2.2.2 Minimum curing before freezing—Maintaining adequate curing conditions, including preventing the excessive loss of moisture and maintaining adequate temperature, will ensure that the concrete has hydrated sufficiently so that the amount of freezable water is substantially reduced. A recommended minimum strength that should be attained before the concrete temperature is allowed to drop below freezing is 3.5 MPa (500 psi) (Powers 1962). Once this strength has been achieved, a single freeze will generally not result in permanent damage to the concrete (ACI 308R). If repeated cycles of freezing and thawing are anticipated, the concrete should be kept warm for a long enough period of time to allow it to develop a compressive strength of at least 25 MPa (3600 psi).

4.2.3 Preventing frost damage by proper design—Much concrete now in service has withstood, for years, the damaging effects of repeated cycles of freezing and thawing. While some of this concrete has remained undamaged because it was never allowed to contain enough freezable water to cause damage from freezing, most of this concrete has remained durable because proper precautions were taken to avoid such damage (Mather 1990). The three most important precautions to provide resistance to freezing and thawing are discussed as follows.

4.2.3.1 Entrained air-void system—Resistance to freezing and thawing of a concrete mixture is substantially improved by incorporating entrained air voids into the concrete. To achieve maximum effectiveness, these air voids should be evenly distributed throughout the paste portion of the concrete. Their spacing should be close enough to prevent the development of sufficient pressures from freezing to fracture the concrete.

Because voids reduce the strength and stiffness of most concrete mixtures, there is a natural tendency to avoid including too much entrained air in concrete. A sufficient amount of entrained air, however, is necessary to provide resistance to freezing and thawing. The specific parameters normally used to evaluate an entrained air-void system along with the generally accepted minimum (or maximum) values are described as follows.

4.2.3.1.1 Spacing factor $\bar{L}$ —Spacing factor $\bar{L}$ is an approximation of the largest distance from anywhere in the cement paste to an air void. The following assumptions are made for this parameter: the voids are spherical and of equal size, and the voids are evenly distributed in a simple cubic lattice throughout the paste (Powers 1949). The method for determining this parameter is described in ASTM C457. The range of spacing factors is generally from 0.1 mm (0.004 in.) or less to approaching 1 mm (0.04 in.) for mixtures that do not contain entrained air. The generally accepted maximum spacing factor value for concrete with good resistance to freezing and thawing is approximately 0.20 mm (0.008 in.).

4.2.3.1.2 Specific surface α —Specific surface α is a measure of the surface area per unit volume of voids. The method for determining this parameter is described in ASTM C457. A basic assumption of specific surface is that all voids are spherical; this makes it a function of average chord length alone (Powers 1949). Specific surface is, therefore, a good indicator of average void size. As average void size goes up, specific surface goes down. Smaller voids provide compliance with the $\bar{L}$ requirement at lower values of air content. The generally accepted value of specific surface for resistance to freezing and thawing is a minimum of 25 mm²/mm³ (630 in.²/in.³).

4.2.3.1.3 Philleo factor F' —Philleo (1955) developed an air-void parameter as a means to eliminate the assumptions made for the spacing factor, namely, that all voids are of equal size and spacing. His equation works on the concept that all voids are randomly sized and distributed and establishes a relationship between the air-void distribution and the percentage of paste that is within a given distance of an air void. Philleo used the work of Willis and Lord (1951) to establish a relationship between air-void chord lengths and voids per unit volume. Besides providing the percentage of paste protected, the equation can provide, alternatively, a value for the distance from an air void within that a given percentage of paste is located. This distance, called the Philleo factor, is often compared with the spacing factor. In actuality, however, the Philleo factor is more sensitive to the actual air-void distribution than air content, paste content, and number of voids that influence the spacing factor.

This parameter has not been widely accepted as a measure of resistance of concrete to freezing and thawing, primarily due to the difficulty in acquiring the data necessary for calculation. These data consist of a record of all chord lengths measured in a linear traverse (ASTM C457). Modern data acquisition techniques permit the collection of these data, and modern data analysis methods allow the use of improved procedures for characterizing the air-void chord distribution (Roberts and Scheiner 1981; Larson et al. 1967). This should allow the determination of the Philleo factor to become more common in the future. While no specific criterion for the maximum F' (the distance for a given percentage of paste protected) has been determined, an examination of a considerable amount of linear traverse (ASTM C457) data for a number of concrete specimens having spacing factors of approximately 0.20 mm (0.008 in.) and specific surface values of approximately 25 mm²/mm³ (630 in.²/in.³)

Table 4.1—Recommended air contents for concrete resistant to freezing and thawing*

Nominal maximum aggregate size, mm (in.)	Average air content, % [†]	
	Severe exposure [‡]	Moderate exposure [§]
9.5 (3/8)	7-1/2	6
12.5 (1/2)	7	5-1/2
19.0 (3/4)	6	5
25.0 (1)	6	4-1/2
37.5 (1-1/2)	5-1/2	4-1/2
50 (2)	5	4
75 (3)	4-1/2	3-1/2

*There is conflicting opinion on whether air contents lower than those given in the table should be permitted for high-strength (more than approximately 40 MPa [6000 psi]) concrete. The committee believes that, where supporting experience and experimental data exist for particular combinations of materials, construction practices, and exposure, the air contents can be reduced by approximately 1%. For nominal maximum aggregate sizes over 37.5 mm (1-1/2 in.), this reduction applies to the fraction of the mixture passing the 37.5 mm (1-1/2 in.) sieves.

[‡]A reasonable tolerance for air content in field construction is $\pm 1-1/2\%$.

[†]Outdoor exposure in a cold climate where the concrete may be in almost continuous contact with moisture before freezing or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, and water tanks.

[§]Outdoor exposure in a cold climate where the concrete will be only occasionally exposed to moisture before freezing and where no deicing salts will be used. Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil or accumulated snow.

^{||}These air contents apply to the whole concrete, as is the case for the preceding aggregate sizes. When testing this concrete, however, aggregate pieces larger than 37.5 mm (1-1/2 in.) are removed by handpicking or wet-sieving, and the air content is determined on the fraction of the mixture passing the 37.5 mm (1-1/2 in.) sieve. (The field tolerance applies to this value.) From this, the air content of the whole mixture is computed.

suggests that a maximum acceptable distance between an air void and 90% of the paste F'_{90} should be approximately 0.04 mm (0.002 in.) (Janssen and Snyder 1993, 1994).

4.2.3.1.4 Air content—The aforementioned air-void parameters, while excellent indicators of the extent of protection from freezing and thawing provided by the air-void system, are difficult to measure in the field for quality-control purposes. Total air content is therefore the parameter generally specified and measured. Total air content includes both the entrained air voids and the larger air voids that are entrapped in the concrete and not removed by consolidation.

The use of an air-entraining admixture complying with ASTM C260 can provide a proper system of entrained air voids when a specified total air content of the concrete mixture is achieved. The actual air content necessary to ensure the production of the necessary air-void system is affected by many parameters: mixing action, workability of the mixture, cement composition, types and amounts of other admixtures, and others. In addition, concrete handling during transport, placing, and finishing can have an effect on the entrained air-void system (Section 4.2.4). Recommendations for air contents for fresh concrete are given in Table 4.1. These recommendations take into account the higher air requirements of concrete mixtures with higher paste contents (smaller nominal maximum aggregate sizes) and the severity of the exposure (higher exposure severity increases the probability of damage from freezing and thawing, and therefore demands a higher level of protection to offset this higher probability of damage). The values shown are general recommendations; local conditions and experience with specific mixtures and procedures could warrant other values.

Air content does not protect from damage due to freezing and thawing; rather, achieving the minimum and maximum values for air-void parameters discussed previously provides the protection.

4.2.3.2 Reducing freezable water—The likelihood of damage from freezing and thawing is reduced by reducing the amount of freezable water in concrete. For conventional mixtures, this has generally been accomplished by the use of a low w/cm (maximum of 0.50 for mild exposure and 0.45 for severe exposure), combined with adequate curing to ensure a minimum compressive strength of approximately 25 MPa (3600 psi) before exposure to repeated cycles of freezing and thawing.

Limiting the w/cm to a specified maximum value has the effect of reducing the amount of freezable water in the cured concrete initially, and requiring a minimum strength before freezing helps ensure that the fractional volume that could be occupied by freezable water in saturated concrete has been adequately reduced by the formation of hydration products.

Modern concrete mixtures may contain admixtures, additives, and SCMs that permit reduction of the amount of freezable water in concrete. Slag cement or pozzolans can refine the pore structure at a given w/cm , resulting in a smaller fraction of the porosity containing freezable water. The use of slag cement or pozzolans has the added beneficial effect of reducing the rate that water can migrate through the concrete. This means that water removed by evaporation or hydration will be replaced more slowly when the concrete is exposed to water.

4.2.3.3 Design details—Physical details that allow for the repeated wetting or the prevention of evaporative drying of concrete surfaces should be avoided. Examples of these include roof driplines, overhanging sidewalks, and driveways to unsealed joints on bridge slabs over support girders. Care should always be taken to provide positive drainage of runoff water from flat concrete areas by methods such as sloping surfaces (minimum 2%). Special precautions for water removal should be taken when runoff is likely to contain deicing salts or other aggressive chemicals.

Consideration should also be given to structures that allow water access to the side away from freezing, such as water tanks. In many cases, the evaporation rate on the outside of the tank will exceed the rate that water can migrate through the tank wall to replenish the evaporation. In these instances, the amount of freezable water in the portions of the concrete that freeze will be accommodated by the air-void system. In some cases, however, water flow to the freezing surface can exceed the evaporation rate, leading to potential damage problems from freezing and thawing. Also, in very cold climates, freezing can extend all the way through the concrete wall, resulting in severe damage. Low rates of moisture movement and consideration of the depth of freezing are necessary in the design of these types of structures (ACI 350).

4.2.4 Preventing frost damage by proper practice—A properly proportioned concrete mixture can still suffer damage from freezing and thawing if either the air-void system or the amounts of freezable water are detrimentally affected during construction. The effects of construction

practices on maintaining the quality of the concrete mixture are described in Sections 4.2.4.1 through 4.2.4.4.

4.2.4.1 Transporting and placing—The air voids that provide protection against freezing-and-thawing damage are produced during the process of mixing the fresh concrete (Whiting and Stark 1983). The air-entraining admixtures stabilize the bubbles that are produced during mixing, but do not generate those bubbles. Therefore, mixing is critical in causing the bubbles to form and in distributing those bubbles throughout the mixture. Upon delivery and discharge of the concrete, the total number of air bubbles, their total volume, and the distribution of bubble sizes depend on mixing action and the degree that the particular combination of air-entraining admixture, other admixtures, water, cement, and aggregates have worked together to generate, stabilize, and retain the bubbles.

The system of air bubbles in the fresh concrete at delivery will be sensitive to many factors, including the type and effectiveness of the mixer, duration of mixing, mixing speed (rpm), haul time, batch size, and general condition of the mixing equipment. The Portland Cement Association has published graphs of the relationship between air content, mixing time, slump, and mixing speed (rpm) (Kosmatka et al. 2002). The aforementioned issues (such as mixing speed, duration of mixing, and batch size) affect the time required for the air-entraining admixture to stabilize bubbles. Fast-acting air-entraining admixtures may perform well even with short mixing periods or brief haul times, whereas more slowly-acting admixtures may perform better with a longer mixing or haul time. The degree that air is incorporated in the fresh concrete during mixing depends on the tumbling or wave-breaking action of the concrete in the mixer itself. This action will depend on the slump of the concrete and on the cleanliness and efficiency of the mixing blades. Air incorporated in the concrete is subsequently subdivided into smaller bubbles by continued mixing and stabilized by the air-entraining admixture, thus minimizing air loss. Immediately before being discharged, the freshly mixed concrete contains air bubbles of a wide distribution of sizes and with varying effectiveness in terms of providing frost resistance, which all have their origin in the engulfment or entrapment of air during the mixing process (Mielenz et al. 1958a,b; Backstrom et al. 1958a,b).

Placing concrete is often accomplished by pumping the concrete from the delivery truck to the formwork. Advances in pumping technology have resulted in substantial improvements in concrete pump capabilities, including increased pumping distances, increased pumping lift heights, and increased delivery rates. These advances have been accompanied by increases in the pressure capacity of the pumps, with pressures in the range of 2 to 3.5 MPa (300 to 500 psi) being fairly common (Cooke 1990). Unfortunately, concerns have developed when the air content measured after pumping was not the same as the air content before pumping. In most cases, the air content had decreased after pumping, but in some instances, it increased (Cooke 1990; Dyer 1991; Whiting and Nagi 1998). The possible loss of the entrained

air can be a major concern for concrete exposed to severe freezing-and-thawing environments.

The pumping process includes a number of activities that could contribute to a change in the air content of the concrete. During pumping, the concrete falls through a grating in the pump hopper, is forced under pressure through a relatively small-diameter pipe, moves through a series of bends, experiences changes in both elevation and pipe material (generally steel to rubber), is then released from pressure as it exits the pipe, and often falls some distance into the formwork. Pumping may also be continuous or interrupted at various time periods. This sequence of events makes it difficult to isolate the mechanism if there is only one that induces the loss (and occasional gain) in air content of air-entrained concrete.

A number of mechanisms have been proposed to explain the change in air content that sometimes occurs when concrete is pumped. During pumping, a vacuum could form in the line, (especially at low pumping rates with a long portion of the pumping being downhill), which would enlarge and remove air voids. Air voids could also be lost when the concrete falls through the grating in the pump hopper or when it falls into the formwork after exiting the pump line (Yingling et al. 1992; Janssen et al. 1995). While changes in the air-void system have been documented in multiple cases (Whiting and Nagi 1998; Janssen et al. 1995; Pleau et al. 1995), evidence of reduced resistance to freezing and thawing has not been identified (Elkey et al. 1993; Whiting and Nagi 1998).

4.2.4.2 Consolidating—Upon discharge of fresh concrete, additional opportunities are present for the further entrapment of air voids in the concrete. In the process of filling forms, it is virtually impossible to avoid the inclusion of air pockets. When such large air pockets form during mixing, the continued mixing action breaks them down to smaller sizes and allows for their distribution in the mixture. No such opportunity exists once the concrete is cast. For this reason, it is necessary to attempt to reduce the number and size of these pockets trapped in the mixture by vibration.

Vibration accomplishes two purposes with regard to the removal of air pockets in fresh concrete. First, vibration can liquefy the concrete in the same manner as earthquakes liquefy certain types of soils. Many of the large, buoyant air pockets that were previously trapped in the semifluid mixture can rise to the surface through the temporarily, yet fully fluid, mixture. Second, vibration imposes a cyclic compression in the concrete that locally increases, then decreases, the water pressure surrounding the air bubbles. This causes the bubbles themselves to compress and decompress at the frequency of the vibrator (Young 1989). Air bubbles break when forced to compress and decompress at a critical frequency that varies with their size—the larger the bubble, the lower the frequency required to break the bubble. Conversely, the higher the frequency of the vibrator, the smaller the bubble that can be broken. Simon et al. (1992) showed that vibration at approximately 6000 VPM begins to break bubbles in the size range of the so-called entrained air. Evidence also indicates that some portion of the air from

broken bubbles escapes, while the balance of the air is reincorporated into other air bubbles in the mixture.

It takes time for the energy imparted by the vibrator to liquefy the surrounding concrete and for the water pressure to build up so as to compress the air bubbles. It also takes time for the bubbles to rise to the surface, to break, or both. For these reasons, short intermittent vibration may have little significant effect on the concrete in general or on the air bubble system. Refer to ACI 309R for guidelines for appropriate vibration procedures.

In summary, vibration of fresh concrete has the effect of refining the system of air voids by encouraging the loss of larger air voids and leaving a larger percentage of the smaller air voids. The exact distinction between the size of bubbles removed and those left in place depends on the mixture and on the frequency, duration, and intensity of the vibration.

4.2.4.3 Finishing—After the concrete is placed and consolidated, it is still possible to modify the air bubbles at the surface of the fresh concrete (and therefore, the air voids in the hardened concrete) through finishing operations. Repeated passes of a finishing tool can force bubbles together, resulting in fewer, but larger, bubbles. Upon close observation at the trailing edge of a bull float, float, or trowel as it is passed over the surface of the concrete, one can see the bursting of air bubbles within the cement paste. (This may be related to the relative velocity of the tool over the surface, the pressure of the tool, or both.) The air content of the concrete at the surface can be reduced by overfinishing, implying that too much manipulation of the surface can reduce resistance to freezing and thawing. This is particularly the case if the surface is finished while the surface is still covered with bleed water or if water has been applied to make it easier to finish. Finishing a wet surface not only weakens that surface, but also makes it more porous. When this is coupled with a localized reduction in air content, resistance to freezing and thawing is likely to be significantly reduced.

4.2.4.4 Curing—Curing is defined as the maintenance of a satisfactory moisture content and temperature in concrete during its early stages so that desired properties may develop (American Concrete Institute 2008; ACI 308R). Overall, resistance to freezing and thawing increases in response to curing as continued curing develops microstructure and reduces the porosity and hydraulic conductivity of the concrete. The result is a concrete that is less likely to become critically saturated. Curing further increases the compressive and tensile strength of the concrete, which increases the resistance to hydraulic pressure, osmotic pressure, and ice accretion pressure. These attributes combine to produce a concrete that is less susceptible to freezing-and-thawing damage. The issue is made more complicated, however, when concrete is cast in weather where there is risk of freezing. In this situation, attempts to cure the concrete to improve its overall durability can conflict with the fact that curing measures can increase the risk of saturating concrete during exposure to early freezing. It is generally accepted that properly air-entrained concrete can sustain one freeze cycle when it has attained a compressive strength of 3.5 MPa (500 psi), and

repeated freezing-and-thawing cycles when saturated at a compressive strength (in-place) of 25 MPa (3600 psi). This means that curing procedures need to be carried out to maintain moisture content to improve the quality of the concrete, as well as measures to prevent early freezing (ACI 306R, 308R).

4.2.5 Preventing frost damage in existing concrete that lacks adequate air-void system—Concrete that lacks an adequate air-void system to protect it from the anticipated exposure conditions is sometimes encountered. Protecting the concrete from damage caused by freezing and thawing then requires that the concrete be kept dry, with an internal relative humidity below approximately 75 to 80%.

4.2.5.1 Sealers—Concrete that can be adequately dried can sometimes be kept dry by sealing the surface of the concrete with some sort of protective barrier system to prevent the re-introduction of moisture. Slab-on-ground and similar construction would also require a vapor retarder beneath the concrete to reduce the movement of water in either liquid or vapor form from entering the concrete from underneath. Details of various protective barrier systems can be found in ACI 515.1R. Vapor retarders for use under slabs are discussed in ACI 302.1R.

In many cases, not all sides of a concrete member are accessible for sealer treatments. Care should be taken so that the sealed surface does not prevent the evaporation of moisture that may have entered from the unsealed surface. An example of such a situation would be the sealing of the top surface of a slab-on-ground. Moisture entering the slab from the bottom side could be prevented from evaporating from the top of the slab by the sealer. The resulting moisture condition could be wetter than if the concrete had never been sealed. Vapor-permeable sealers exist, and their effectiveness in both preventing the intrusion of water while permitting the evaporation of moisture from the sealed surface should be evaluated for application-specific conditions.

4.2.5.2 Drainage and other methods—The moisture level in concrete can often be kept sufficiently low to reduce the possibility of damage due to freezing and thawing if attention is given to the removal of water from the area of the concrete. While methods of moisture reduction are generally application-specific, the examples in Sections 4.2.5.2.1 through 4.2.5.2.3 are presented.

4.2.5.2.1 Drainage—Water allowed to pond on a concrete surface can contribute significantly to the moisture level in the concrete. By providing adequate drainage to prevent the ponding of water, the amount of water absorbed in the concrete can be minimized.

4.2.5.2.2 Maintenance—Good designs that provide for the rapid removal of water from concrete surfaces can be defeated by a lack of maintenance. An example is the ponding of water on a concrete slab being caused by leaves blocking a drain. Proper design should be followed by adequate maintenance.

Other times, improper maintenance or other activities could result in the unnecessary accumulation of water. Snow pushed off of a sidewalk and against a concrete wall could result in the accumulation of moisture in the wall. As the snow against the wall melts, the unmelted snow could serve as a dam, holding the water against the wall. Normally a

surface with little moisture absorption, this vertical wall section could absorb enough of the melted snow to lead to damage from subsequent freezing.

4.2.5.2.3 Redirection of water flow—Downspouts emptying water across a concrete slab can be redirected so that rainwater drainage does not flow across the concrete surface. This can reduce the moisture exposure of the surface enough to reduce damage from freezing and thawing.

