

Guide to Durable Concrete

Reported by ACI Committee 201

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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, all important considerations to ensure concrete durability. Some guidance as to repair techniques is also provided.

This document contains substantial revisions to Section 2.2 (chemical sulfate attack) and also includes a new section on physical salt attack (Section 2.3). The remainder of this document is essentially identical to the previous "Guide to Durable Concrete." However, all remaining sections of this document are in the process of being revised and updated, and these revisions will be incorporated into the next published version of this guide.

Both terms water-cement ratio and water-cementitious materials ratio are used in this document. Water-cement ratio is used (rather than the newer term, water-cementitious materials ratio) when the recommendations are based on data referring to water-cement ratio. If cementitious materials other than portland cement have been included in the concrete, judgment regarding required water-cement ratios have been based on the use of that ratio. This does not imply that new data demonstrating concrete performance developed using portland cement and other cementitious materials should not be referred to in terms of water-cementitious materials. Such information, if available, will be included in future revisions.

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INTRODUCTION

Durability of hydraulic-cement concrete is defined as 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. Some excellent general references on the subject are available (Klieger 1982; Woods 1968).

This guide discusses the more important causes of concrete deterioration and gives recommendations on how to prevent such damage. Chapters on freezing and thawing, aggressive chemical exposure, abrasion, corrosion of metals, chemical reactions of aggregates, repair of concrete, and the use of protective-barrier systems to enhance concrete durability are included. Fire resistance of concrete and cracking are not covered, because they are covered in ACI 216, ACI 224R, and ACI 224.1R, respectively.

Freezing and thawing in the temperate regions of the world can cause severe deterioration of concrete. Increased use of concrete in countries with hot climates has drawn attention to the fact that deleterious chemical processes, such as corrosion and alkali-aggregate reactions, are aggravated by high temperatures. Also, the combined effects of cold winter and hot summer exposures should receive attention in proportioning and making of durable concrete.

Water is required for the chemical and most physical processes to take place in concrete, both the desirable ones and the deleterious. Heat provides the activation energy that makes the processes proceed. The integrated effects of water and heat, and other environmental elements are important and should be considered and monitored. Selecting appropriate materials of suitable composition and processing them correctly under existing environmental conditions is essential to achieve concrete that is resistant to deleterious effects of water, aggressive solutions, and extreme temperatures.

Freezing-and-thawing damage is fairly well understood. The damage is accelerated, particularly in pavements by the use of deicing salts, often resulting in severe scaling at the surface. Fortunately, concrete made with quality aggregates, low water-cement ratio (w/c), proper air-void system, and allowed to mature before being exposed to severe freezing and thawing is highly resistant to such damage.

Sulfates in soil, groundwater, or seawater are resisted by using suitable cementitious materials and a properly proportioned concrete mixture subjected to proper quality control. 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 will resist occasional exposure to mild acids, but no concrete offers good resistance to attack by strong acids or compounds that convert to acids; special protection is necessary in these cases.

Abrasion can cause concrete surfaces to wear away. Wear can be a particular problem in industrial floors. In hydraulic structures, particles of sand or gravel in flowing water can erode surfaces. The use of high-quality concrete and, in extreme cases, a very hard aggregate, will usually result in adequate durability under these exposures. The use of studded tires on automobiles has caused serious wear in concrete pavements; conventional concrete will not withstand this damage.

The spalling of concrete in bridge decks is a serious problem. The principal cause of reinforcing-steel corrosion is mainly due to the use of deicing salts. The corrosion produces an expansive force that causes the concrete to spall above the steel. Ample cover over the steel and use of a low-permeability, air-entrained concrete will ensure durability in the majority of cases, but more positive protection, such as epoxy-coated reinforcing steel, cathodic protection, or chemical corrosion inhibitors, is needed for severe exposures.

Although aggregate is commonly considered to be an inert filler in concrete, that is not always the case. Certain aggregates can react with alkalis in cement, causing expansion and deterioration. Care in the selection of aggregate sources and the use of low-alkali cement, pretested pozzolans, or ground slag will alleviate this problem.

The final chapters of this report discuss the repair of concrete that has not withstood the forces of deterioration and the use of protective-barrier systems to enhance durability.

The use of good materials and proper mixture proportioning will not ensure durable concrete. Quality control and workmanship are also absolutely essential to the production of durable concrete. Experience has shown that two areas should receive special attention: 1) control of entrained air and 2) finishing of slabs. ACI 311.1R describes good concrete practices and inspection procedures. ACI 302.1R describes in detail proper practice for consolidating and finishing floors and slabs. ACI 325.9R reviews pavement installation. ACI 330R discusses parking lot concrete, and ACI 332R covers residential concrete, including driveways and other flatwork.

CHAPTER 1—FREEZING AND THAWING

1.1—General

Exposing damp concrete to freezing-and-thawing cycles is a severe test for concrete to survive without impairment. Air-entrained concrete, which is properly proportioned with quality materials, manufactured, placed, finished, and cured, resists cyclic freezing for many years.