4.2.6 Theories for frost damage—Damage in concrete from freezing and thawing occurs as internal damage or surface deterioration, and appears in many forms (Cordon 1966). Historically, the capillary-void system of the cement paste has been the focus of most investigative efforts. There is no consensus regarding mechanisms responsible for damage in cement paste. For example, it has been attributed to hydraulic pressure buildup as water is forced away from the freezing front, to osmotic pressure gradients driving water toward the freezing centers, to vapor pressure potentials, and to combinations of these processes (Powers 1945, 1954, 1955, 1975; Powers and Helmuth 1956; Helmuth 1960; Litvan 1972).

4.2.6.1 Moisture expulsion—Powers, in his early work, proposed that ice nucleated and grew in capillary pores, forcing them to dilate or expel excess water from freezing sites. Elevated hydraulic stresses would arise as water was expelled due to the relatively low permeability of cement paste. Distance from the void boundary, the degree of saturation, and the rate of freezing would influence the magnitude of hydraulic-stress buildup (Cordon 1966; Powers 1945, 1954, 1955).

4.2.6.2 Osmotic pressure—The hydraulic-pressure concept was modified when experiments (Powers and Helmuth 1956; Helmuth 1960; Powers 1975) showed significant evidence that moisture was moving toward, rather than away from, freezing sites. An osmotic pressure gradient, arising from solute rejection as relatively pure ice grows from the pore solution, was then proposed. Continued freezing in pores would increase the solute concentration in the remaining pore solution, hence increasing the osmotic pressure gradient drawing water from capillaries and gel pores. Litvan (1972) proposed a similar theory, also founded on thermodynamic arguments, but cast in terms of vapor-pressure gradients. He recognized that the controlling variables included degree of saturation, length of migration path, rate of freezing, and the presence of entrained-air bubbles. A compromise view emerged; it was suggested that capillaries were subjected to the combined action of the hydraulic pressures described in the first theory and opposing pressure resisting flow in that direction (Powers and Helmuth 1956; Cordon 1966). This conceptual model, in effect, acknowledged that both directions of flow were possible depending upon the circumstances, primarily dictated by concrete properties and cooling rates. Individual researchers recognize both the hydraulic theory alone and the osmotic theory coupled with the hydraulic theory.

4.2.6.3 Ice lens growth—Whereas the 9% volume expansion accompanying the transition of water from liquid to solid may dominate the deterioration of small saturated samples frozen rapidly and cyclically in the laboratory,

progressive ice accretion in cracks during periods of sustained temperatures slightly below 0 °C (32 °F) may generally dominate the degradation of concrete in many environments (Litvan 1978). A theoretical model was developed and proven successful for predicting freezing-and-thawing damage in another porous media: rock (Walder and Hallet 1985; Hallet et al. 1991). In this model, freezing attack in homogeneous porous solids is viewed as occurring in an open system where microcracks are internally pressurized by ice accretion fueled by migration of water; the latter is in turn induced by thermally driven free-energy gradients. This ice accretion model, also known as the segregation ice model, is consistent with the current soil physics understanding of freezing in porous media and parallels Gilpin's (1980) study of freezing effects in soils.

4.2.6.4 Implications of freezing and thawing damage mechanisms—Uncertainty and opposing views of fundamental processes governing the freezing attack on concrete undermine efforts to develop procedures for optimal testing of the resistance to freezing and thawing of concrete. Several testing strategies have been used throughout the last 50 years that involved relatively rapid cycling of freezing-and-thawing events in saturated and dry environments. In North America, a freezing-and-thawing cycling test, ASTM C666/C666M, is used to determine the resistance of concrete mixtures to internal damage. ASTM C672/C672M evaluates scaling deterioration. The latter relies solely on visual classifications, sometimes supplemented with mass loss measurements. These tests, like some of their predecessors, are criticized for not adequately representing typical environmental conditions. The tests cycle between extreme temperatures too rapidly. A procedure developed by the RILEM TC 117 committee, the CDF test, provides greater reproducibility and is more quantitative with respect to scaling measurements (RILEM TC 117-FDC 1996). Hallet et al. (1991) proposed that conventional tests give only limited guidance for understanding the processes governing freezing attack at field sites where the thermal or hydraulic regimes are very different than in the laboratory. Extrapolation from diurnal or more frequent freezing-and-thawing experiments to field conditions should be viewed with particular caution because distinct physical processes may govern each. This situation calls for consideration of diagnostic freezing-and-thawing tests in which both processes can be distinguished and probed systematically. It also suggests that considerably more caution is needed when attempting to relate standard laboratory test results to spacing factors and to other design criteria for effective resistance to freezing-and-thawing under use conditions. Some concrete mixtures pass accepted laboratory tests and do not perform well in the field, while others fail the tests and perform quite satisfactorily in the field. This is probably because the current tests for internal damage and scaling due to freezing and thawing do not address all of the significant variables. Stark (1989) indicated that the potential role of several key factors in freezing-and-thawing damage has not been appreciated; these factors include the magnitude and duration of exposure to sustained temperature and moisture gradients and the

cumulative time of exposure to specified temperature ranges. In addition, advances in high-performance concrete have produced another element of uncertainty in studies of resistance to freezing-and-thawing. With much finer pore systems, the question arises as to whether the conventional wisdom of entraining air and reducing the total freezable water in the pore system will help reduce damage from freezing and thawing in high-performance concrete (Hooton 1993; Pigeon 1994).

4.3—Frost attack of concrete made with nondurable aggregates

Deterioration due to freezing and thawing of properly proportioned, air-entrained concrete made with aggregate susceptible to freezing-and-thawing damage is often referred to as D-cracking. Many types of coarse aggregate have been identified as susceptible to D-cracking, while other sources of the same kind of rock have not been found to be susceptible. The pore structure of the coarse aggregate is thought to be the primary contributing factor to the D-cracking susceptibility of the aggregate. Descriptions of the appearance and development of D-cracking are presented in Section 4.3.1, while the prevention of D-cracking in new construction is covered in Section 4.3.2. A discussion of efforts to mitigate D-cracking in existing construction is given in Section 4.3.3. Theories and mechanisms of D-cracking are covered in detail in Section 4.3.4.

4.3.1 Description of D-cracking

4.3.1.1 General description—D-cracking is characterized by cracks through both the coarse aggregate and the mortar portion of the concrete. Away from the cracks, both the mortar and the coarse aggregate are strong and show no signs of deterioration.

The development of D-cracking requires considerable moisture and repeated cycles of freezing and thawing. As a result, D-cracking usually appears close to joints, cracks, edges, and corners where moisture is available from more than one direction. D-cracking generally appears as a series of cracks approximately parallel to the primary moisture source.

4.3.1.2 Flatwork—The most common appearance of D-cracking is in at-grade flatwork such as highway pavements, parking lots, and sidewalks. These are areas that frequently have readily available moisture from precipitation runoff and also have moisture available from multiple directions at joints and cracks, and on the bottom side of the slab except in cases where there is an effective vapor retarder beneath the slab. Often, the earliest appearance of D-cracking will be at the intersection of transverse and longitudinal joints in a pavement. At this joint intersection, moisture is often available at both vertical joint faces and the horizontal surfaces on both the top and bottom of the slab.

In climates cold enough to freeze all the way through the concrete slab, D-cracking usually starts at the bottom of the concrete slab and progresses to the surface (Schwartz 1987). This is probably due to the greater availability of moisture at the bottoms of slabs. By the time telltale cracks appear on the surface, deterioration can extend 0.5 m (1.5 ft) or more away from the joint.

The appearance of D-cracking in milder climates is somewhat different because the concrete never freezes all the way through. D-cracking in these cases often appears as shallow spalling at joints; closer examination reveals the typical cracks parallel to the joint.

4.3.1.3 Vertical construction—Though much less common, D-cracking can also appear on vertical construction. Construction details, maintenance, or both, that allow the accumulation of moisture against the corners of walls or columns can contribute to the development of D-cracking if susceptible aggregates were used in the construction. An example of such maintenance at a building would be the shoveling of snow off of a sidewalk and depositing it against a concrete foundation wall.

4.3.2 Prevention of D-cracking

4.3.2.1 Role of mixture proportioning—The primary factor in a concrete mixture that contributes to the development of D-cracking is the susceptibility of the coarse aggregate, whereas the air-void system and the *w/cm* have little or no effect (Schwartz 1987; Missouri Highway and Transportation Department 1990). Most coarse aggregates identified as being susceptible to D-cracking are sedimentary rocks, though many sedimentary rocks have not been found to be susceptible to D-cracking. Igneous rocks are generally not considered to be susceptible to D-cracking unless the rocks are quite weathered. Weathered rocks would probably be undesirable for concrete production due to their low strength and likelihood to break down from handling. Most metamorphic rocks have not shown D-cracking susceptibility; however, some partially metamorphosed sedimentary rocks have been identified as D-cracking-susceptible (Stark 1976).

The maximum aggregate size is also important in the development of D-cracking in concrete made with a D-cracking-susceptible aggregate. Numerous studies (Stark and Klieger 1973; Klieger et al. 1974; Stark 1976; Missouri Highway and Transportation Department 1990; Almond and Janssen 1991) have shown that reducing the nominal maximum size of the aggregate reduces the D-cracking susceptibility of a given aggregate. Unfortunately, reducing the nominal maximum aggregate size can have less-desirable side effects, including increased paste demand to maintain workability at a given strength level, increased drying shrinkage potential, and reduced joint load transfer in pavements.

4.3.2.2 Importance of aggregate identification—D-cracking can require a number of years to fully develop, and a considerable amount of concrete from a susceptible source could be placed before a problem is identified. This, combined with natural variability of aggregate sources, leads to the need for identification of D-cracking-susceptible aggregates before their use in concrete exposed to moisture and repeated cycles of freezing and thawing.

D-cracking has been known to exist since the 1930s (Stark and Klieger 1973); wide ranges of tests have been developed to try to identify susceptible aggregates. The most commonly used procedure for identifying D-cracking-susceptible aggregates is ASTM C666/C666M. Concrete specimens made with the aggregate in question are subjected to repeated cycles of freezing and thawing in the laboratory and

are evaluated in terms of either increase in length or decrease in dynamic modulus of elasticity.

4.3.2.3 Aggregate beneficiation—A variety of techniques have been proposed to improve the performance of D-cracking-susceptible aggregates. One of these methods, limiting the nominal maximum size, has already been discussed (Section 4.3.2.1). Schwartz (1987) summarized other methods, including coating the aggregates to prevent their absorption of water, heavy media separation, and blending durable aggregate with a nondurable aggregate to reduce its D-cracking susceptibility. He reported that aggregate size reduction was the most effective method of reducing D-cracking susceptibility.

4.3.3 Mitigation of existing D-cracking—A considerable amount of concrete containing D-cracking-susceptible aggregates has been placed in locations where the concrete can be expected to be exposed to both moisture and repeated cycles of freezing and thawing. This is especially true of concrete pavements. Joint deterioration associated with the development of D-cracking can be expected to significantly reduce the service life of those pavements. The concrete, away from the joints by as little as 0.5 m (1.5 ft), is often unaffected, showing no deterioration or loss of strength.

A typical concrete pavement can have transverse joints 10 m (30 ft) or more apart. With less than 0.5 m (1.5 ft) of D-cracked concrete at each end of a slab, most of the concrete is in good condition with only the slab ends deteriorated. While rehabilitation of the deteriorated concrete near the joints by replacing the deteriorated concrete with a full-depth patch would seem to be a cost-effective method of extending the life of such a concrete pavement, D-cracking often appears at the newly created joints adjacent to the patches in as little as 5 years. Thus, the D-cracking continues as before, only at twice as many joints (Janssen and Snyder 1994).

4.3.3.1 General—Three conditions are necessary for the development of D-cracking: concrete made with D-cracking-susceptible aggregate; moisture; and repeated cycles of freezing and thawing. As the aggregates already in the concrete would be difficult or impossible to treat so as to render them nonsusceptible, D-cracking mitigation should attempt to either prevent freezing and thawing or remove the source of moisture.

4.3.3.2 Preventing freezing—Portland-cement concrete pavements often receive asphalt concrete overlays as rehabilitation treatments to improve the condition of the pavement and extend the life of the pavement. In climates that do not get too cold in winter, freezing in a concrete pavement that contains D-cracking-susceptible aggregates could possibly be prevented by covering the portland-cement concrete with a sufficient thickness of asphaltic concrete. Previous work (Janssen et al. 1986) suggests that the freezing should be almost completely prevented in the concrete to stop the progression of D-cracking; merely decreasing the number of cycles of freezing and thawing with an overlay could actually accelerate the rate of D-cracking. More than 150 mm (6 in.) of asphalt concrete overlay would be required to prevent freezing from occurring at the surface of the concrete pavement for central latitudes of the U.S. Using asphalt concrete overlay to prevent freezing in concrete

made with D-cracking-susceptible aggregates is probably not an effective D-cracking mitigation method for these conditions (Janssen et al. 1986; Janssen and Snyder 1994).

4.3.3.3 Reducing moisture—Replacing 1 m (3 ft) on each end of a concrete slab with D-cracking at the joints gives the appearance of a completely restored concrete pavement. Unfortunately, the patching process produces two new joints that normally show evidence of D-cracking on the old concrete side of the joints within approximately 5 years. If lateral moisture movement at these new joints could be prevented, the rate of D-cracking progression at the new joints could possibly be slowed.

The use of sealers on the cut ends of the existing concrete pavement sections, described previously, before placing the concrete patches, could possibly reduce the lateral movement of moisture into the concrete. This could increase the time period before D-cracking appeared in the patched concrete. This method was attempted in a D-cracked concrete pavement section in Ohio in 1992. Though initial laboratory testing indicated that sealer treatment delayed the resumption of D-cracking (Janssen and Snyder 1994), field monitoring showed that the D-cracking was returning after 6 years (Janssen 2001).

4.3.4 Theories and mechanisms of D-cracking—Theories of damage to concrete from freezing and thawing have already been discussed in [Section 4.2.5](#). With the exception of the role that air-voids play in protecting concrete from damage, these same theories generally apply to D-cracking. This section, therefore, will concentrate on the characteristics of specific aggregates that make them D-cracking susceptible, while other aggregates of the same type are not susceptible.

4.3.4.1 Pore size and size distribution—Kaneuji et al. (1980) observed qualitative correlations between concrete durability and pore size distributions of aggregates. At a constant total pore volume, aggregates with smaller pore sizes result in a lower resistance to freezing and thawing. For aggregates with similar predominating pore sizes, a greater pore volume results in less resistance to freezing and thawing aggregate. By correlating aggregate service records with mercury porosimetry studies, Marks and Dubberke (1982) found that, with one exception, the D-cracking-susceptible aggregates analyzed exhibited a predominance of pore sizes of 0.04 to 0.2 μm (1.6×10^{-6} to 7.9×10^{-6} in.), while aggregates with good to excellent service records had a majority of pores that were larger than the 0.04 to 0.2 μm (1.6×10^{-6} to 7.9×10^{-6} in.) diameter pore sizes.

4.3.4.2 Deicing salt effect—Researchers in Iowa (Dubberke and Marks 1985) noted a reduced resistance to D-cracking for some aggregates when pavements containing the aggregates were exposed to deicing salt. Other aggregates that they examined showed no effect. A possible explanation for this effect of salt exposure is a change in pore structure due to etching of the pore walls by the salt. Such action has been identified for aggregates containing calcite (Gillot 1978). This possible deicing salt effect should be kept in mind when using field performance records to evaluate the D-cracking potential of an aggregate source. Satisfactory performance in a pavement that never received deicing salt

may not ensure the same performances if deicing salts are used on the pavement.

CHAPTER 5—ALKALI-AGGREGATE REACTION

5.1—Introduction

This chapter presents guidelines for minimizing the risk of damaging expansion caused by AAR in concrete construction. Procedures for evaluating aggregates and selecting appropriate measures for controlling expansion are discussed. A comprehensive report on AAR was prepared by ACI Committee 221 (ACI 221.1R) and provides more detailed information on types of reaction, reaction mechanisms, reactive rock types, methods of testing aggregates, and preventive measures.

A wide range of testing procedures for assessing aggregate reactivity has been developed since AAR was first reported in 1940 (Stanton 1940). The reliability of these techniques varies and depends, to some extent, on the nature of the aggregate being tested. Criteria for interpreting test results vary between national standards and also within states or provinces, with different limits being adopted by various local agencies and state or provincial authorities.

Methods for controlling expansion due to AAR also vary on a regional basis. Many specifications do not permit the use of reactive aggregates and, consequently, preventive methods are not an issue. Generally accepted preventive methods include the use of low-alkali cement, limiting the alkali content of the concrete, the use of natural pozzolans, fly ash, slag, or silica fume, or any combination of these methods.

The flowchart in [Fig. 5.1](#) shows the general sequence of testing and decision-making that could be adopted when dealing with potential alkali-aggregate reactivity. If there is sufficient evidence of satisfactory field performance, aggregates should be accepted for use without AAR testing ([Section 5.3.1](#)). In the absence of such data, however, aggregates should be subjected to suitable testing procedures to establish the degree of reactivity. If the results from laboratory tests do not indicate a potential for AAR, aggregates can be used without any precautionary measures. Aggregates that demonstrate potential for alkali-silica reaction (ASR) can also be used in concrete provided that suitable measures are implemented to control the risk of expansion. Alkali-carbonate reactive aggregates are normally avoided, as it has proven difficult (or uneconomic) to control expansion with such materials.

5.2—Types of reactions

Two types of AARs are recognized: 1) alkali-carbonate reaction (ACR); and 2) ASR. ACR involves certain argillaceous dolomitic limestones. Known sources of alkali-reactive carbonate rocks are restricted to a few quarries in North America, mainly in Ontario and Virginia. ACR involves a reaction between an alkali source and certain calcium-magnesium carbonate rocks (dolomites). It distinctly differs from the ASR reaction that results from a reaction between an alkali source and certain forms of reactive silica that can originate from some types of siliceous aggregate. ASR can occur in limestone aggregates that contain siliceous components. [Table 5.1](#) presents a list of several of the

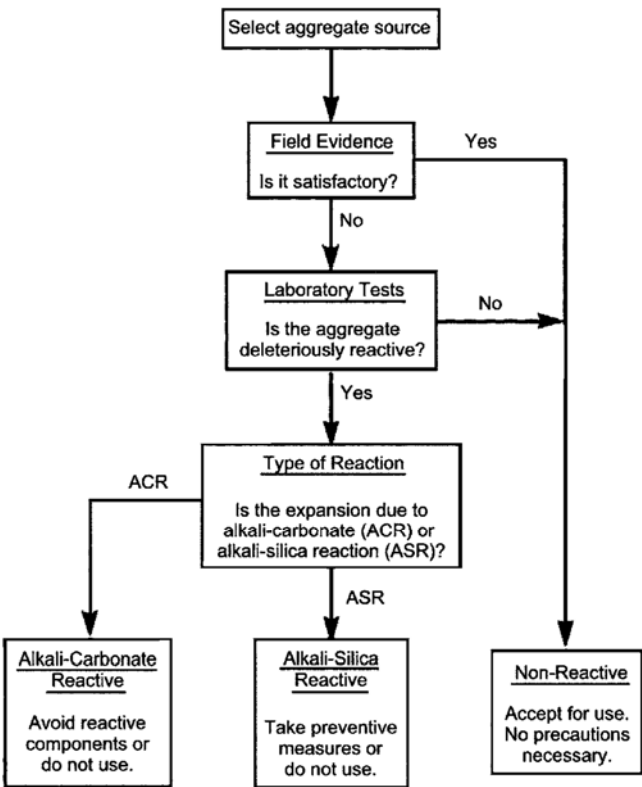


Fig. 5.1—Flowchart showing general sequence of testing and decision-making when dealing with alkali-aggregate activity.

common reactive rock types and mineral forms that are susceptible to ASR.

ASR is far more widespread than ACR, and is further subdivided into two categories: 1) reactions involving poorly crystalline or metastable silica materials; and 2) reactions involving certain varieties of quartz. The main distinctions between these two categories, from an engineering perspective, are the time to the onset of expansion and cracking and the perceived duration of the reaction in the field. Reactions in the first category, which are sometimes referred to as classical ASR, are characterized by a relatively short time to the onset of cracking and limited duration (cracking usually occurs within 5 to 10 years, and the reaction is often exhausted within 10 to 20 years), whereas the manifestation of reactions involving quartz minerals usually takes much longer, and the reaction may continue for many decades.

5.3—Evaluating aggregates for potential alkali-aggregate reactivity

5.3.1 *Field performance*—A history of satisfactory field performance is possibly the best method for evaluating the potential for an aggregate to cause AAR. A number of factors have to be considered when analyzing field performance data. These include:

- The cement content and the alkali content of the cement should be the same, or higher, in the field concrete as is proposed in the new structure;
- The concrete examined should be at least 10 years old;
- The exposure conditions of the field concrete should be

Table 5.1—Some examples of rock types and minerals susceptible to alkali-silica reaction (ASR)

Reactive rocks	Reactive minerals
Shale	Opal
Sandstone	Tridymite
Silicified carbonate rock	Cristobalite
Chert	Volcanic glass
Flint	Cryptocrystalline (or microcrystalline) quartz
Quartzite	Strained quartz
Quartz-arenite	
Gneiss	
Argillite	
Granite	
Greywacke	
Siltstone	
Arenite	
Arkose	
Hornfels	

- at least as severe as those in the proposed structure;
- A petrographic examination should be conducted to demonstrate that the aggregate in the structure is sufficiently similar to that under investigation in the absence of conclusive documentation;
- The possibility of pozzolans or slag having been used should be considered; and
- The *w/cm* of the concrete should be considered, as it may affect performance.

Provided that satisfactory field performance can be demonstrated, the aggregate can be used in concrete, with no further testing for AAR.

5.3.2 *Petrographic examination (ASTM C295)*—A petrographic examination should be the first step in assessing the suitability of a particular aggregate source for use in concrete construction. Petrography is a powerful tool that yields a wide range of information regarding the physical, chemical, and mineralogical characteristics of an aggregate, including the presence of rocks or mineral phases that are known to cause deleterious reaction in concrete.

In some cases, a petrographic examination may produce sufficient evidence to reject an aggregate on the basis of potential alkali reactivity or require that suitable preventive measures be implemented. Generally, the examination cannot predict whether the type and distribution of reactive minerals present will cause damaging expansion in concrete, and further laboratory testing is usually required for this purpose. Results of petrographic examination may form the basis for directing the laboratory test program in terms of selecting the type and sequence of tests and any relevant evaluation criteria.

The reliability of petrographic examination for screening aggregates for potential reactivity is strongly dependent on the skills and experience of the individual petrographer. There have been cases where aggregates that were accepted for use on the basis of results of petrographic examination have been later implicated in AAR. This is not necessarily the result of incorrect material classification, but more due to a failure to recognize certain minerals as potentially reactive

(Rogers and Hooton 1991). Furthermore, the reactive constituents of some rocks may not be readily identified by optical microscopy and, therefore, will not be detected by such procedures.

5.3.3 Laboratory tests to identify alkali-silica reactive aggregates

5.3.3.1 Mortar bar test (ASTM C227)—This test method was developed to assess the potential alkali-silica reactivity of cement-aggregate combinations, and is not recommended for evaluating alkali-carbonate reactive aggregates. Mortar bars (aggregate/cement = 2.25) are monitored during storage at 38 °C (100 °F) in storage containers designed to provide a high humidity. The alkali content of the cement is not specified, but the cement should be selected to have the highest alkali content representative of that generally intended for use with the aggregate.

The standard specification for concrete aggregates (ASTM C33) regards as reactive those cement-aggregate combinations that have an expansion greater than 0.05% at 3 months or 0.10% at 6 months. More stringent limits have been applied by other agencies; for example, the U.S. Bureau of Reclamation requires specific combinations to expand by less than 0.05% at 6 months, or 0.10% at 12 months.

There are a number of limitations associated with this test. It is difficult to attain a high level of humidity in the containers without promoting the leaching of alkalis from the mortar bars. Alkali leaching can lead to an underestimation of the expansion of certain combinations, especially if the reactive component of the aggregate reacts relatively slowly. This is seen with argillites and greywackes where the reactive component is microcrystalline quartz (Grattan-Bellew 1989; Rogers and Hooton 1991). This limitation may be overcome by storing bars over water without wicks and by raising the equivalent alkali content or Na_2O_e (note: $\text{Na}_2\text{O}_e = \text{Na}_2\text{O} + 0.658 \text{ K}_2\text{O}$) of the cement to 1.0%, although expansion may still be less than 0.10% at 6 months with some reactive aggregates.

The test provides direct information regarding the specific cement-aggregate combination tested. When an aggregate is to be evaluated for general use, the cement should be of the highest alkali content involved in such use.

5.3.3.2 Quick chemical method (ASTM C289)—In this test, a sample of the aggregate, crushed to pass a 300 μm (No. 50) sieve and retained on a 150 μm (No. 100) sieve, is immersed in 1M NaOH solution for 24 hours, and the solution is analyzed. The amount of silica dissolved and the reduction in alkalinity are plotted on a graph that classifies the aggregate as innocuous, potentially reactive, or deleterious.

The reliability of this test in detecting aggregate reactivity is poor for a number of reasons. Other mineral phases present in the aggregate may interfere with the test result and lead to a reduction in the dissolved silica due to precipitation (Bérubé and Fournier 1992a). Furthermore, reactive phases may be lost during crushing and sieving operations. Thus, reactive aggregate may actually plot as innocuous based on the test results. In contrast, the high surface area and temperature used in this test dissolves some siliceous mineral phases that are stable in the conditions that prevail in concrete. This results

in aggregates with good field performance being classified as deleterious by this test (Bérubé and Fournier 1993).

5.3.3.3 Accelerated mortar bar test (ASTM C1260)—This test is essentially the same as that developed at the National Building Research Institute (NBRI) by Oberholster and Davies (1986). The method involves fabrication of ASTM C227 mortar bars. After the molds are stripped at 1 day, the bars are placed in water that is then heated to 80 °C (176 °F). On the second day, the initial length readings are made (at 80 °C [176 °F]), and the bars are transferred to a 1M NaOH solution at 80 °C (176 °F), where they are stored. Expansions are monitored for a period of 14 days in the NaOH solution, at which point the test is normally completed.

The expansions obtained in this rapid test are generally comparable or higher than those obtained after 1 year in the ASTM C227 test (38 °C [100 °F] at 100% humidity) (Oberholster and Davies 1986; Hooton and Rogers 1989). The test has been successfully used to identify alkali-silica reactive aggregates from across Canada (Grattan-Bellew 1989; Hooton and Rogers 1989, 1992; Bérubé and Fournier 1992b; Durand et al. 1990; Hooton 1991) and the United States (Stark et al. 1993), but is not considered suitable for evaluating reactive alkali-carbonate rocks.

Interpretation of results is not simple, and various expansion criteria have been suggested. Bérubé and Fournier (1992b) proposed different expansion limits depending on the type of rock being tested. These investigators suggest a limit of 0.10% expansion (after 14 days in 1M NaOH) for quarried silicate and siliceous carbonate aggregates, and a higher limit of 0.20% for natural sands and gravels. Many aggregates with satisfactory field performance, however, produce expansion values in excess of 0.25% in this test (Bérubé and Fournier 1992b). Consequently, aggregates should not be rejected on the basis of this test unless petrographic examination confirms that the material is similar to known deleteriously reactive aggregates.

5.3.3.4 Concrete prism test (ASTM C1293)—In this test, the concrete mixture is proportioned with a cement content of $420 \pm 10 \text{ kg/m}^3$ ($708 \pm 17 \text{ lb/yd}^3$) using a portland cement with an equivalent alkali content (Na_2O_e) of $0.90 \pm 0.10\%$. NaOH is then added to the mixing water to provide the equivalent of 1.25% alkali cement that results in a total alkali loading of the concrete of 5.25 kg/m^3 (8.85 lb/yd^3). This high alkali level is necessary to detect the expansion of slowly reactive rocks such as greywacke and argillites (Magni et al. 1987). Prisms are stored at 38 °C (100 °F), and the expansion is monitored for at least 1 year.

An expansion limit of 0.04% at 1 year is currently specified in both Canada and France (Thomas et al. 1997) and is recommended in the Appendix to ASTM C1293. This expansion is approximately where cracking and signs of distress are first observed on the prisms. It also relates well in terms of observed field performance. The Canadian Standard (CSA A23.2-27A and A23.2-28A) carries a caveat stating that lower expansion limits may be appropriate for assessing aggregates for use in critical structures, such as nuclear containment, or dimensionally sensitive structures, such as

hydraulic dams, where small expansions may result in relatively large movements.

This test method has distinct advantages over the mortar bar test (ASTM C227), as coarse aggregates can be tested without crushing to sand sizes. Furthermore, the larger test specimen reduces the effect of alkali leaching. The 12-month time period is necessary unless the temperature is raised or other changes are made to accelerate expansion; however, further acceleration of the test may have undesirable side effects.

5.3.4—Laboratory tests to identify reactive alkali-carbonate rock aggregates

5.3.4.1 Rock cylinder method (ASTM C586)—In this method, cylinders (or prisms) cut from the rock are immersed in a solution of 1M NaOH at room temperature (after having attained dimensional stability in distilled water), and the expansion is monitored for at least 1 month. Expansions in excess of 0.10% at 1 month are generally taken to indicate potentially deleterious chemical reaction between the alkalis and the rock. This test does not provide an indication of the potential for expansion in concrete, and further testing of the aggregate in concrete (ASTM C1105) is recommended if the rock cylinder expansion exceeds 0.10%.

5.3.4.2 Chemical composition (CSA A23.2-26A)—The determination of potential alkali-carbonate reactivity by chemical composition (CSA A23.2-26A) involves analysis for CaO, MgO, and Al₂O₃. Relatively pure limestones or dolomites are readily identified by composition, and tend not to be alkali-carbonate reactive. These aggregates may require further testing for alkali-silica reactivity. Potentially reactive dolomitic limestones plot in the potentially expansive area of a CaO/MgO versus Al₂O₃ plot, and such aggregates should be tested in the concrete prism test. This test has helped to remove some of the difficulty in identifying reactive dolomitic limestones by petrographic examination.

5.3.4.3 Concrete prism test (ASTM C1105)—This test is similar to the concrete prism test used to assess alkali-silica reactivity (Section 5.3.3.4), except for differences in storage temperature and the alkali content of the concrete. ASTM C1105 requires the testing to be carried out using a specific concrete mixture, with specimens being stored at 23 °C (73 °F). Potentially deleterious reactivity is indicated if the expansion exceeds 0.015% at 3 months, 0.025% at 6 months, or 0.030% at 1 year.

Users of this test should recognize that the test yields information about the specific cement-aggregate combination tested, and the absence of significant expansion in this test does not necessarily indicate the aggregate to be nonreactive. For instance, deleterious expansion may occur if the aggregate was used in concrete with higher alkali content. The Canadian Standards Association (CSA A23.2-14A) requires potentially alkali-carbonate reactive rocks to be tested in concrete prisms under the same conditions as those used for ASR (that is, with 5.25 kg/m³ [8.85 lb/yd³] Na₂O_e and stored at 38 °C [100 °F]). Furthermore, the same expansion criteria are applied; aggregates are deemed to be reactive if the expansion exceeds 0.040% at 12 months, with no criteria for earlier test results. The Canadian test is aimed at establishing the reactivity

of the rock and not assessing the performance of a particular cement-aggregate combination.

5.3.5 Other tests for AAR—There are a wide range of test methods available for evaluating the reactivity of aggregates or determining the potential for expansion of cement-aggregate combinations. Some of these test methods are discussed in ACI 221.1R.

5.3.6 Developing testing strategies and criteria—ASTM covers a number of test methods related to AAR testing. Not all of these test methods are suitable for all types of aggregates, and agencies need to establish strategies for minimizing the risk of AAR that are based on locally available materials and environmental conditions. The best approach is likely to vary between different geographic regions, and possibly even for different structures in the same vicinity. In 1994, the Canadian Standards Association (1994) published a guideline (CSA A23.1-94, Appendix B) to deal with the problem of alkali-aggregate reactivity that combines a number of the tests described previously; the approach has been summarized by Thomas et al. (1997). This guideline was published as a “Standard Practice” in 2000, and has since been republished (CSA A23.1-27A). The Canadian general procedure for evaluating aggregates and determining appropriate precautionary measures is shown in Fig. 5.2. This flowchart has been simplified for clarity, and serves only to illustrate the sequence of testing and decision making involved. It is not intended for use instead of the detailed procedures described in the standard.

The CSA guideline permits aggregates to be used without laboratory testing provided that satisfactory field performance can be demonstrated. In the absence of sufficient field data, however, aggregates are subjected to a sequence of laboratory tests. Quarried carbonate rocks are first evaluated on the basis of chemical composition. If the composition reveals the aggregate to be relatively pure limestone or dolomite, the aggregate is deemed not to be potentially alkali-carbonate reactive, and is subsequently tested in the same manner as potentially alkali-silica-reactive aggregates. If the chemical composition classifies the quarried carbonate rock as a potentially reactive dolomitic limestone, however, it should be tested in a concrete prism test. If the aggregate is not deleteriously expansive by this test (that is, expansion < 0.040% at 1 year), it may be accepted for use with no further precautions. If this is not the case (that is, expansion ≥ 0.040% at 1 year), then the concrete prism is subjected to petrographic examination to determine the nature of the reaction. If the expansion is due to reaction of siliceous components of the aggregate, the aggregate may still be used for concrete provided that suitable precautionary measures are taken (those specified for ASR aggregates). If results of petrographic examination show evidence of ACR, the aggregate should be rejected or the reactive phases avoided by selective quarrying or adequately reduced by beneficiation.

All other aggregates, except quarried carbonates, are subjected to ASR testing only. The first test is a petrographic examination carried out in accordance with ASTM C295. Although an aggregate may be selected for use purely on the basis of petrographic examination (Fig. 5.2), this is not

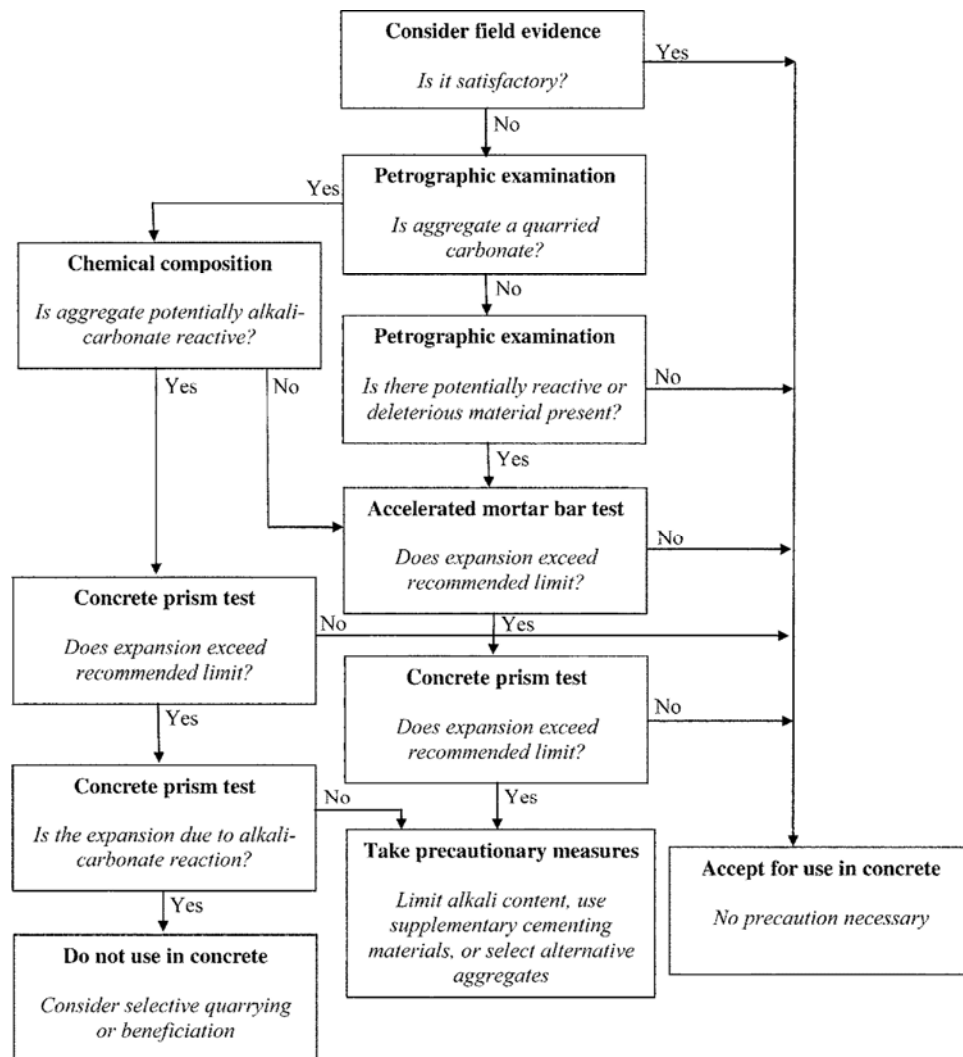


Fig. 5.2—Canadian general procedure for evaluating aggregates and determining risk of AAR damage.

common practice. In the absence of satisfactory field data, aggregates will normally be subjected to the accelerated mortar bar test even if potentially reactive or deleterious material is not detected by petrographic examination. If the aggregate does not exhibit excessive expansion in the accelerated mortar bar test, it may be accepted for use in concrete. If expansions in the mortar bar test are considered potentially deleterious, however, then the concrete prism test is suggested to confirm the results before an aggregate is either rejected or conditions are put on its use. If the aggregate does not exhibit excessive expansion in the prism test, it may be accepted for use in concrete with no limitations, regardless of its performance in the mortar bar test. Aggregates that expand excessively in the concrete prism test may still be used in concrete, provided that suitable precautionary measures are adopted.

5.4—Preventive measures

Methods for minimizing the risk of damaging ASR include one or more of the following:

- Use of nonreactive aggregate;
- Use of low-alkali cement;

- Limits on the alkali content of the concrete;
- Use of pozzolan or slag; and
- Use of suitable chemical admixtures.

These approaches are discussed in the following sections. For alkali-carbonate rock reactive aggregates, avoidance or reduction in proportion of the reactive phases is the only recommended practice. The other methods listed, though proven effective with alkali-silica-reactive aggregates, may not be a sufficient remedy for the alkali-carbonate rock reaction (Rogers and Hooton 1992).

5.4.1 Use of nonreactive aggregate—This approach is perhaps the most obvious and certain way of avoiding damaging reaction in concrete structures. Nonreactive aggregates are not available in many locations, and importing nonreactive material may not be economically viable. Furthermore, AAR has occurred in a number of cases where prior testing of the aggregate indicated the aggregates were not deleteriously reactive. Methods of testing aggregates for reactivity have increased in severity, and acceptance criteria have become more stringent with the passage of time to reflect the increasing number of aggregates implicated in field cases of AAR. Adoption of existing testing practices,

however, does not guarantee that aggregates will give satisfactory performance in every situation. Consequently, even if aggregates are found to be not deleteriously reactive, further precautions are frequently taken if circumstances demand. Such circumstances may include prestigious (or critical) structures, aggressive environments (external source of alkalis such as seawater or deicing salts), high cement contents, or extended service life.

5.4.2 Limiting alkali content of concrete—Stanton's (1940) work on AAR indicated that expansive reaction is unlikely to occur when the alkali content of the cement is below 0.60% Na_2O_e . This value has become the accepted maximum limit for cement to be used with reactive aggregates in the United States, and appears in ASTM C150 as an optional limit. This criterion, however, takes no account of the cement content of the concrete, which, together with the cement alkali content, governs the total alkali content of concrete and is considered to be a more accurate index of potential reactivity. Some national specifications recognize this fact by specifying a maximum alkali level in the concrete; this limit is reported (Nixon and Sims 1992) to range from 2.5 to 4.5 kg/m^3 (4.21 to 7.58 lb/yd^3) Na_2O_e . In some countries, the limit may vary, depending on the reactivity of the aggregate (Oberholster 1994). In Canada (CSA A23.2-27A), the limit ranges between 1.8 and 3.0 kg/m^3 (3.03 to 5.05 lb/yd^3) Na_2O_e .

The use of low-alkali cement and limitation of the alkali content in concrete is not a sufficient safeguard in all cases. Stark (1980) reported damaging AAR in highway structures constructed using cements with alkalis in the range 0.45 to 0.57% Na_2O_e ; the reactivity of certain aggregates with low-alkali cements was confirmed in laboratory mortar bar expansion tests. Lane (1987) reported that some aggregates, classed as innocuous after 6 or 12 months in the ASTM C227 mortar bar test with low-alkali cement (0.54% Na_2O_e), showed delayed expansion and cracking after longer periods. Thomas (1996) reported evidence of ASR in a number of hydraulic structures with alkali contents below 3 kg/m^3 (5.05 lb/yd^3) Na_2O_e .

Aggregates that are not normally reactive when used in concrete with low-alkali cement may be deleteriously reactive in concrete of higher alkali content. This may occur through alkali concentration caused by drying gradients, alkali release from aggregates, or the ingress of alkalis from external sources (deicing salts or seawater). Stark (1978) reported increases in soluble alkali from 1.1 to 3.6 kg/m^3 (1.85 to 6.07 lb/yd^3) Na_2O_e close to the surface of some highway structures. Migration of alkalis due to moisture, temperature, and electrical gradients has also been demonstrated by laboratory studies (Nixon et al. 1979; Zhong and Hooton 1993).

A number of workers have demonstrated that many aggregates contain alkalis that may be leached out into the concrete pore solution, thereby increasing the risk of AAR (Stark 1980; Stark and Bhatti 1986; Way and Cole 1982; Van Aardt and Visser 1977; Thomas et al. 1992). Stark and

Bhatti (1986) reported that, in extreme circumstances, some aggregates release alkalis equivalent to 10% of the portland cement content.

Alkalis may penetrate concrete from external sources such as brackish water, sulfate-bearing groundwater, seawater, or deicing salts. Nixon et al. (1987) showed that seawater (or NaCl solutions) present in the mixing water elevates the hydroxyl-ion concentration and increases the amount of expansion of concrete. Oberholster (1992) showed that the expansion of large concrete blocks exposed to saltwater spray may be doubled compared with the same blocks exposed to tapwater spray. In addition, studies in Denmark (Chatterji et al. 1987) have shown that exposure to NaCl solution (and other alkali salts) can cause considerable expansion and cracking in concrete.

5.4.3 Use of supplementary cementitious materials—Pozzolans or slags can be used as a component of concrete (as a cement replacement or an addition) or as a constituent of blended cements. Blended cements are discussed in detail in ASTM C1157 and C595. Although many national specifications and guidelines for minimizing AAR permit the use of slag or pozzolans, such as fly ash and silica fume, with potentially reactive aggregate, there is conflicting evidence regarding the efficacy of these materials in this role, and this is reflected in the lack of consensus advice. Much of the controversy is centered on the alkalis in the pozzolan or slag and whether they are potentially available for reaction. A review of extensive literature dealing with the effect of these materials on AAR is not within the scope of this document. Some guidance on using these materials with alkali-silica-reactive aggregates is presented herein.

5.4.3.1 Use of fly ash and slag—A summary of national specifications covering the use of fly ash and slag to combat ASR in concrete is shown in Table 5.2 (Nixon and Sims 1992). When fly ash or slag is being used with reactive aggregates, most specifications control the total alkali content of the concrete, typically within the range 2.5 to 4.5 kg/m^3 (4.21 to 7.58 lb/yd^3) Na_2O_e . There is considerable variation in the treatment of the alkalis from pozzolan or slag when calculating the total alkali content of the concrete. A number of countries assume both fly ash and slag behave as an inert diluent and do not contribute alkalis. Other countries use either the available alkali content (ASTM C311) or a proportion of the total alkali content of the pozzolan or slag. The values of one-sixth and one-half the total alkalis from fly ash and slag, respectively, have been adopted by a number of countries based on experimental studies by Hobbs (1989). There is conflicting advice within the United Kingdom with the Building Research Establishment adopting Hobb's recommendations and the Concrete Society recommending the use of the water-soluble alkalis. The water-soluble alkalis in fly ash and slag are usually very low (less than 0.1% Na_2O_e), and are typically much less than one-sixth or one-half of the total alkalis. The available alkali, determined in accordance with

Table 5.2—Summary of national specifications for fly ash and slag to combat ASR (Nixon and Sims 1992)

Country	Maximum alkali, kg/m ³ (lb/ft ³)	Alkali		Limit on alkalis		Minimum proportions		Other tests/requirements
		Fly ash	Slag	Fly ash	Slag	Fly ash	Slag	
Belgium	3.0 (0.19)	—	—	—	—	—	—	—
Canada (draft)	3.0 (0.19)	0	0	Maximum 4.5% total and 0.5% water-soluble	1.0% maximum total	20 to 30%	50%	Concrete prism test (0.04% maximum at 2 years)
Denmark	Varies	0	0	—	—	—	—	—
France	3.0 to 3.5* (0.19 to 0.22)	1/6 total	1/2 total	—	—	—	—	—
Ireland	4.0 or 4.5† (0.25 or 0.28)	1/6 total	1/2 total	—	—	—	—	—
Japan	3.0 (0.19)	0	0	—	—	—	—	—
New Zealand	2.5 (0.16)	ASTM C311		—	—	—	—	ASTM C441
South Africa	2.0 to 4.5‡ (0.12 to 0.28)	ASTM C311		1.5% maximum (ASTM C311)		20%	40%	NBRI (ASTM C1260) (maximum 0.10% at 14 days)
UK (BRE)	3.0 (0.19)	1/6 total	1/2 total	—	—	—	—	—
UK (Concrete Society)	3.0 (0.19)	Water-soluble	Water-soluble	—	—	25%	25%	—
United States	—	ASTM C311		1.5% maximum (ASTM C311)		—	—	ASTM C441 (maximum 0.02% at 14 days)
U.S.S.R.	3.0 (0.19)	ASTM C311		1.5 to 3.0% maximum (ASTM C311)		—	—	—

*Depends on information about cement variability.

†4.5 kg/m³ (0.28 lb/ft³) used for carboniferous chert.

‡Depends on aggregate source.

ASTM C311, generally gives proportions higher than one-sixth and one-half total alkali for fly ash and slag, respectively.

Technical guidance for using fly ash and/or slag to combat ASR has emerged from Canada, where after considerable debate and consideration of the available literature, a consensus was reached that these materials could be assumed effectively not to contribute any alkalis. Limits, however, are placed on the total alkali content of the fly ash and slag, and minimum replacement levels are specified in CSA A23.2-27A. The minimum replacement level required depends on the reactivity of the aggregate, the service life and size of the structural element, the exposure environment, and the composition of the fly ash or slag. For fly ashes with total alkali contents less than 3.0% Na₂O_e, the minimum levels of replacement are in the range from 15 to 25% for low-calcium Type F fly ashes (< 8% CaO) and from 20 to 30% for moderate-calcium fly ash (8 to 20% CaO). If the alkali content of the fly ash is between 3.0 to 4.5% Na₂O_e, the minimum replacement level must be increased. For slag with a total alkali content less than 1.0% Na₂O_e, the minimum level of replacement is in the range from 25 to 50%. Fly ashes with alkali contents in excess of 4.5% Na₂O_e or with calcium contents greater than 20% CaO, or slags with alkali contents in excess of 1.0% Na₂O_e, cannot be used with reactive aggregates unless their efficacy is demonstrated by performance testing in accordance with CSA A23.2-28A.

A number of countries permit the use of blended slag cement as an alternative to using low-alkali portland cement for controlling AAR. In such cases, the permissible alkali content of the slag cement may vary with the proportion of the slag, as shown in Table 5.3.

The specifications for using fly ash and slag vary considerably between countries, and even within countries (such as the United Kingdom and the United States, and

Table 5.3—Permissible alkali content of slag cement with various percentages of slag

Country	Slag content (% binder)	Maximum total alkali (% Na ₂ O _e)
Belgium	< 50	0.9
	≥ 50	1.1
France	60 ≤ and < 80	1.1
	≥ 80	2.0
Germany	50 ≤ and < 65	1.1
	≥ 65	2.0
UK	≥ 50	1.1

different state regulations within the United States, respectively). This variation reflects not only the national differences in concrete practice and material properties, but also results from the lack of consensus concerning the role of these materials in AARs.

5.4.3.2 Use of silica fume—Early research data indicated silica fume to be highly effective in controlling expansion of concrete due to ASR, with levels of 10% or less being sufficient to suppress damaging expansion in mortar bars with reactive aggregate from Iceland (Asgeirsson and Gudmundsson 1979) and South Africa (Oberholster and Westra 1981). This phenomenon is readily ascribed to the marked reduction in pore solution alkalinity observed when silica fume is combined with portland cement (Diamond 1983; Page and Vennesland 1983).

The efficacy of lower levels of silica fume in controlling expansion in the long term has been questioned by a number of workers. Perry and Gillott (1985) showed 10% silica fume to substantially delay, but not eliminate, expansion with opaline aggregate. Kawamura et al. (1987) showed that 15% replacement may not be enough for some silica fumes used with opal. Work with cristobalite also showed that levels of silica fume substantially higher than 10% may be required to

eliminate expansion in the long term (Dunster et al. 1990; Alasali 1989). Opal exhibits marked pessimum behavior and cristobalite reacts at relatively low alkali contents, however, and findings from studies using such aggregates may not be appropriate to concrete containing less-reactive forms of silica (Bérubé and Duchesne 1992).

Other investigations in Canada with commercial reactive aggregates have further demonstrated the potential for deleterious expansion in concrete with 10% silica fume (Duchesne and Bérubé 1994; Fournier et al. 1995). Silica fume may provide a respite from damaging reaction for periods of 2 years or more in concrete prisms stored at 38 °C (100 °F), but the evidence suggests that the limit of 0.04% may eventually be exceeded with a number of aggregates when high-alkali cement is used.

There is a wide range of data available from expansion studies, but synthesis of the reported findings is complicated by the considerable variation in test methods and materials used by various investigators. Sources of variation include:

- Properties of the silica fume used and effectiveness of its dispersion in the concrete;
- Nature of the reactive aggregate;
- Alkali content of the portland cement;
- Mixture proportions;
- Type of specimen (dimensions, concrete, or mortar);
- Storage conditions; and
- Duration of testing.

Despite these differences, a number of general trends can be discerned from the literature (Thomas and Bleszynski 2001):

- Expansion decreases with increasing silica fume content, although pessimum behavior has been observed in some studies at levels of replacement of 5% silica fume;
- There are insufficient data to allow the effect of silica fume composition to be evaluated. Silica fume with low SiO_2 or unusually high Na_2O_e are not as effective in controlling expansion;
- At normal levels of silica fume (5 to 10%), expansion increases with the alkali content of the cement or the total alkali content of the concrete;
- The efficacy of silica fume is influenced by the nature of the reactive aggregate, and silica fume appears to be less able to control expansion with highly reactive cristobalite or opal; and
- Silica fume retards the rate of expansion; the effect becomes more marked at higher replacement levels.

The Icelandic construction industry has been using silica fume with high-alkali cement (approximately 1.5% Na_2O_e) and highly reactive aggregate for housing concrete since 1979. To date, there are no reported incidences of ASR in such concrete (Gudmundsson and Olafsson 1996).

In South Africa, a minimum replacement level of 15% silica fume is recommended for controlling ASR (Oberholster 1994). At lower replacement levels, the active alkali content of the cement-silica fume blend should be controlled to meet the alkali content limit for the specific aggregate being used. The active alkalies of the silica fume may be determined using ASTM C311 for available alkalies or by calculation, assuming 30% of the total alkalies are active.

In CSA A23.2-27A, the minimum level of silica fume required to control expansion varies depending on the reactivity of the aggregate, the alkali content of the concrete, the service life and size of the structural element, and the exposure environment. Silica fume with alkali contents in excess of 1.0% Na_2O_e cannot be used unless the efficacy of the silica fume is demonstrated with the job aggregate using performance testing in accordance with CSA A23.2-28A to demonstrate the effectiveness of SCMs and lithium-based admixtures to prevent ASR in concrete.

5.4.3.3 Use of natural pozzolans—The term “natural pozzolans” is used to cover various types of naturally occurring reactive forms of silica (volcanic ash) and material processed to produce such silica (calcined clay or shale, or metakaolin). Some reactive aggregates may be ground to produce a material suitable for use as a pozzolanic admixture for concrete. The use of pozzolans in concrete is described in ACI 232.1R. Chemical and physical properties of raw and calcined natural pozzolans for use as SCMs are specified in ASTM C618.

In the first published article on ASR, Stanton (1940) reported that expansion due to the reaction could be reduced by the use of pozzolanic cement containing finely ground shale or by replacement of 25% of high-alkali portland cement with pumicite. The pumicite replacement was more effective in reducing expansion than an equivalent replacement with Ottawa sand, and the question was asked whether such materials have a role beyond mere dilution of cement alkalies. Subsequent tests by Stanton (1950) confirmed the beneficial effect of a wide range of natural pozzolans. The first major use of a natural pozzolan to control ASR dates back to the 1940s when calcined siliceous shale (Puente shale) was used in the Davis Dam together with low-alkali cement (Gilliland and Moran 1949).

Metakaolin, a natural pozzolan manufactured by the thermal activation of kaolinitic clay currently produced in the United States, and sources of similar material manufactured in the United Kingdom, are reported to be effective in controlling ASR expansion (Walters and Jones 1991; Jones et al. 1992).

Pepper and Mather (1959) tested a wide range of SCMs and concluded that they were all effective in controlling ASR expansion provided that sufficient amounts of the admixture were used. Natural pozzolans vary widely in composition, and the level required to eliminate expansion should be determined by testing either in the ASTM C441 pyrex mortar bar test or, preferably, in concrete with the job aggregate (using ASTM C1293).

5.4.3.4 Tests for evaluating effect of pozzolans and slag on ASR—The pyrex mortar bar test of ASTM C441 is a test method that has been commonly used for evaluating the efficacy of pozzolans and slag in controlling expansion due to ASR. This test method was developed in the 1940s as a method for assessing the suitability of pozzolans for use in concrete containing reactive aggregate, such as specification for siliceous admixtures for the Davis Dam (Gilliland and Moran 1949). Early tests by the U.S. Army Corps of Engineers and U.S. Bureau of Reclamation indicated that fly ash and slag were less effective than highly siliceous natural

pozzolans, and had to be used in proportions exceeding 40% to be effective as defined by ASTM C441 (Buck et al. 1953; Pepper 1964; Blanks 1950). Since then, numerous other workers have used this test to evaluate the performance of pozzolans and slag.

In the current version of the ASTM C441 test, the 14-day expansion of mortar bars (stored at 38 °C [100 °F]) made with high-alkali cement (0.95 to 1.05% Na₂O_e) and 25% fly ash (by volume) or 50% slag is compared with that of control bars (cement only), and the percentage reduction due to the pozzolan or slag is calculated. Alternatively, the materials and replacement levels to be used in the actual job may be used. ASTM C618 (specification for natural pozzolans and fly ash) requires that the expansion of the test mixture (regardless of alkali content of the cement used) be no greater than the expansion of a low-alkali control. SCMs meeting this requirement are considered to be as effective as the low-alkali cement control for mitigating ASR. It is assumed that the percentage of pozzolan used in practice will be equal to or greater than that used in the test mixture, and that the alkali content of the field cement used will not exceed the alkali content of the test cement by more than 0.05%. ASTM C989 (specification for slag) does not contain a requirement relating to ASR, but suggests, in a nonmandatory appendix, the use of the ASTM C441 with the 14-day expansion reduced by 75% of control or kept below 0.020% when using project materials. Early versions of the test required slag to be used at replacement levels of 20% by volume. The criteria used to assess pozzolans or slag in this test have been criticized as too conservative by a number of workers (Klieger and Gebler 1987; Kennerley 1988; Kennerley et al. 1981; Sturup et al. 1983).

The potential for silica fume to reduce ASR expansion is apparent if used at the specified 25% by volume replacement in tests using ASTM C441, with shrinkage often being observed after the normal 14-day testing period (Popovic et al. 1984). Perry and Gillott (1985) used various levels of replacement and found that 10% was effective in reducing the 14-day expansion by more than 75% compared with control (this was the acceptance criteria at the time). Other workers have confirmed the ability of 10% silica fume (Rasheeduzzafar and Hussain 1991; Hooton 1993) or less (Bérubé and Duchesne 1992) to meet this criterion. Perry and Gillott (1985), however, observed continued expansion of the silica fume specimens beyond 14 days and questioned the reliability of short-term testing by this method.

Results from tests with Pyrex glass over the past 40 years have shown the effect of fly ash and slag to vary considerably between studies. The effect of fly ash has been shown to depend variously on its alkali content, calcium content, pozzolanicity, and fineness. The only consensus from the literature is that the effectiveness of fly ash and slag increases as the level of replacement increases and that all fly ashes and slags can be used to control reaction provided that they are used in sufficient quantity. Pepper and Mather (1959) reported the effectiveness of a pozzolan or slag was related to fineness, alkali release, and the amount of silica dissolved.

Pyrex glass is extremely sensitive to test conditions (surface area, alkali content, and temperature) and contains significant quantities of alkalis that may be released into the pore solution. Furthermore, Pyrex reacts very rapidly, producing damaging reaction in just a few days. Consequently, determining the role of pozzolans and slag and, especially, the role of SCMs alkalis, is complicated by the use of Pyrex. There has been increased concern over the validity of using this method (Hobbs 1989) because results from the test do not correlate well with data from concrete tests using natural aggregates (Bérubé and Duchesne 1992). Generally, the replacement level required to limit expansion in the Pyrex mortar bar test is significantly higher than required to limit expansion in concrete containing natural reactive aggregates, because few commercial aggregates are as reactive as Pyrex glass.

Significant interest has been shown in modified versions of the accelerated mortar bar test aimed specifically at evaluating pozzolans and slag (CSA A23.2-25A and ASTM C1567), which involves immersion of mortar specimens in 1M NaOH at 80 °C (176 °F) (Davies and Oberholster 1987; Bérubé and Duchesne 1992; Shayan 1992). Reasonable correlations with other test methods have been established, although this accelerated test usually requires a higher level of SCMs to control deleterious expansion compared with standard concrete tests. There has also been interest in the adaptation of autoclave expansion tests for use with SCMs (Criaud et al. 1992; Duchesne and Bérubé 1992). These tests seem promising and have shown reliable correlations with long-term concrete prism expansion tests.

The most reliable way of assessing the effect of pozzolans and slag on ASR expansion by laboratory testing is undoubtedly by expansion testing of concrete mixtures, based on testing conditions similar to ASTM C1293. Unfortunately, the test may take 2 years to yield satisfactory data for fly ash and slag, and even longer periods may be required for silica fume. Reasonable correlations have been reported when comparing the results of the concrete prism test (2-year expansion limit of 0.04% in ASTM C1293) and the accelerated mortar bar test (14-day expansion limit of 0.10% in ASTM C1567) (Thomas and Innis 1999; Thomas et al. 2007).

Several organizations have developed testing protocols and guides to determine if pozzolans, slag, or combinations of these materials will control ASR. The guides use a combination of mortar bar and concrete prism tests to evaluate the potential reactivity of concrete mixtures (AASHTO 2001; Portland Cement Association 1998; Farney and Kerkhoff 2007; CSA A23.2-28A).

5.4.4 Use of chemical admixtures—Chemical admixture use to inhibit the ASR has not been widely employed by the construction industry. These include lithium salts and other salts such as barium salts, sodium silica fluoride, and alkyl alkoxy silane.

5.4.4.1 Lithium salts—McCoy and Caldwell (1951) reported on the ability of lithium compounds (LiF, LiCl and Li₂CO₃) to control ASR, but the use of lithium has not been adopted by the construction industry, probably as a consequence of its relatively high cost. A number of

researchers, however, have shown renewed interest in the use of lithium since the late 1980s; a major research project (including field trials) was completed under the auspices of the Strategic Highway Research Program (Stark et al. 1993).

The level of lithium required to control deleterious expansion varies depending on the alkali content of the concrete and the nature of the reactive aggregate. Generally, research has demonstrated that $Li/(Na + K)$ molar ratios in the range of 0.60 to 1.00 are sufficient to suppress expansion (Sakaguchi et al. 1989; Stark et al. 1993); however, caution is advised because insufficient lithium may actually increase expansion (Stark et al. 1993), and the efficacy of lithium varies depending on the nature of the reactive aggregate. Several documents provide guidance on using lithium admixtures to control ASR (AASHTO 2001; Folliard et al. 2006).

5.4.4.2 Other chemical admixtures—Other chemical compounds were found to reduce expansion due to ASR; these include various barium salts (Hansen 1960), sodium silicofluoride, and alkyl alkoxy silane (Ohama et al. 1989). A wide range of compounds was studied by Hudec and Larbi (1989), but the results are largely inconclusive. Further research is required to confirm the efficacy of various admixtures and to elucidate their role in controlling ASR before such admixtures are recommended for commercial applications.

CHAPTER 6—CHEMICAL ATTACK

6.1—Introduction

Concrete will perform satisfactorily when exposed to various atmospheric conditions, to most waters and soils containing aggressive chemicals, and to many other kinds of chemical exposure. There are, however, some chemical environments that will shorten the useful life of even the best concrete unless specific measures are taken. An understanding of these conditions permits measures to be taken to prevent deterioration or reduce the rate at which it takes place.

Concrete is rarely, if ever, attacked by solid, dry chemicals. To produce a significant attack on concrete, aggressive chemicals should be in solution and above some minimum concentration. Concrete that is subjected to aggressive solutions under pressure on one side is more vulnerable than otherwise because the pressures tend to force the aggressive solution into the concrete.

Comprehensive tables were prepared by ACI Committee 515 (515.1R) and the Portland Cement Association (2001) that give the effects of many chemicals on concrete. Biczok (1972) gave a detailed discussion of the deteriorating effect of chemicals on concrete, including data from both Europe and the United States.

The effects of some common chemicals on the deterioration of concrete are summarized in Table 6.1. Provided that due care has been taken in selection of the concrete materials and proportioning of the concrete mixture, the most important factors that influence the ability of concrete to resist deterioration are shown in Table 6.2. Therefore, Table 6.1 should be considered as only a preliminary guide.

Major areas of concern are exposure to sulfates, seawater, and salt from seawater, acids, and carbonation. These areas of concern are discussed in Sections 6.2 through 6.6.

6.2—Chemical sulfate attack by sulfate from sources external to concrete

6.2.1 Occurrence—Naturally occurring sulfates of sodium, potassium, calcium, or magnesium, that can attack hardened concrete, are sometimes found in soil or dissolved in groundwater adjacent to concrete structures.

Sulfate salts in solution enter the concrete and attack the cementing materials. If evaporation takes place from a surface exposed to air, the sulfate ions can concentrate near that surface and increase the potential for causing deterioration. Sulfate attack has occurred at various locations throughout the world, and is a particular problem in arid areas, such as the northern Great Plains and parts of the western United States (Bellport 1968; Harboe 1982; Reading 1975, 1982; U.S. Bureau of Reclamation 1975; Verbeck 1968); the prairie provinces of Canada (Hamilton and Handegord 1968; Hurst 1968; Price and Peterson 1968); London, England (Bessey and Lea 1953); Oslo, Norway (Bastiansen et al. 1957); and the Middle East (French and Poole 1976).

The water used in concrete cooling towers can also be a potential source of sulfate attack because of the gradual buildup of sulfates due to evaporation, particularly where such systems use relatively small amounts of makeup water to replace evaporated water or water loss otherwise in the system. Sulfate ions can also be present in fill that contains industrial waste products, such as slags from iron processing, cinders, and groundwater that leaches these materials.

Seawater and coastal soil soaked with seawater constitute a special type of exposure. Recommendations for concrete exposed to seawater are in Section 6.3.

6.2.2 Mechanisms—The two best-recognized chemical consequences of sulfate attack on concrete components are the formation of ettringite (calcium aluminate trisulfate 32 hydrate, $CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$) and gypsum (calcium sulfate dihydrate, $CaSO_4 \cdot 2H_2O$). The formation of ettringite can result in an increase in solid volume, leading to expansion and cracking. The formation of gypsum can lead to softening and loss of concrete strength. The presence of ettringite or gypsum in concrete, however, is not in itself an adequate indication of sulfate attack; evidence of sulfate attack should be verified by petrographic and chemical analyses. When the attacking sulfate solution contains magnesium sulfate, brucite (magnesium hydroxide, $Mg(OH)_2$) is produced in addition to ettringite and gypsum. Some of the sulfate-related processes can damage concrete without expansion. For example, concrete subjected to soluble sulfates can suffer softening of the paste matrix or an increase in the overall porosity, either of which diminishes durability.

Publications discussing these mechanisms in detail include Lea (1971), Hewlett (1998), Mehta (1976, 1992),

*Many of these substances occur as minerals, and the mineral names are often used in reports of sulfate attack. The following is a list of such names and their general composition:

anhydrite	$CaSO_4$	thenardite	Na_2SO_4
bassanite	$CaSO_4 \cdot 1/2H_2O$	mirabilite	$Na_2SO_4 \cdot 10H_2O$
gypsum	$CaSO_4 \cdot 2H_2O$	arcanite	K_2SO_4
kieserite	$MgSO_4 \cdot H_2O$	glauberite	$Na_2Ca(SO_4)_2$
epsomite	$MgSO_4 \cdot 7H_2O$	langbeinite	$K_2Mg_2(SO_4)_3$
thaumasite	$Ca_3Si(CO_3)(SO_4)(OH)_6 \cdot 12H_2O$		

Table 6.1—Effect of commonly used chemicals on concrete

Rate of attack at ambient temperature	Inorganic acids	Organic acids	Alkaline solutions	Salt solutions	Miscellaneous
Rapid	Hydrochloric Nitric Sulfuric	Acetic Formic Lactic	—	Aluminum chloride	—
Moderate	Phosphoric	Tannic	Sodium hydroxide* > 20%	Ammonium nitrate Ammonium sulfate Sodium sulfate Magnesium sulfate Calcium sulfate	Bromine (gas) Sulfite liquor
Slow	Carbonic	—	Sodium hydroxide* 10 to 20% Sodium hypochlorite	Ammonium chloride Magnesium chloride Sodium cyanide	Chlorine (gas) Seawater Soft water
Negligible	—	Oxalic Tartaric	Sodium hydroxide* < 10% Sodium hypochlorite Ammonium hydroxide	Calcium chloride Sodium chloride Zinc nitrate Sodium dichromate	Ammonia (liquid)

*The effect of potassium hydroxide is similar to that of sodium hydroxide.

DePuy (1994), Taylor (1997), Skalny et al. (1998), Thomas and Skalny (2006), and Naik et al. (2006). Publications with particular emphasis on permeability and the ability of concrete to resist ingress and movement of water include Reinhardt (1997), Hearn et al. (1994), Hearn and Young (1999), Diamond (1998), and Diamond and Lee (1999).

6.2.3 Recommendations—Protection against sulfate attack is obtained by using concrete that retards the ingress and movement of water and concrete-making ingredients appropriate for producing concrete having the needed sulfate resistance. The ingress and movement of water are reduced by lowering the w/cm . Care should be taken to ensure that the concrete is designed and constructed to minimize shrinkage cracking. Air entrainment is beneficial if it is accompanied by a reduction in the w/cm (Verbeck 1968). Proper placement, compaction, finishing, and curing of concrete are essential to minimize the ingress and movement of water that is the carrier of the aggressive salts. Recommended procedures for these are found in ACI 304R, 302.1R, 308R, 305R, and 306R.

The sulfate resistance of portland cement generally decreases with an increase in its calculated tricalcium-aluminate (C_3A) content (Mather 1968). ASTM C150 permits the use of Type V sulfate-resisting cement and C_3A with a maximum limit of 5%, and Type II moderately sulfate-resisting cement and C_3A limited to 8%. There is also some evidence that the alumina in the aluminoferrite phase of portland cement can participate in sulfate attack. Therefore, ASTM C150 states that the $C_4AF + 2C_3A$ in Type V cement should not exceed 25% unless the alternate requirement based on the use of the performance test (ASTM C452) is invoked. In the case of Type V cement, the sulfate-expansion test (ASTM C452) can be used instead of the chemical requirements (Mather 1978). The use of ASTM C1012 is discussed by Patzias (1991).

Table 6.3 provides recommendations for various degrees of potential exposure. These recommendations are designed to protect against concrete distress from sulfate from sources external to the concrete, such as adjacent soil and groundwater.

Recommendations for the maximum w/cm and the type of cementitious material for concrete that will be exposed to sulfates in soil or groundwater are given in **Table 6.3**. Both

Table 6.2—Factors influencing chemical attack on concrete

Factors that accelerate or aggravate attack	Factors that mitigate or delay attack
1. High porosity due to: - High water absorption - Permeability - Voids	1. Dense concrete achieved by: - Proper mixture proportioning* - Reduced unit water content - Increased cementitious material content - Air entrainment - Adequate consolidation - Effective curing†
2. Cracks and separations due to: - Stress concentrations - Thermal shock	2. Reduced tensile stress in concrete by:‡ - Using tensile reinforcement of adequate size, correctly located - Inclusion of pozzolan (to reduce temperature rise) - Provision of adequate contraction joints
3. Leaching and liquid penetration due to: - Flowing liquid§ - Ponding - Hydraulic pressure	3. Structural design: - To minimize areas of contact and turbulence - Provision of membranes and protective-barrier system(s) to reduce penetration

*The mixture proportions and the initial mixing and processing of fresh concrete determine its homogeneity and density.

†Poor curing procedures result in flaws and cracks.

‡Resistance to cracking depends on strength and strain capacity.

§Movement of water-carrying deleterious substances increases reactions that depend on both the quantity and velocity of flow.

||Concrete that will be frequently exposed to chemicals known to produce rapid deterioration should be protected with a chemically resistant protective-barrier system.

of these recommendations are important. Limiting only the type of cementitious material is not adequate for satisfactory resistance to sulfate attack (Kalousek et al. 1976).

The field conditions of concrete exposed to sulfate are numerous and variable. The aggressiveness of the conditions depends on soil saturation, water movement, ambient temperature and humidity, concentration of sulfate, and type of sulfate or combination of sulfates involved. Depending on the aforementioned variables, solutions containing calcium sulfate are generally less aggressive than solutions of sodium sulfate, which is generally less aggressive than magnesium sulfate. **Table 6.3** provides criteria that should maximize the service life of concrete subjected to the more aggressive exposure conditions.

Table 6.3—Requirements to protect against damage to concrete by sulfate attack from external sources of sulfate

Severity of potential exposure	Water-soluble sulfate (SO ₄) in soil, % by mass*	Sulfate (SO ₄) [*] in water, ppm	w/cm by mass, max. ^{†‡}	Cementitious material requirements
Class 0 exposure	0.00 to 0.10	0 to 150	No special requirements for sulfate resistance	No special requirements for sulfate resistance
Class 1 exposure	> 0.10 and < 0.20	> 150 and < 1500	0.50 [‡]	C150 Type II or equivalent [§]
Class 2 exposure	0.20 to < 2.0	1500 to < 10,000	0.45 [‡]	C150 Type V or equivalent [§]
Class 3 exposure	2.0 or greater	10,000 or greater	0.40 [‡]	C150 Type V plus pozzolan or slag [§]
Seawater exposure	—	—	See Section 6.4	See Section 6.4

*Sulfate expressed as SO₄ is related to sulfate expressed as SO₃, as given in reports of chemical analysis of portland cements as follows: SO₃% × 1.2 = SO₄%.

[†]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, if it is lower than that stated in Table 6.3 of 201.2R.

[‡]Values applicable to normalweight concrete. They are also applicable to structural lightweight concrete except that the maximum w/cm ratios 0.50, 0.45, and 0.40 should be replaced by specified 28-day compressive strengths of 26, 29, and 33 MPa (3750, 4250, and 4750 psi), respectively.

[§]For Class 1 exposure, equivalents are described in Sections 6.2.5, 6.2.6, and [6.2.9](#). For Class 2 exposure, equivalents are described in Sections 6.2.5, 6.2.7, and [6.2.9](#). For Class 3 exposure, pozzolan and slag recommendations are described in Sections 6.2.5, 6.2.8, and [6.2.9](#).

Portland-cement concrete can be also be attacked by acidic solutions, such as sulfuric acid. Information on acid attack is provided in [Section 6.5](#).

6.2.4 Sampling and testing to determine potential sulfate exposure—To assess the severity of the potential exposure of concrete to detrimental amounts of sulfate, representative samples should be taken of water that might reach the concrete or of soil that might be leached by water moving to the concrete. A procedure for making a water extract of soil samples for sulfate analysis is given in [Appendix A](#). The extract should be analyzed for sulfate by a method suitable to the concentration of sulfate in the extract solution, such as the photometer methods used in ASTM C1580. If the amount of sulfate determined in the first analysis is outside of the optimum concentration range for the analytical procedure used, the extract solution should be either concentrated or diluted to bring the sulfate content within the range appropriate to the analytical method, and the analysis should be repeated on the modified extract solution.

6.2.5 Material qualification of pozzolans and slag for sulfate-resistance enhancement—Tests 1 year in duration are necessary to establish the ability of pozzolans and slag to enhance sulfate resistance. Once this material property has been established for specific materials, proposed mixtures using them can be evaluated for Class 1 and 2 exposures using the 6-month criteria in Sections 6.2.6 and 6.2.7.

Fly ashes, natural pozzolans, silica fumes, and slags may be qualified for sulfate resistance by demonstrating an expansion ≤ 0.10% in 1 year when tested individually with portland cement by ASTM C1012 in the following mixtures.

For fly ash or natural pozzolan, the portland cement portion of the test mixture should consist of cement with Bogue-calculated C₃A of not less than 7%. The fly ash or natural pozzolan proportion should be between 25 and 35% by mass, calculated as percentage by mass of the total cementitious material.

For silica fume, the portland cement portion of the test mixture should consist of a cement with Bogue-calculated C₃A of not less than 7%. The silica fume proportion should be between 7 and 15% by mass, calculated as percentage by mass of the total cementitious material.

For slag, the portland cement portion of the test mixture should consist of a cement with Bogue-calculated C₃A of not

less than 7%. The C₃A should be calculated for the sum of the portland cement plus calcium sulfate in the cement. Some processing additions, if present in sufficient proportions, can distort the calculated Bogue values. Formulas for calculating Bogue compounds may be found in ASTM C150. The slag proportion should be between 40 and 70% by mass, calculated as a percentage by mass of the total cementitious material.

Material qualification tests should be based on passing results from two samples taken at times a few weeks apart. The qualifying test data should be no older than 1 year from the date of test completion.

The reported calcium-oxide content analyzed in accordance with ASTM C114 of the fly ash used in the project should be no more than 2.0 percentage points greater than that of the fly ash used in qualifying test mixtures. The reported aluminum-oxide content analyzed in accordance with ASTM C114 of the slag used in the project should be no more than 2.0 percentage points higher than that of the slag used in qualifying test mixtures.

6.2.6 Type II equivalent for Class 1 exposure

- ASTM C150 Type III cement with the optional limit of 8% maximum C₃A; ASTM C595 Type IS(MS), Type IP(MS), Type IS-A(MS), or Type IP-A(MS); ASTM C1157 Type MS; or
- Any blend of portland cement of any type meeting ASTM C150 or C1157 with fly ash or natural pozzolan meeting ASTM C618, silica fume meeting ASTM C1240, or slag meeting ASTM C989 that meets the following requirement when tested in accordance with ASTM C1012:

$$\text{Expansion} \leq 0.10\% \text{ at 6 months}$$

Any fly ash, natural pozzolan, silica fume, or slag used should be previously qualified in accordance with Section 6.2.5.

6.2.7 Type V equivalent for Class 2 exposure

- ASTM C150 Type III cement with the optional limit of 5% maximum C₃A or ASTM C150 cement of any type having expansion at 14 days no greater than 0.040% when tested by ASTM C452 or ASTM C1157 Type HS; or
- Any blend of portland cement of any type meeting

ASTM C150 or C1157 with fly ash or natural pozzolan meeting ASTM C618, silica fume meeting ASTM C1240, or slag meeting ASTM C989 that meets the following requirement when tested in accordance with ASTM C1012:

Expansion $\leq 0.05\%$ at 6 months

Any fly ash, natural pozzolan, silica fume, or slag used should be previously qualified in accordance with [Section 6.2.5](#) for a test of only 6 months to be acceptable.

If one or more of the fly ash, natural pozzolan, silica fume, or slag has not been qualified in accordance with [Section 6.2.5](#), then 1-year tests should be performed on the proposed combination and the expansion should comply with the following limit:

Expansion $\leq 0.10\%$ at 1 year

6.2.8 Class 3 exposure—Any blend of portland cement meeting ASTM C150 Type V or ASTM C1157 Type HS with fly ash or natural pozzolan meeting ASTM C618, silica fume meeting ASTM C1240, or slag meeting ASTM C989 that meets the following requirement when tested in accordance with ASTM C1012:

Expansion $\leq 0.10\%$ at 18 months

6.2.9 Proportions and uniformity of pozzolans and slag—The proportion of fly ash, natural pozzolan, silica fume, or slag used in the project mixture (in relation to the amount of portland cement) should be the same as that used in the test mixture prepared to meet the recommendations of [Section 6.2.6](#), [6.2.7](#), or [6.2.8](#). In blends with portland cement containing only one blending material, such as fly ash, natural pozzolan, silica fume, or slag, the proportion of fly ash or natural pozzolan can generally be expected to be in the range of 20 to 50% by mass of the total cementitious material. Similarly, the proportion of silica fume can be expected to be in the range of 7 to 15% by mass of the total cementitious material, and the proportion of slag can be expected to be in the range of 40 to 70% by mass of the total cementitious material. When more than one blending material, such as fly ash, natural pozzolan, silica fume, or slag, or combinations of these, is used in a blend, the individual proportions of the pozzolan, silica fume, or slag, or combinations of these may be less than these values.

The uniformity of the fly ash or slag used in the project should be within the following of that used in the mixtures tested to meet the recommendations of [Section 6.2.6](#), [6.2.7](#), or [6.2.8](#):

- *Fly ash*—reported calcium-oxide analyzed in accordance with ASTM C114 content no more than 2.0 percentage points higher than that of the fly ash used in the test mixture; and
- *Slag*—reported aluminum-oxide content analyzed in accordance with ASTM C114 no more than 2.0 percentage points higher than that of the slag used in the test mixture.

The portland cement used in the project should have a Bogue C_3A value no higher than that used in the mixtures tested to meet the recommendations of [Section 6.2.6](#), [6.2.7](#), or [6.2.8](#).

Studies have shown that some pozzolans and ground-granulated iron blast-furnace slags, used either in blended cement or added separately to the concrete in the mixer, considerably increase the life expectancy of concrete in sulfate exposure. Many slags and pozzolans significantly reduce the permeability of concrete (Bakker 1980; Mehta 1981). They also combine with the alkalis and calcium hydroxide released during the hydration of the cement (Vanden Bosch 1980; Roy and Idorn 1982; Idorn and Roy 1986), reducing the potential for gypsum formation (Biczok 1972; Lea 1971; Mehta 1976; Kalousek et al. 1972).

[Table 6.3](#) requires a suitable pozzolan or slag along with Type V cement in Class 3 exposures. Research indicates that some pozzolans and slags are effective in improving the sulfate resistance of concrete made with Types I and II cement (ACI 232.2R, 233R, and 234R). Some pozzolans, especially some Class C fly ashes, decrease the sulfate resistance of mortars that are used (Mather 1981b, 1982; Shashiprakash and Thomas 2001). Research indicates that good results are obtained when the pozzolan is a fly ash that meets the requirements of ASTM C618 Class F (Dikeou 1975; Dunstan 1976). Slag should meet requirements of ASTM C989.

In concrete made with non-sulfate-resisting cements, calcium chloride reduces resistance to attack by sulfate (U.S. Bureau of Reclamation 1975), and its use should be prohibited in concrete exposed to sulfate (Class I or greater exposure). If Type V cement is used, however, it is not harmful to use calcium chloride in normally acceptable amounts as an accelerating admixture to mitigate the effects of cold weather (Mather 1992). If corrosion is a concern, calcium chloride should not be added because it can induce and accelerate corrosion of embedded metal, such as reinforcing steel and aluminum conduit.

6.3—Physical salt attack

Field examples have been cited (Reading 1975; Tuthill 1978; Haynes and O'Neill 1994; Haynes et al. 1996) where deterioration has occurred by physical action of salts from groundwater containing sodium sulfate, sodium carbonate, and sodium chloride. The mechanism of the attack is not fully understood, but discussions of possible mechanisms were presented in Hansen (1963), Folliard and Sandberg (1994), Haynes and O'Neil (1994), Haynes et al. (1996), Marchand and Skalny (1999), and Haynes et al. (2008). The mechanism for sodium or magnesium sulfate physical attack may be similar to that used in the Brard test (Schaffer 1932), which is the basis of ASTM C88. The damage typically occurs at exposed surfaces of moist concrete that is in contact with soils containing the above salts. Once dissolved, the ions may transport through the concrete, and subsequently, concentrate and precipitate at the exposed surface. The distress in surface scaling is similar in appearance to freezing-and-thawing damage. Loss of exposed concrete is progressive, and continued exposure, caused by repeated humidity or temperature cycling, can lead to total disintegration.

of low-quality concrete. Numerous cycles of dehydration and rehydration of the salts caused by temperature cycling accelerate this deterioration.

The problem can be mitigated with measures that minimize the movement of water in the concrete. While air entrainment can also be helpful, it is not a substitute for an adequately low w/cm concrete for reducing the rate of moisture movement in concrete. Haynes et al. (1996) recommend a maximum w/cm of 0.45, along with a pozzolan for improved durability. Adequate curing of the concrete is also an important preventive measure. Vapor barriers and adequate drainage of water away from the concrete are also recommended to reduce moisture ingress into the concrete. This group of measures is considered more effective in protecting concrete from this distress than the use of any specific type of cement or admixture.

6.4—Seawater exposure

6.4.1 Seawater in various locations in the world has a range of concentration of total salts; it is more concentrated in some areas than in others. The proportions of the constituents of seawater salts, however, are essentially constant. The concentration is lower in the colder and temperate regions than in the warm seas, and it is especially high in shallow coastal areas with high evaporation rates.

Where concrete structures are placed on reclaimed coastal areas with foundations below saline groundwater level, capillary suction and evaporation may cause super-saturation and crystallization in the concrete above ground. This can result in chemical sulfate attack, physical salt attack, or both. In addition, aggravated corrosion of embedded steel can be induced by the chloride in seawater. In tropical climates, these combined deleterious effects may cause severe defects in concrete in the course of very few years.

6.4.2 The reaction of mature concrete with sulfates in seawater is similar to that with sulfates in fresh water or leached from soils, but the effects are different (Mather 1966). The concentration of sulfates in seawater can be increased to high levels by capillary action and evaporation under extreme climatic conditions. The presence of chloride ions, however, alters the extent and nature of the chemical reaction so that less expansion is produced by a cement of a given C_3A content than would be expected of the same cement in a freshwater exposure where the water has the same sulfate ion content. Concrete made with portland cement having C_3A contents as high as 10% may have proven satisfactory for continuous immersion in seawater, provided that the permeability of the concrete is low (Browne 1980). The U.S. Army Corps of Engineers (1984) permits and the Portland Cement Concrete Association recommends up to 10% calculated C_3A for concrete that will be permanently submerged in seawater if the w/cm is kept below 0.45 by mass.

Verbeck (1968) and Regourd et. al (1980) showed, however, that there may be a considerable difference between the calculated and the measured clinker composition of cement, especially as far as C_3A and C_3AF are concerned. Therefore, the interrelation between the measured C_3A content and the seawater resistance may be equally uncertain.

6.4.3 The requirement for low permeability is essential not only to delay the effects of sulfate attack, but also to afford adequate protection to reinforcement with the minimum concrete cover as recommended by ACI 357.1R for exposure to seawater. The required low permeability is attained by using concrete with a low w/cm that is well consolidated and adequately cured.

The permeability of concrete made with appropriate amounts of suitable ground blast-furnace slag or pozzolan can be as low as 1/10 or 1/100 that of comparable concrete of equal strength made without slag or pozzolan (Bakker 1980). The satisfactory performance of concrete containing ground slag in a marine environment has been described (Mather 1981a; Vanden Bosch 1980; Lea 1971). Concrete made with combinations of cement and microsilica, and with combinations of cement, slag, and microsilica, also have lower permeability and good performance in seawater exposure (Fidjestøl and Frearson 1994).

Concrete should be designed and constructed to minimize crack widths, thereby limiting seawater access to the reinforcement. Careful attention should also be given to structural aspects with properly designed and constructed joints to ensure that cracking is minimized to prevent the exposure of reinforcement. Additionally, concrete should reach a maturity equivalent of not less than 35 MPa (5000 psi) at 28 days when fully exposed to seawater.

Conductive coatings applied at the time of construction as part of a cathodic protection system may provide additional protection for concrete that is partially submerged or reaches down to saline groundwater. Silane coatings, which are water-repellent, have shown excellent protection characteristics. Coatings that significantly restrict evaporation of free water from the interior of concrete can reduce resistance to freezing and thawing.

Marine structures often involve thick sections and rather high cement factors. Such concrete may need to be treated as mass concrete in which the effect of the heat of hydration needs to be considered. When this is the case, the recommendations of ACI 207.1R, 207.2R, and 224R should be followed.

6.5—Acid attack

In general, portland cement does not have good resistance to acids, although some weak acids can be tolerated, particularly if the exposure is occasional.

6.5.1 Occurrence—The products of combustion of many fuels contain sulfurous gases that combine with moisture to form sulfuric acid. Also, sewage can be collected under conditions that lead to acid formation. Water draining from some mines and some industrial waters can contain or form acids that attack concrete.

Peat soils, clay soils, and alum shale can contain iron sulfide (pyrite) that, upon oxidation, produces sulfuric acid. Further reaction can produce sulfate salts that produce sulfate attack (Hagerman and Roosaar 1955; Lossing 1966; Bastiansen et al. 1957; Mourn and Rosenquist 1959).

Mountain streams are sometimes mildly acidic due to dissolved free carbon dioxide. These waters usually attack only the surface if the concrete is of good quality and has a

low absorption. Some mineral waters containing large amounts of either dissolved carbon dioxide, hydrogen sulfide, or both, can seriously damage any concrete (RILEM 1962; Thornton 1978). In the case of hydrogen sulfide, bacteria that convert this compound to sulfuric acid may play an important role (RILEM 1962).

Organic acids from farm silage or from manufacturing or processing industries, such as breweries, dairies, canneries, and wood-pulp mills, can cause surface damage. This can be of considerable concern in the case of floors, even where structural integrity is not impaired.

6.5.2 Mechanism—The deterioration of concrete by acids is primarily the result of a reaction between these chemicals and the calcium hydroxide of the hydrated portland cement. (Where limestone and dolomitic aggregates are used, they are also subject to attack by acids.) In most cases, the chemical reaction results in the formation of water-soluble calcium compounds that are then leached away by the aqueous solutions (Biczok 1972). Oxalic and phosphoric acid are exceptions, because the resulting calcium salts are insoluble in water and are not readily removed from the concrete surface. In the case of sulfuric acid attack, additional or accelerated deterioration results because the calcium sulfate formed will affect concrete by the sulfate attack mechanism described in [Section 6.2.2](#). If acids, chlorides, or other aggressive or salt solutions are able to reach the reinforcing steel through cracks or pores in the concrete, corrosion of steel can result ([Chapter 7](#)) that will, in turn, cause cracking and spalling of the concrete.

6.5.3 Carbonation by contact with water—Carbon dioxide can dissolve in rain, creating carbonic acid, which may then enter the ground. The decay of organic matter liberates carbon dioxide that may also form carbonic acid in ground-water systems. (Plant decay also produces humic acid, which is not discussed further in this section.) The concentration of carbonic acid in groundwater can become high enough to attack concrete. Calcium carbonate dissolves in the presence of carbonic acid to form free calcium (Ca^{2+}) and bicarbonate (HCO_3^-) ions. Detailed discussions of carbonate equilibria in natural systems can be found in Stumm and Morgan (1981) and Krauskopf and Bird (1995). If the alkalinity of the soil is high enough, the soil will neutralize or buffer the carbonic acid component of the water, preventing carbonic acid attack of the concrete. If the acid is not neutralized, it can attack concrete to varying degrees, ranging from mild to significant. This type of attack was referred to as aggressive CO_2 attack in the literature when, in fact, it was not CO_2 , but carbonic acid attack. Test criteria relating to carbonic acid attack have used the term “aggressive CO_2 ” (reported as milligrams of CO_2 per liter) for what was really carbonic acid in sufficient concentration to attack concrete. A discussion of this can be found in Lea (1971) and Hewlett (1998).

Waters that have potentially harmful concentrations of carbonic acid tend to have pH values that range from approximately neutral to slightly acidic. Because the rate of attack is dependent on both the properties of the concrete and concentration of the carbonic acid, neither the pH nor the amount of free CO_2 in water is a reliable indicator of the degree of potential harm. In addition, there is no consensus

as to the limiting values of the pH or CO_2 concentration in water, in part because of widely varying conditions in underground construction. Studies have shown that water containing more than 20 mg/L (20 ppm) of carbonic acid (reported as mg/L or ppm of CO_2) can result in rapid carbonation and attack of the hydrated cement paste. On the other hand, freely moving water with 10 mg/L (10 ppm) or less of carbonic acid (reported as mg/L or ppm of CO_2) can also result in significant carbonation (Terzaghi 1948, 1949; Hewlett 1998). German Institute of Standardization Specification DIN 4030 includes both criteria and a test method for assessing the potential of damage from carbonic acid-bearing water.

6.5.4 Recommendations—A dense concrete with a low w/cm provides a degree of protection against mild acid attack. Certain pozzolanic materials, and silica fume in particular, increase the resistance of concrete to acids (Sellevold and Nilson 1987). In all cases, however, exposure time to acids should be minimized, if possible, and immersion should be avoided.

No hydraulic-cement concrete, regardless of its composition, will long withstand water of high acid concentration (pH of 3 or lower). In such cases, an appropriate protective-barrier system or treatment should be used. ACI 515.1R gives recommendations for barrier systems to protect concrete from various chemicals.

6.6—Carbonation

6.6.1 General—When concrete or mortar is exposed to CO_2 , a reaction producing carbonates takes place that is accompanied by shrinkage.

Virtually all the constituents of hydrated portland cement are susceptible to carbonation. The results can be either beneficial or harmful, depending on the time, rate, and extent that they occur and the environmental exposure. Intentional carbonation during production can improve the strength, hardness, and dimensional stability of concrete products. In other cases, however, carbonation can result in deterioration and a decrease in the pH of the cement paste leading to corrosion of reinforcement near the surface. Exposure to CO_2 during the hardening process can affect the finished surface of slabs, leaving a soft, dusting, less wear-resistant surface. During the hardening process, the use of unvented heaters or exposure to exhaust fumes from equipment or other sources can produce a highly porous surface subject to further chemical attack. The source of the CO_2 can be either the atmosphere or water carrying dissolved CO_2 .

6.6.2 Atmospheric carbonation—Reaction of hydrated portland cement with CO_2 in the air is generally a slow process (Ludwig 1980). It is highly dependent on the relative humidity of the environment, temperature, permeability of the concrete, and concentration of CO_2 . The highest rates of carbonation occur when the relative humidity is maintained between 50 and 75%. Below 25% relative humidity, the degree of carbonation that takes place is considered insignificant (Verbeck 1958). Above 75% relative humidity, moisture in the pores restricts CO_2 penetration.

Relatively permeable concrete undergoes more rapid and extensive carbonation than dense, well-consolidated, and cured concrete. Lower w/cm and good consolidation serve to

reduce permeability and restrict carbonation to the surface. Industrial areas with higher concentrations of CO₂ in the air result in higher rates of carbonation.

CHAPTER 7—CORROSION OF METALS AND OTHER MATERIALS EMBEDDED IN CONCRETE
7.1—Introduction

The understanding of the conditions that cause corrosion (rusting) of reinforcing steel and prestressing steel is of vital importance. Avoidance of these conditions is necessary if concrete structures containing embedded steel are to have long service lives. The purpose of this chapter is to summarize the mechanisms of corrosion, the conditions under which corrosion occurs, and the methods and techniques that can be used to prevent or reduce corrosion and also to discuss the preservation of other embedded materials.

Concrete provides protection against corrosion of embedded steel because of the highly alkaline environment provided by the portland cement paste. The adequacy of the protection depends on the amount of concrete cover, the quality of the concrete, the details of the construction, the degree of exposure to chlorides (from concrete component materials and from the environment), and the service environment.

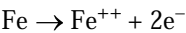
The reader seeking a more comprehensive treatment of the subject should consult ACI 222R. Sections 7.2 through 7.6 summarize the information in ACI 222R.

7.2—General principles of corrosion initiation in concrete

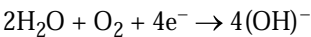
7.2.1 General—The process of corrosion in concrete is divided into several phases:

- *Initiation*—the elimination of the normal protective mechanism;
- *Corrosion growth (propagation)*—the corrosion process is established and corrosion progresses; and
- *Damage*—corrosion is sufficiently severe that the structure is unlikely to perform its intended function.

7.2.2 Corrosion process—Corrosion of steel in concrete is an electrochemical process that requires the development of an anode where oxidation takes place and a cathode where reduction takes place. At the anode, electrons are liberated and ferrous ions are formed



At the cathode, electrons are consumed and hydroxyl ions are created and liberated



The ferrous ions may subsequently combine with oxygen or hydroxyl ions and produce various forms of corrosion products (rust). The formation of rust often causes expansion that, in turn, may cause cracking and spalling of the concrete cover.

7.2.3 Protection mechanism in concrete

7.2.3.1 General—The high alkalinity (pH > 12.5) of concrete protects embedded steel reinforcement in concrete

from corrosion. When oxygen is present, the alkalinity of the pore solution causes an oxide film to form on the steel surface. This oxide film has very low solubility and, thus, the steel is protected against corrosion. The steel is said to exhibit passive behavior due to this oxide film.

The concrete cover over the steel maintains and protects the alkalinity and the passive film. If the passive film breaks down (depasivation), corrosion can initiate. The film can break down locally so that localized corrosion attacks result. If breakdown occurs over larger areas, general corrosion takes place. The primary causes of film breakdown include:

- Chemical, physical, or mechanical degradation of the concrete cover;
- Chloride penetration to the reinforcement; and
- Carbonation of the concrete to reinforcement depth.

7.2.3.2 Breakdown due to insufficient oxygen supply—

The passive layer requires an oxygen flux corresponding to approximately 0.2 to 0.3 mA/m² (1.3 × 10⁻⁴ to 1.9 × 10⁻⁴ mA/in²). If the oxygen flux is less than this, the passive film will gradually reduce in thickness, and bare steel will be exposed. The result is corrosion at an extremely low rate (corresponding to the oxygen flux), but at an active, though very negative, potential (Fidjestøl and Nilsen 1980). Such low oxygen diffusion can occur in submerged concrete, in sea or fresh water (Fidjestøl et al. 1985), or by exposure below the groundwater table.

7.2.3.3 Carbonation—The reaction between CO₂ from the ambient air and the paste of the concrete can cause a reduction in pH of the concrete to less than 9, resulting in the loss of passivity. Corrosion follows, and the rate depends on availability of oxygen and on the moisture content, because this affects the electrical resistivity of the concrete.

7.2.3.4 Pitting: local breakdown due to chloride—Chloride, at a certain level of concentration, will cause local breakdown of the passive layer, leading to corrosion. The rate depends on the availability of oxygen, the anode-cathode area ratio, and the resistivity of the concrete. The chloride concentration necessary to cause breakdown of the film depends on the pH and composition of the pore water, the quality of the oxide film, and the properties of the steel and concrete interface.

Although modern concrete has more variations in pore chemistry, chiefly due to the use of cementitious materials such as fly ash, slag cement, and silica fume, the limits for chlorides in new construction as given in ACI 222R are still appropriate, as follows:

	Acid-soluble (ASTM C1152/ C1152M)	Water-soluble (ASTM C1218/ C1218M)	Soxhlet method*
Prestressed concrete	0.08	0.06	0.06
Reinforced concrete in wet conditions	0.10	0.08	0.08
Reinforced concrete in dry or protected conditions	0.20	0.15	0.15

*Soxhlet method described in Appendix to ACI 222.1.
Note: All chloride contents expressed as percent Cl⁻ by mass of cement.

Committee 222 also comments that because some of the concrete-making materials contain chloride that will not be released into the concrete, past good performance of these materials may provide a basis for permitting higher chloride levels. The suggested levels provide a conservative approach that is necessary because of conflicting reports on chloride threshold levels, the effects of different exposure environments, and materials combinations. The conservative approach is also recommended because exposure conditions, such as that in bridge decks, parking structures, and marine environments, allow the intrusion of foreign chlorides. In instances where foreign chlorides are present, concrete should be made with concrete-making components that contain only trace amounts of chloride.

7.3—Propagation of corrosion

7.3.1 General—Even if corrosion has been initiated, as discussed previously, a structure may be far away from the end of serviceability, especially if the rate of corrosion is very low. One or more of the following factors in Sections 7.3.2 through 7.3.4 controls the corrosion rate.

7.3.2 Anodic control—Anodic control is based on sufficiently controlling the rate of dissolution of corrosion products formed at the anode. The rate at which the dissolution of iron takes place determines the corrosion rate.

7.3.3 Cathodic control—The rate of reduction of oxygen at the cathode limits the corrosion rate. The rate of corrosion is controlled by the availability of oxygen at the cathode and the ratio between cathodic and anodic areas, and is limited by the availability of oxygen, the size of the cathode, or both. Normally, the availability of oxygen at the cathode is far in excess of that needed to sustain corrosion, so the rate is controlled by other factors. Coating of the reinforcement is one way of limiting the oxygen supply to the cathode surface (the coating also prevents access of aggressive media to the steel surface). Another example of cathodic control is where the concrete is completely water-saturated, which greatly reduces the oxygen flux from the concrete surface to the steel.

7.3.4 Resistivity control—Resistivity control requires the resistance in the concrete to be high enough as to limit the amount of current that can be developed from the two half-cell reactions.

There may be a large difference in half-cell potential between the cathodic and anodic areas of the steel. If the electrical resistance R in the concrete between the two areas is sufficiently large, however, then most of the potential difference is spent in overcoming the voltage drop IR caused by the current flow I against this resistance, even at minute corrosion current densities. This effect can often be seen in fully carbonated concrete in a reasonably dry environment, in concrete that contains SCMs, or where there is a large distance between anodic and cathodic areas.

7.4—Corrosion-related properties of concreting materials

7.4.1 Portland cement—The high pH of portland cement paste results from the presence of hydroxides of calcium, potassium, and sodium. Calcium hydroxide is the most

abundant, and constitutes 15 to 25% of the paste. Because the pH of saturated solutions of calcium hydroxide is only 12.4, the pH of 13.5 to 14 often found in concrete pore water (Justnes and Nygaard 1994) is explained by the OH^- ions associated with alkalis in the concrete.

The presence of C_3A in the cement appears to be beneficial to the reduction of chloride ingress. This was first established by Verbeck (1968) and has since been confirmed by several other researchers, such as Rasheeduzzafar et al. (1992). The main conclusion of this work is that the use of very low C_3A (Type V) cements in a strong chloride environment is generally not recommended.

7.4.2 Supplementary cementitious materials—Fly ash, slag cement, and silica fume are generally assumed to improve the resistance of concrete to chloride-induced corrosion. While it has been found that the introduction of such materials to concrete will consume some of the CaOH_2 that acts as a buffer against changes in pH due to carbonation of concrete (Bijen and van Selst 1991; Horiguchi et al. 1994; Branca et al. 1992), the improvement to pore distribution and permeability can counteract this depletion in CaOH_2 (Torii and Kawamura 1994; Hakkinen 1992). Also, the effect of the cementitious materials is often to increase the electrical resistivity of the concrete, thus reducing the rate of any corrosion that has been initiated (Schiessl et al. 1994; Fidjestøl 1987, 1991; Alonso et al. 1992).

In the 1990s, the development of corrosion-resistant concrete focused on blends of portland cement and other cementitious materials. Summary reports (Bajewa et al. 1994) and experience (Maage and Helland 1991; Maage et al. 1994; Berke et al. 1991) support laboratory data (Collepari et al. 1994; Anqi et al. 1991; Hussain and Rasheeduzzafar 1994; Decter et al. 1989; Haque et al. 1992; Ozyildirim 1994).

Fly ash is added to concrete in typical amounts of 10 to 30% by mass of cement. Several specifications for large marine civil works have included fly ash in the concrete mixture.

Slag cement has been used in marine work since the early 1900s; the experiences with respect to resistance against chloride-induced corrosion are generally good (Wiebenga 1984).

Silica fume works in several ways to reduce the risk of corrosion. The improved impermeability of silica fume concrete means a greatly reduced rate of chloride penetration in marine structures and structures exposed to deicing salts. Such concrete also has very high electrical resistivity, thereby greatly diminishing the rate of corrosion, should it be initiated (Wolsiefer 1991; Petterson 1995; Fidjestøl 1993; Fidjestøl and Frearson 1994; Alonso et al. 1992; Berke et al. 1992; Gautefall and Vennesland 1985; Zhang and Gjorv 1991; Fidjestøl 1987).

7.4.3 Aggregates—Aggregates can contain chloride salts, particularly those aggregates that have been exposed to seawater or whose natural sites are in groundwater containing chloride. There have been reported instances (Gaynor 1985) where quarried stone, gravel, and natural sand contained small amounts of chloride that have provided concrete with chloride levels that exceed the maximum permissible levels previously described.

7.4.4 Mixing water—Potable mixing water can contain small amounts of chloride, usually at levels from 20 to 100 ppm.

7.4.5 Admixtures

7.4.5.1 General—Admixtures composed principally or to a large part of CaCl_2 should normally not be used in concrete containing embedded metal. Some water-reducing admixtures may contain chloride to improve admixture performance, but may contribute only small amounts of chloride to the concrete when they are added at recommended rates. Normal-setting admixtures that contribute much less than 0.1% chloride by mass of cement are most common, and their use should be evaluated based on the application. Chemical admixtures are described in detail in ACI 212.3R.

7.4.5.2 Accelerators—Accelerating admixtures, other than those based on CaCl_2 , have been used in concrete with varying success. Accelerators that do not contain chloride should not automatically be assumed to be noncorrosive. The materials most commonly used in chloride-free accelerators are calcium formate, sodium thiocyanate, calcium nitrate, and calcium nitrite. It is generally accepted that formates (Holm 1987) are noncorrosive in concrete, and that calcium nitrite is also an inhibitor.

7.4.5.3 Inhibitors—Calcium nitrite has been widely used as an accelerating admixture that will also function as a corrosion inhibitor. Laboratory studies have demonstrated that it will delay the onset of corrosion or reduce the rate after it has been initiated (Berke 1985; Berke and Roberts 1989). The ratio of chloride ions to nitrite ions is important. Studies (Berke 1987) show that calcium nitrite can provide corrosion protection even at chloride-nitrite ratios exceeding 1.5 to 1.0 by mass. Although dosage rates are varied, 26 to 110 mL/kg (40 to 170 oz per 100 lb) of cement is the common range. Berke and Rosenberg (1989) compiled an extensive review of calcium nitrite's use in concrete. The review documents the effectiveness of calcium nitrite as a corrosion inhibitor for steel, galvanized steel, and aluminum in concrete. It was later updated by Berke et al. (1994). If the accelerating effect from calcium nitrite is undesirable, use of a retarder is recommended. Increased amounts of air-entraining admixture may be necessary when calcium nitrite is used to maintain the desired air content.

Other corrosion-inhibiting admixtures are also commercially available. These are primarily organic-based, and include an aqueous solution of amines and fatty esters (Nmai et al. 1992); another contains proprietary alkanolamines (Collins et al. 1993). The former is reported to protect by reducing chloride ingress and by forming a protective film at the steel surface (Nmai et al. 1992; Bobrowski and Youn 1993). Laboratory evaluations indicate that the aqueous solutions of amines and fatty esters will delay the onset and reduce the rate of corrosion (Nmai et al. 1992; Nmai and Krauss 1994).

7.5—Preventing corrosion

7.5.1 General—In preventing corrosion, each of the following critical points should be evaluated in terms of concrete service life:

1. Initiation of corrosion;
2. Corrosion products becoming visible (staining);

3. Cracking, spalling, or both; and

4. The load-carrying capacity of the structure if it is seriously reduced or the structure can no longer perform its intended purpose.

Traditional service life considerations only take into account the initiation of corrosion; however, even for very conspicuous structures, the use of Criterion 2 or 3 can be considered target for design considerations.

Corrosion can be minimized by selecting processes and materials that delay the onset of corrosion and that will, once corrosion is initiated, minimize the rate of corrosion. Detailed guidance on preventive strategies can be found in ACI 222.3R. A general discussion of some of the factors affecting corrosion resistance are described in the following sections.

7.5.2 Design and process

7.5.2.1 Concrete quality and cover over steel

7.5.2.1.1 Cover depth—Extensive studies (Clear 1976; Pfeifer et al. 1987; Marusin and Pfeifer 1985) have shown that 25 mm (1 in.) cover over bare steel bars is inadequate for chloride protection in severe corrosion environments, even if the concrete has a w/cm as low as 0.30. Tests have also shown that the chloride content in the top 12 mm (1/2 in.) of concrete can be very high compared with that at depths of 25 to 50 mm (1 to 2 in.), even in concrete of high quality, such as one having a w/cm of 0.30. As a result, adequate cover for moderate-to-severe corrosion environments should be a minimum of 40 to 50 mm (1-1/2 to 2 in.).

Concrete will absorb salts applied in deicing operations. To reduce the likelihood of corrosion, a minimum cover of 50 mm (2 in.) and a low w/cm (0.40 maximum) are desirable. Because of construction tolerances, a design cover of at least 65 mm (2.6 in.) is needed to obtain a minimum cover of 50 mm (2 in.) over 90 to 95% of the reinforcing steel (Van Daveer and Sheret 1975).

Typical requirements for cover depth in chloride environments are 75 mm (3 in.) (Kjaer et al. 1994), and 50 to 60 mm (2 to 2-1/2 in.) (ACI 318).

7.5.2.1.2 Concrete quality—Numerous test programs have shown that concrete made with a low w/cm and adequate cover over the steel performs significantly better than concrete made with a higher w/cm . Chloride-ion permeability to a 25 mm (1 in.) depth is approximately 400 to 600% greater for concrete made with w/cm of 0.40 and 0.50 than for concrete made with a w/cm of 0.32. Similarly, information gained from the testing of concrete with other cementitious materials shows that the proper use of such materials can extend the time to corrosion and reduce the rate of corrosion should it initiate.

New requirements in specifications place strict requirements on the mixture proportions for severe chloride environments. One example is the 1995 Norwegian Public Roads Administration bridge code, which has very specific requirements for mixture proportions depending on the location of the structure ($w/cm < 0.38$ to 0.40 and a certain silica fume content). Another example is the Great Belt project, where the combined benefits of a low w/cm with the use of silica fume and fly ash are expected to ensure a 100-year lifetime. A similar mixture proportioning philosophy is used for 1 to

2 million m³ (1.3 to 2.6 million yd³) of concrete for the 10 km (6 mile) connection across Oresund from Sweden to Denmark.

7.5.2.1.3 Cracks—One cause of chloride intrusion into concrete is cracks. Cracks allow chloride infiltration at a much faster rate than by the slower diffusion processes, and establish chloride concentration cells that can accelerate corrosion. The permissible crack width depends on the location of the crack and the ambient environment (Fidjestøl et al. 1985). In structures submerged in seawater, for example, even large, dynamic cracks may be closed by brucite and aragonite (magnesium hydroxide and calcium carbonate) within a fairly short period (Espelid and Fidjestøl 1986; Buenfeld and Newman 1986).

To minimize crack formation, concrete should always be made with the lowest practical water content. Also, proper detailing of reinforcement, sufficient minimum and structural reinforcement, and control of heat of hydration and restraint effects are important in producing a structure in which cracks do not degrade corrosion resistance.

7.5.2.2 Concrete resistivity—When concrete is kept moderately dry, corrosion of steel is minimized. For example, if concrete containing up to 2% CaCl₂ is allowed to dry to a maximum internal relative humidity of 50 to 60%, embedded steel should either not corrode or corrode at an inconsequential rate (Tutti 1982).

While the surface regions of exposed concrete structures will have high or low electrical conductivity values depending upon the wetting and drying conditions of the environment, the concrete interior usually requires long drying periods to achieve low electrical conductivity. Tests (Pfeiffer et al. 1987) show that 180 to 230 mm (7 to 9 in.) thick reinforced concrete slabs with *w/cm* ranging from 0.30 to 0.50 have essentially equal initial AC electrical-resistance values between the top and bottom reinforcing bar mats at 28 days.

Cementitious materials can give very high electrical resistivity in concrete. Data are available showing that silica fume, slag cement, and fly ash (particularly silica fume) will give concrete resistivities far in excess of what is provided by portland-cement concrete (Cabrera and Ghodoussi 1994; Fidjestøl and Frearson 1994; Gautefall and Vennesland 1985; Berke 1988). Similarly, AC resistance tests on concrete made with silica fume at a *w/cm* of 0.20 show extremely high initial electrical resistance values when compared with concrete having a *w/cm* of 0.30 to 0.50. The high electrical resistance of silica fume concrete can be due to denser paste microstructure and to changes in the pore chemistry.

The high electrical resistivity of blended binder systems is confirmed by tests using ASTM C1202, which provides a method to determine conductivity that is then used as an indirect indication of chloride diffusivity. In several investigations, there has been a relationship to chloride diffusion determined by more conventional diffusion or ponding tests (Detwiler and Fapohunda 1993; Wolsiefer 1991; Misra et al. 1994; Burg and Ost 1992). As implied in the standard, however, this relationship cannot be assumed to be universal because it will also depend on the composition of the binder system, such as content of and type of cement and cementitious materials used.

7.5.3 Construction aspects

7.5.3.1 Workmanship—Good workmanship is vital for securing uniform concrete with low permeability. For low-slump concrete, segregation and honeycombing can be avoided by good consolidation practices. It is also essential that the requirements of the specifications be met. It helps very little to specify 30 mm (1.2 in.) cover when the construction results in a mean value of 25 mm (1 in.) with less than 20% of measured cover depths satisfying requirements (Kompén 1994).

7.5.3.2 Reinforcement detailing—Two factors are important to consider in the detailing of the reinforcement:

- Adequate spacing should be provided to allow for proper placing of the concrete cover so that honeycombing and poor compaction are avoided and good bond between concrete and steel are obtained; and
- Corrosion is relatively more severe for small bars than for large bars. One mm (0.04 in.) corrosion of a 10 mm (No. 3) bar means nearly 40% loss of cross section, while for a 25 mm (No. 8) bar, it will mean 15% loss of cross section. On the other hand, large bars could cause larger cracks than smaller bars.

7.5.3.3 Curing—Good curing reduces permeability because of increased hydration of the cement. At least 7 days of uninterrupted moist-curing or membrane-curing should be specified. Prevention of the development of excessive early thermal stresses is also important (Acker et al. 1986; Marusin 1989).

7.5.3.4 Formwork—Good, tight formwork is essential. Properly supported screeding equipment and correct supports for the reinforcement are important for attaining the cover protection specified. The use of side form spacers for reinforcing bars in vertical formwork is similarly important.

Controlled permeability liners for formwork may improve the quality of the cover (Sha'at et al. 1993).

7.5.4 Design—The design process can do much to reduce corrosion attack because proper detailing can minimize accumulation of salts and the establishment of high humidity areas where the corrosion process can be sustained.

7.5.4.1 General layout of structure—The placement and general layout of the structure are important for a favorable environmental load on the structure. For instance, an increase in the height of a bridge over the sea will reduce the chloride load: field inspection of concrete bridges (Fluge and Blankvoll 1995) has shown that an increase in bridge height above sea level from 8 to 28 m (26 to 92 ft) will reduce the chloride load (that is, the amount of chlorides deposited on the surface) by as much as 85%. The chloride exposure was also up to eight times higher in the lee side of the structure than on the windward side.

Similarly, moving bridge columns away from traffic splash will reduce the chloride load in the concrete.

7.5.4.2 Drainage—Particular attention should be given to design details to ensure that water will drain and not pond on surfaces. There are a number of details that are important (Kompén 1994), such as proper slope and extended drainage pipes that take the water away from the concrete surface.

7.5.4.3 Exposed items—Careful attention should be given to partially embedded or partially exposed items, such as bolts, that are exposed directly to corrosive environments. The resistance of these items to the corrosive environment should be investigated, and the coupling of dissimilar metals should be avoided. Concrete should be carefully placed around embedded items so that it is well-consolidated and does not create paths that will permit corrosive solutions to easily reach the concrete interior.

7.5.5 Special protective systems—Costs of repairing corrosion-induced damage are very high. Many protective systems have been proposed, some of which have been shown to be effective, while others have failed. It is beyond the scope of this guide to discuss all possible systems. Some systems, however, are listed as follows:

1. Overlays and patches of a very low w/cm (0.32), latex-modified concrete overlays (Clear and Hay 1973; Federal Highway Administration 1975b), concrete containing silica fume, and concrete containing high-range water-reducing admixtures;

2. Epoxy-coated reinforcing steel;

3. Corrosion-resistant steels (stainless of various grades) (Rasheeduzzafar et al. 1992);

4. Waterproof membranes (Van Til et al. 1976);

5. Surface protective-barrier systems produced from silanes, siloxanes, epoxies, polyurethanes, and methacrylates (Van Daveer and Sheret 1975); and

6. Cathodic protection.

7.6—Corrosion of materials other than steel

Discussions of some of these materials are given by Erlin (1994).

7.6.1 Aluminum—Corrosion of aluminum embedded in concrete can occur and can crack the concrete. Conditions conducive to corrosion are created if the concrete contains steel in contact with the aluminum, chlorides are present in appreciable concentrations, or the cement is high in alkali content (Woods 1968; Erlin 1994). When the metals are coupled, increasing ratios of steel area, particularly in the presence of appreciable amounts of chloride, increase corrosion of the aluminum. Additionally, hydrogen gas evolution may occur when fresh concrete contacts aluminum. This may increase the porosity of the concrete and, therefore, the penetration of future corrosive agents. Some aluminum alloys are more susceptible to this problem than others. Corrosion inhibitors, such as calcium nitrite, have been shown to improve the corrosion resistance of aluminum in concrete (Berke and Rosenberg 1989).

7.6.2 Lead—Lead in damp concrete will be attacked by the $CaOH_2$ in the concrete, and may be destroyed in a few years. Contact of lead with reinforcing steel can accelerate the attack. Lead should be isolated from the concrete, by protective plastic, or other materials that are unaffected by damp concrete. Corrosion of embedded lead is not likely to damage the concrete.

7.6.3 Copper and copper alloys—Copper is not normally corroded by concrete, as is evidenced by the widespread and successful use of copper waterstops and the embedment of

copper pipes in concrete for many years (Erlin and Woods 1978). Corrosion of copper pipes, however, has been reported where ammonia is present. Also, there have been reports that small amounts of ammonia, and possibly of nitrates, can cause stress corrosion cracking of embedded copper. Galvanic corrosion of steel will occur if the steel is connected to the copper.

7.6.4 Zinc—Zinc reacts with alkaline materials, such as those found in concrete. Zinc in the form of a galvanizing coating on reinforcing steel, however, is sometimes intentionally embedded in concrete. Available data are conflicting as to the benefit, if any, of this coating (Cook 1980; Stark and Perenchio 1975; Hill et al. 1976; Griffin 1969; Federal Highway Administration 1976). A chromate dip on the galvanized bars or the use of 400 ppm of chromate in the mixing water is recommended to prevent hydrogen evolution in the fresh concrete. Caution should be exercised when using chromium salts because of possible skin allergies. Additionally, users are cautioned against permitting galvanized and black steel to come in contact with each other in a structure because the use of dissimilar metals can cause galvanic corrosion. Corrosion inhibitors, such as calcium nitrite, have been shown to improve the corrosion resistance of zinc in concrete (Berke and Rosenberg 1989; Page et al. 1989).

Some difficulty has been experienced with the corrosion and perforation of corrugated galvanized sheets used as permanent bottom forms for concrete roofs and bridge decks. Such damage has been confined largely to concrete containing appreciable amounts of chloride and to areas where chloride solutions are permitted to drain directly onto the galvanized sheet.

7.6.5 Other metals—Chromium and nickel alloyed metals generally have good resistance to corrosion in concrete, as do silver and tin. The corrosion resistance of some of these metals may be adversely affected by the presence of soluble chlorides from seawater or deicing salts. Special circumstances might justify the cost of stainless steel in marine locations if data are available to document their superior performance in concrete containing moisture and chloride or other electrolytes. The 300 Series stainless steels, however, are susceptible to stress corrosion cracking when the temperature is over 60 °C (140 °F) and chloride solutions are in contact with the steel material. Embedded natural-weathering steels generally do not perform well in concrete containing moisture and chloride. Weathering steels adjoining concrete may discharge rust and cause staining of concrete surfaces.

7.6.6 Plastics—Plastics are being used increasingly in concrete in applications such as pipes, shields, waterstops, chairs, as well as a component in concrete. Many plastics are resistant to strong alkalis and would therefore be expected to perform satisfactorily in concrete. Because of the great variety of plastics and materials compounded with them, however, specific test data should be developed for each intended use.

7.6.7 Wood—Wood has been widely used in or against mortars and concrete. Such use includes the incorporation of sawdust, wood pulp, and wood fibers in the concrete and the embedment of timber.

The use of untreated sawdust, wood chips, or fibers usually results in slow-setting and low-strength concrete. The addition of hydrated lime equal to 1/3 to 1/2 the volume of the cement is usually effective in minimizing these problems. The further addition of up to 5% of CaCl_2 dihydrate by mass of cement has also helped to minimize these problems. CaCl_2 in such amounts can cause corrosion of embedded metals, however, and can have adverse effects on the concrete itself.

Another problem with such concrete is the high volume change, which occurs even with changes in atmospheric humidity. This volume change may lead to cracking and warping.

The embedment of lumber in concrete has sometimes resulted in leaching of the wood by $\text{Ca}(\text{OH})_2$ with subsequent deterioration. Soft woods, preferably with high resin content, are reported to be most suitable for such use.

7.7—Summary

Portland-cement concrete can provide excellent corrosion protection to embedded steel. When corrosion occurs, the costs of repairs can be exceedingly high. The use of high-quality concrete, adequate cover over the steel, and good design are prerequisites if deterioration due to corrosion is to be minimized.

ACI 222R and ACI 222.3R provide a summary of the causes and mechanisms of corrosion of steel. It includes information on how to protect against corrosion in new structures and procedures for identifying corrosive environments; it also describes some remedial measures for existing situations where corrosion is occurring.

CHAPTER 8—ABRASION

8.1—Introduction

The abrasion resistance of concrete is defined as the “ability of a surface to resist being worn away by rubbing and friction” (American Concrete Institute 2008). Abrasion of floors and pavements can result from production operations or foot or vehicular traffic. Abrasion resistance is therefore of concern in industrial floors (Covell 1928). Wind or water-borne particles can also abrade concrete surfaces (Price 1947). There are instances where abrasion is of little concern structurally, yet there may be a dusting problem that can be objectionable in some kinds of service. Abrasion of concrete in hydraulic structures is discussed only briefly in this guide; the subject is treated in more detail in ACI 210R and 210.1R.

8.2—Testing concrete for resistance to abrasion

Research to develop meaningful laboratory tests on concrete abrasion has been ongoing since the early 1900s. There are several different types of abrasion, and no single test method has been found that is adequate for all conditions. A detailed description of abrasion/erosion test methods can be found in Bakke (2006). Prior (1966) described four broad areas related to abrasion, shown as follows. The current document discusses only the first two broad areas, and does not discuss erosion or cavitation.

1. Wear on floor and slab construction. **Table 8.1** shows classes of wear and special considerations required for good wear resistance (ACI 302.1R);

2. Wear on concrete road surfaces due to attrition, scraping, and percussion from heavy trucks and automobiles;

3. Erosion of hydraulic structures, such as dams, spillways, tunnels, bridge piers, and abutments, due to the action of abrasive materials carried by flowing water (attrition and scraping); and

4. Cavitation action on concrete in dams, spillways, tunnels, and other water-carrying systems due to high flow velocities and negative pressures.

ASTM C779/C779M covers three operational procedures for evaluating floor surfaces: Procedure A, revolving discs; Procedure B, dressing wheels; and Procedure C, ball bearings.

Each method has been used to develop information on wear resistance. Liu (1994) commented that the most reproducible results are obtained by the method involving the use of revolving discs. Reproducibility of abrasion testing is an important factor in selecting the test method. Replication of results is necessary to avoid misleading results from single tests.

The concrete surface condition, loose aggregates that are dislodged and abraded during the test procedure, and care and selection of representative samples affects test results. Samples that are fabricated in the laboratory should be identical for comparison, and the selection of field-testing sites should be made on the basis of providing representative results.

To set limits for abrasion resistance of concrete, it is necessary to rely on the relative values obtained during testing to provide a prediction of service.

Underwater abrasion presents special demands for test procedures. ASTM C1138 uses agitation of steel balls in water to determine abrasion resistance.

The ASTM C418 test procedure uses a sandblasting apparatus to measure the depth or wear to simulate comparative sand-impinged wear resistance. This test provides a means for evaluating resistance to abrasion caused by wind-blown sand.

8.3—Factors affecting abrasion resistance of concrete

The abrasion resistance of concrete is a progressive phenomenon. Initially, the resistance is related to compressive strength of the wearing surface. Therefore, initial judgments regarding relative floor wear can be made on the basis of compressive strength.

As softer paste wears away, however, the particles of fine and coarse aggregate are exposed, and abrasion and impact will cause additional degradation that is more related to the paste-to-aggregate bond strength and the relative hardness of the aggregate than to the compressive strength of the concrete.

Tests and field experience have generally shown that abrasion resistance is proportional to the compressive strength of concrete (Scripture et al. 1953; Witte and Backstrom 1951). Because abrasion occurs at the surface, it is critical that the surface strength be maximized. Resistance can be greatly improved by the use of dry shakes and toppings, finishing techniques, and curing. In addition, the use of concrete mixtures having low to moderately low (< 0.45) w/cm is recommended to improve the strength and wear resistance of surface paste.

Table 8.1—Floor classifications*

Class	Anticipated type of traffic	Use	Special considerations	Final finish
1. Single course	Exposed surface—foot traffic	Offices, churches, commercial, institutional, multi-unit residential Decorative	Uniform finish, nonslip aggregate in specific areas, curing Colored mineral aggregate, color pigment or exposed aggregate, stamped or inlaid patterns, artistic joint layout, curing	Normal steel-troweled finish, nonslip finish where required As required
2. Single course	Covered surface—foot traffic	Offices, churches, commercial, multi-unit residential, institutional with floor coverings	Flat and level slabs suitable for applied coverings, curing. Coordinate joints with applied coverings	Light steel-troweled finish
3. Two course	Exposed or covered surface—foot traffic	Unbonded or bonded topping over base slab for commercial or non-industrial buildings where construction type or schedule dictates	<i>Base slab</i> —good uniform level surface tolerance, curing <i>Unbonded topping</i> —bondbreaker on base slab, minimum thickness 3 in. (75 mm), reinforced, curing <i>Bonded topping</i> —properly sized aggregate, 3/4 in. (19 mm) minimum thickness curing	<i>Base slab</i> —troweled finish under unbonded topping; clean, textured surface under bonded topping <i>Topping</i> —for exposed surface, normal steel-troweled finish. For covered surface, light steel-troweled finish
4. Single course	Exposed or covered surface—foot and light vehicular traffic	Institutional or commercial	Level and flat slab suitable for applied coverings, nonslip aggregate for specific areas, curing. Coordinate joints with applied coverings	Normal steel-troweled finish
5. Single course	Exposed surface—industrial vehicular traffic, that is, pneumatic wheels <u>and moderately soft solid wheels</u>	Industrial floors for manufacturing, processing, and warehousing	Good uniform subgrade, joint layout, abrasion resistance, curing	Hard steel-troweled finish
6. Single course	Exposed surface— heavy-duty industrial vehicular traffic, that is, hard wheels <u>and heavy wheel loads</u>	Industrial floors subject to heavy traffic; may be subject to impact loads	Good uniform subgrade, joint layout, load transfer, abrasion resistance, curing	Special metallic or mineral aggregate surface hardener; repeated hard steel-troweling
7. Two course	Exposed surface— heavy-duty industrial vehicular traffic, that is, hard wheels <u>and heavy wheel loads</u>	Bonded two-course floors subject to heavy traffic and impact	<i>Base slab</i> —good uniform subgrade, reinforcement, joint layout, level surface, curing <i>Topping</i> —composed of well-graded all-mineral or all-metallic aggregate. Minimum thickness 3/4 in. (19 mm). Mineral or metallic aggregate surface hardener applied to high-strength plain topping to toughen, curing	Clean, textured base slab surface suitable for subsequent bonded topping. Special power floats for topping are optional, hard steel-troweled finish
8. Two course	As in Classes 4, 5, or 6	Unbonded topping—on new or old floors where construction sequence or schedule dictates	Bondbreaker on base slab, minimum thickness 4 in. (100 mm), abrasion resistance, curing	As in Classes 4, 5, or 6
9. Single course or topping	Exposed surface—superflat or critical surface tolerance required. Special materials-handling vehicles or robotics requiring specific tolerances	Narrow-aisle, high-bay warehouses; television studios, ice rinks, or gymnasiums. Refer to ACI 360R for design guidance	Varying concrete quality requirements. Special application procedures and strict attention to detail are recommended when shake-on hardeners are used. F_F 50 to F_F 125 (“superflat” floor). Curing	Strictly following techniques as indicated in Section 8.9 of ACI 302.1R-04.

*Table 2.1 of ACI 302.1R-04.

Although useful as a relative indicator, reliance should not be placed solely on the results of compressive strength tests. Rather, careful inspection should be made during installation and finishing of floor slabs (Kettle and Sadegzadeh 1987).

With a given concrete mixture, compressive strength at the surface is improved by:

- Avoiding segregation;
- Eliminating bleeding;
- Properly timed finishing;
- Minimizing surface w/cm (forbid any water addition to the surface during finishing);
- Hard troweling of the surface; and
- Proper curing procedures.

Economical proportioning of the mixture for increased compressive strength includes using a minimum w/cm and proper aggregate size.

Consideration should be given to the quality of the aggregate (Scripture et al. 1953; Smith 1958). The service life of

some concrete slabs, such as warehouse floors that are subjected to abrasion by steel or hard rubber-wheeled traffic, is greatly lengthened by the use of hard, tough aggregates.

Special aggregates can be used either with the dry-shake method (ACI 302.1R) or as part of a high-strength topping mixture. If abrasion is the principal concern, addition of high-quality quartz, traprock, or emery aggregates properly proportioned with cement will increase the wear resistance by improving the compressive strength at the surface. The aggregates used in topping mixtures or dry shakes should be harder than the aggregate in the concrete. For additional abrasion resistance, a change to a blend of metallic aggregate and cement will further increase the abrasion resistance and increase service life. Another advantage of using metallic aggregate is improved impact resistance, especially at joints.

The use of two-course floors using a high-strength topping is generally limited to floors where both abrasion and impact resistance are required. While providing excellent abrasion

resistance, a two-course floor will generally be more expensive. Additional impact resistance can be obtained by using a topping that contains portland cement and metallic aggregate.

A key element in the production of satisfactory floor surfaces is curing (Liu 1994) (ACI 302.1R; 308R). Because the uppermost part of the concrete surface is the region that is abraded by traffic, maximum strength and toughness are the most important elements for ensuring resistance to surface abrasion. This is partially accomplished through proper finishing operations, troweling techniques, and adequate and timely curing practices. The effect of curing efficiency (absorptivity) at the top-wearing surface has been shown to be directly related to abrasion resistance. Curing has less effect on the abrasion resistance of deeper sections of the same concrete (Senbetta and Scholer 1984).

8.4—Recommendations for obtaining abrasion-resistant concrete surfaces

8.4.1 The following factors directly impact concrete strength and, therefore, abrasion resistance (ACI 302.1R):

- *A low w/cm at the surface*—Steps to lower w/cm include the use of water-reducing admixtures, mixture proportions to reduce bleeding, timing of finishing operations that avoid the addition of water during troweling, and vacuum dewatering;
- *Well-graded fine and coarse aggregates (meeting ASTM C33)*—The maximum size of coarse aggregate should be chosen for optimum workability and minimum water content;
- *Lowest slump consistent with proper placement and consolidation as recommended in ACI 309R; and*
- *Air contents consistent with exposure conditions*—In addition to a detrimental effect on compressive strength, air content levels can contribute to surface blistering and delamination if finishing operations are improperly timed. Entrained air should never be used for dry-shake toppings unless special precautions are followed.

8.4.2 *Two-course floors*—High-strength toppings in excess of 40 MPa (6000 psi) provide increased abrasion resistance. The nominal maximum aggregate size for topping mixtures is 12.5 mm (1/2 in.).

8.4.3 *Special concrete aggregates*—Selection of aggregates for improved strength performance at a given w/cm also improves abrasion resistance. Typically, aggregates are applied as dry shakes or in high-strength, bonded toppings.

8.4.4 *Proper finishing procedures*—Floating and troweling operations should be delayed until the concrete has lost its surface sheen. It may be necessary to remove free water from the surface to permit finishing operations to continue before the base concrete hardens. Standing water should never be worked into concrete surfaces because it reduces the compressive strength of the surface paste. The delay period will vary greatly depending on temperature, humidity, and air movement. Greater detail regarding proper finishing operations is provided in ACI 302.1R.

8.4.5 *Vacuum dewatering*—Vacuum dewatering is a method for removing water from concrete immediately after placement (New Zealand Portland Cement Association

1975). While this permits a reduction in w/cm , the quality of the finished surface is still highly dependent on the timing of finishing and subsequent actions by the contractor. It should be ensured that proper dewatering is accomplished at the edges of the vacuum mats. Improperly dewatered areas are less resistant to abrasion because of a higher w/cm .

8.4.6 *Special dry shakes and toppings*—When severe wear is anticipated, the use of special dry shakes or topping mixtures should be used. The recommendations in ACI 302.1R should be followed.

8.4.7 *Proper curing procedures*—For most concrete floors, water curing (keeping the concrete continuously wet) is the most effective method of producing a hard, dense surface. Water curing, however, may not be always practical. Curing compounds, which seal moisture in the concrete, are used as an alternative. Curing compounds also provide protection against early carbonation and prevent premature or excessive loss of surface moisture. Moist curing of metallic shake toppings is not recommended because some water sources and rainwater have a pH of less than 7, which may result in the oxidation of the metallic aggregate particles.

Water curing is accomplished through the use of sprays, damp burlap, or cotton mats. Water-resistant paper or plastic sheets are satisfactory provided that the concrete is first wetted and then immediately covered, with the edges overlapped and sealed using water-resistant tape.

Curing compounds should meet the minimum requirements of ASTM C309. They should be applied in a uniform coat immediately after concrete finishing and in accordance with the manufacturer's recommendations. Recommended coverage rates will vary depending on the surface texture of the finished surface. A smoother floor surface will have better moisture retention properties compared with a highway slab. Smaller peaks and valleys result in a lower evaporation rate and, therefore, a lower coverage rate. The compound should be covered with scuff-resistant paper if the floor is subjected to traffic before curing is complete. Curing compounds should not be required for surfaces that receive paint or floor tile unless the curing compound is compatible with these materials.

Wet curing is recommended for concrete with a low w/cm (to supply additional water for cement hydration), where cooling of the surface is desired, where concrete will later be bonded, or where liquid hardeners will be applied. Curing methods are described in detail in ACI 308R.

Heaters burning fossil fuels or other sources of CO₂, such as finishing machines, vehicles, and welding machines, should not be used without attention to proper ventilation (Section 6.6). Carbon dioxide can adversely affect fresh concrete surfaces between the time of placement and the application of a curing compound. The severity of the effect is dependent on the concentration of CO₂, the humidity and ambient temperature, and the length of exposure to the air (Kauer and Freeman 1955). Early carbonation will greatly reduce the abrasion resistance of concrete surfaces. The extent of the reduction depends on the depth of carbonation. The only effective repair is to grind the surface to sound, hard concrete.

8.5—Studded tire and tire chain wear on concrete

Tire chains and studded snow tires cause considerable wear to concrete surfaces, even where the concrete is of good quality. Abrasive materials, such as sand, are often applied to the pavement surface when roads are slippery. Experience from many years' use of sand in winter, however, indicates that this causes little wear if the concrete is of good quality and the aggregates are wear-resistant.

Studded snow tires cause serious damage, even to high-quality concrete. The damage is due to the dynamic impact of the small tungsten carbide tip of the studs, of which there are roughly 100 in each tire. One laboratory study showed that studded tires running on surfaces to which sand and salt were applied caused 100 times as much wear as unstudded tires (Krukar and Cook 1973). Fortunately, the use of studded tires has been declining for a number of years.

Wear caused by studded tires is usually concentrated in the wheel tracks. Ruts from 6 to 12 mm (1/4 to 1/2 in.) deep can form in a single winter in regions where approximately 30% of passenger cars are equipped with studded tires and traffic is heavy (Smith and Schonfeld 1970). More severe wear occurs where vehicles stop, start, or turn (Keyser 1971).

Investigations have been made, principally in Scandinavia, Canada, and the United States, to examine the properties of existing concrete as related to studded tire wear (Smith and Schonfeld 1970, 1971; Keyser 1971; Preus 1973; Wehner 1966; Thurmann 1969). In some cases, there was considerable variability in the data, and the conclusions of the different investigators were not in agreement; however, most found that a hard coarse aggregate and a high-strength mortar matrix are beneficial in resisting abrasion.

Another investigation was aimed at developing more wear-resistant types of concrete overlays (Preus 1971). Polymer cement concrete and polymer-fly ash concrete provide better resistance to wear, although at the sacrifice of skid resistance. Steel fiber concrete overlays were also tested and showed reduced wear. Abraded, loose steel fibers can produce additional scour, and the exposed fibers can adversely affect the tire wear.

Although the reported test results show promise, no affordable concrete surface has yet been developed that will provide the same service life, when studded tires are used, as concrete surfaces exposed to rubber tire wear. A report (Transportation Research Board 1975) summarizes available data on pavement wear and on the performance and winter accident records for studded tire use.

8.6—Skid resistance of pavements

The skid resistance of concrete pavements depends on the surface texture of the concrete. There are two types of surface texture:

1. Macrotexture from surface irregularities that are built in at the time of construction; and
2. Microtexture from the type and hardness of fine aggregate.

The microtexture is more important at speeds of less than approximately 80 km/h (50 mph) (Kummer and Meyer 1967; Murphy 1975; Wilk 1978). At speeds greater than 80 km/h

(50 mph), the macrotexture becomes quite important because it is relied on to help prevent hydroplaning.

The skid resistance of concrete pavement initially depends on the texture built into the surface layer (Dahir 1981). In time, rubber-tire traffic abrades the surface paste, which removes the beneficial macrotexture and exposes the coarse- and fine-aggregate particles. The rate that the surface paste is removed and the consequences on the skid resistance of a pavement depends on the depth and quality of the surface paste and the rock type (toughness) of the fine and coarse aggregate.

Fine aggregates containing significant amounts of silicate minerals in the larger particle sizes will assist in slowing down surface wear and maintaining the microtexture necessary for satisfactory skid resistance at slow speed. Certain rock types, however, polish under rubber-tire wear. These include very fine-textured limestones, dolomites, and serpentine; the finer the texture, the more rapid the polishing. Where both the fine and coarse aggregate are made of these rock types, there may be a rapid polishing of the entire pavement surface and a serious reduction in skid resistance. Where only the coarse aggregate is of the polishing type, the problem is delayed until the coarse aggregate is exposed by wear. On the other hand, if the coarse aggregate is, for example, a coarse-grained silica or vesicular slag, the skid resistance may increase when the aggregate is exposed.

Macrotexture is very important because it prevents hydroplaning. An example of constructing macrotexture in pavement surfaces is placing grooves in the concrete—either before hardening or by sawing after the concrete has hardened—to provide channels for the escape of water that is otherwise trapped between the tire and the pavement. It is vital that the space between grooves be especially resistant to surface abrasion and frost action. A high-quality concrete that is properly finished and cured has the required durability and abrasion resistance.

CHAPTER 9—REFERENCES

9.1—Referenced standards and reports

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

American Concrete Institute

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| 201.2R | Guide to Durable Concrete |
| 207.1R | Mass Concrete |
| 207.2R | Report on Thermal and Volume Change Effects on Cracking of Mass Concrete |
| 210R | Erosion of Concrete in Hydraulic Structures |
| 210.1R | Compendium of Case Histories on Repair of Erosion-Damaged Concrete in Hydraulic Structures |
| 211.1 | Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete |
| 211.2 | Standard Practice for Selection Proportions for Structural Lightweight Concrete |
| 212.3R | Chemical Admixtures for Concrete |

216.1	Code Requirements for Determining Fire Resistance of Concrete and Masonry Construction Assemblies	C418	Test Method for Abrasion Resistance of Concrete by Sandblasting
221R	Guide for Use of Normal Weight and Heavy-weight Aggregates in Concrete	C441	Test Method for Effectiveness of Pozzolans or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction
221.1R	Report on Alkali-Aggregate Reactivity		
222R	Protection of Metals in Concrete Against Corrosion		
222.1	Provisional Standard Test Method for Water-Soluble Chloride Available for Corrosion of Embedded Steel in Mortar and Concrete Using the Soxhlet Extractor	C452	Test Method for Potential Expansion of Portland-Cement Mortars Exposed to Sulfate
		C457	Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete
222.3R	Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures	C586	Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock-Cylinder Method)
224R	Control of Cracking in Concrete Structures	C595	Specification for Blended Hydraulic Cements
224.1R	Causes, Evaluation, and Repair of Cracks in Concrete Structures	C618	Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
232.1R	Use of Raw or Processed Natural Pozzolans in Concrete	C666/C666M	Test Method for Resistance of Concrete to Rapid Freezing and Thawing
232.2R	Use of Fly Ash in Concrete	C672/C672M	Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
233R	Slag Cement in Concrete and Mortar	C779/C779M	Test Method for Abrasion Resistance of Horizontal Concrete Surfaces
234R	Guide for the Use of Silica Fume in Concrete	C989	Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars
302.1R	Guide for Concrete Floor and Slab Construction	C1012	Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution
304R	Guide for Measuring, Mixing, Transporting, and Placing Concrete	C1105	Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction
305R	Hot Weather Concreting	C1138	Test Method for Abrasion Resistance of Concrete (Underwater Method)
306R	Cold Weather Concreting	C1152/ C1152M	Test Method for Acid-Soluble Chloride in Mortar and Concrete
308R	Guide to Curing Concrete	C1157	Performance Specification for Hydraulic Cement
309R	Guide for Consolidation of Concrete	C1202	Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
318	Building Code Requirements for Structural Concrete and Commentary	C1218/ C1218M	Test Method for Water-Soluble Chloride in Mortar and Concrete
350	Code Requirements for Environmental Engineering Concrete Structures and Commentary	C1240	Specification for Silica Fume Used in Cementitious Mixtures
357.1R	Report on Offshore Concrete Structures for the Arctic	C1260	Test Method for Potential Alkali Reactivity of Aggregates (Mortar Bar Method)
515.1R	Guide to the Use of Waterproofing, Damp-proofing, Protective, and Decorative Barrier Systems for Concrete	C1293	Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction
<i>ASTM International</i>		C1567	Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar Bar Method)
C33	Specification for Concrete Aggregates	C1580	Test Method for Water-Soluble Sulfate in Soil
C88	Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate		
C114	Test Methods for Chemical Analysis of Hydraulic Cement		
C150	Specification for Portland Cement		
C227	Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar Bar Method)		
C260	Specification for Air-Entraining Admixtures for Concrete		
C289	Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)		
C295	Guide for Petrographic Examination of Aggregates for Concrete		
C309	Specification for Liquid Membrane-Forming Compounds for Curing Concrete		
C311	Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use in Portland-Cement Concrete		
		<i>Canadian Standards Association</i>	
		CSA A23.1-94	Concrete Materials and Methods of Concrete Construction

- CSA A23.2-94 Methods of Test for Concrete
 CSA A23.2-14A Test for Alkali-Aggregate Reaction
 CSA A23.2-26A Concrete Materials and Methods of Concrete Construction/Methods of Test for Concrete
 CSA A23.2-27A Standard Practice to Identify Degree of Alkali-Reactivity of Aggregates and to Identify Measures to Avoid Deleterious Expansion in Concrete
 CSA A23.2-28A Standard Practice for Laboratory Testing to Demonstrate the Effectiveness of Supplementary Cementing Materials and Lithium-Based Admixtures to Prevent Alkali-Silica Reaction in Concrete
 CSA A13.2-25A Accelerated Mortar Bar Test

German Institute of Standardization

- DIN 4030 Assessment of Water, Soil, and Gases for Their Aggressiveness to Concrete: Principles and Limiting Values

These publications may be obtained from these organizations:

American Concrete Institute
 P.O. Box 9094
 Farmington Hills, MI 48333-9094
www.concrete.org

ASTM International
 100 Barr Harbor Drive
 West Conshohocken, PA 19428
www.astm.org

Canadian Standards Association
 178 Rexdale Blvd.
 Etobicoke (Toronto), ON M9W 1R3
 Canada
www.csa.ca

German Institute of Standardization
 DIN Deutsches Institut für Normung e.V.
 10772 Berlin
 Germany
www.din.de

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APPENDIX A—METHOD FOR PREPARING EXTRACT FOR ANALYSIS OF WATER-SOLUBLE SULFATE IN SOIL

Loss on ignition

Determine the mass of a representative portion of soil (approximately 100 g) in a tared ceramic crucible. Record mass of crucible plus sample. Dry at 110 °C overnight. Remove and desiccate until cool. Record dry mass. Calculate loss on ignition as follows:

$$\% \text{LOI } 100^\circ \text{C} = 100 \times \frac{[\text{mass of sample + crucible before drying(g)} - \text{mass of sample + crucible after drying(g)}]}{[\text{mass of sample before drying(g)}]}$$

Grinding

Grind dried soil sample to pass a 600 µm sieve.

Extraction

Determine the mass of a 10 g sample of dried soil to the nearest milligram. Record mass of sample. Put soil in a 400 mL beaker. Add a stir bar. Add 200 mL of room temperature deionized water. Cover beaker with a watch glass. Stir using a mechanical stir plate for 1 hour, making sure that all soil is suspended in the water. Filter the solution through dry double Whatman 40 filter paper using a Buchner funnel into a 500 mL sidearm flask. If the solution is cloudy, refilter through a Whatman 42 or Millipore filter. Transfer without rinses to a 250 mL Nalgene bottle.



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