Under extremely severe conditions, however, even quality concrete can suffer damage from cyclic freezing, for example, if it is kept in a state of nearly complete saturation. This situation may be created when cold concrete is exposed to warmer, moist air on one side and evaporation is insufficient or restricted on the cold side, or when the concrete is subjected to a head of water for a period of time before freezing.

A general discussion on the subject of frost action in concrete is provided by Cordon (1966).

1.2—Mechanisms of frost action

Powers and his associates conducted extensive research on frost action in concrete from 1933 to 1961. They developed reasonable hypotheses to explain the rather complex mechanisms. Hardened cement paste and aggregate behave quite differently when subjected to cyclic freezing and are considered separately.

1.2.1 Freezing in cement paste—In his early papers, Powers (1945, 1954, 1955, 1956) attributed frost damage in cement paste to stresses caused by hydraulic pressure in the pores. The pressure was due to resistance to water movement away from the regions of freezing. It was believed that the magnitude of the pressure depended on the rate of freezing, degree of saturation, coefficient of permeability of the paste, and the length of the flow-path to the nearest place for the water to escape. The benefits of entrained air were explained in terms of the shortening of flow-paths to places of escape. Some authorities still accept this hypothesis.

Later studies by Powers and Helmuth produced strong evidence that the hydraulic pressure hypothesis was not consistent with experimental results (Powers 1956, 1975; Helmuth 1960a, 1960b; Pickett 1953). They found that during freezing of cement paste most of the water movement is toward, not away from, sites of freezing, as had been previously believed. Also, the dilations (expansions) during freezing generally decreased with an increased rate of cooling. Both of these findings were contrary to the hydraulic pressure hypothesis and indicated that a modified form of a theory previously advanced by Collins (1944) (originally developed to explain frost action in soil) is applicable.

Powers and Helmuth pointed out that the water in cement paste is in the form of a weak alkali solution. When the temperature of the concrete drops below the freezing point, there is an initial period of supercooling, after which ice crystals will form in the larger capillaries. This results in an increase in alkali content in the unfrozen portion of the solution in these capillaries, creating an osmotic potential that impels water in the nearby unfrozen pores to begin diffusing into the solution in the frozen cavities. The resulting dilution of the solution in contact with the ice allows further growth of the body of ice (ice-accretion). When the cavity becomes full of ice and solution, any further ice-accretion produces dilative pressure, which can cause the paste to fail. When water is being drawn out of unfrozen capillaries, the paste tends to shrink. (Experiments have verified that shrinkage of paste or concrete occurs during part of the freezing cycle.)

According to Powers, when the paste contains entrained air and the average distance between air bubbles is not too great, the bubbles compete with the capillaries for the unfrozen water and normally win this competition. For a better understanding of the mechanisms involved, the reader is directed to the references previously cited. Many researchers now believe that stresses resulting from osmotic pressure cause most of the frost damage to cement paste.

Litvan (1972) has further studied frost action in cement paste. Litvan believes that the water adsorbed on the surface or contained in the smaller pores cannot freeze due to the interaction between the surface and the water. Because of the difference in vapor pressure of this unfrozen and supercooled liquid and the bulk ice in the surroundings of the paste system, there will be migration of water to locations where it is able to freeze, such as the larger pores or the outer surface. The process leads to partial desiccation of the paste and accumulation of ice in crevices and cracks. Water in this location freezes, prying the crack wider, and if the space fills with water in the next thaw portion of the cycle, further internal pressure and crack opening results. Failure occurs when the required redistribution of water cannot take place in an orderly fashion either because the amount of water is too large, that is, high w/cm for the same level of saturation, the available time is too short (rapid cooling), or the path of migration is too long (lack of entrained air bubbles). Litvan believes that in such cases the freezing forms a semi-amorphous solid (noncrystalline ice), resulting in great internal stresses. Additional stresses can be created by the nonuniform moisture distribution.

There is general agreement that cement paste of adequate strength and maturity can be made completely immune to damage from freezing by means of entrained air, unless unusual exposure conditions result in filling of the air voids. Air entrainment alone, however, does not preclude the possibility of damage of concrete due to freezing, because freezing in aggregate particles should also be taken into consideration.

1.2.2 Freezing in aggregate particles—Most rocks have pore sizes much larger than those in cement paste, and Powers (1945) found that they expel water during freezing. The hydraulic pressure theory, previously described for cement paste, plays a major role in most cases.

Dunn and Hudec (1965) advanced the ordered-water theory, which states that the principal cause of deterioration of rock is not freezing but the expansion of adsorbed water (which is not freezable); specific cases of failure without freezing of clay-bearing limestone aggregates seemed to support this conclusion. This, however, is not consistent with the results of research by Helmuth (1961) who found that adsorbed water does not expand but actually contracts during cooling. Nevertheless, Helmuth agrees that the adsorption of large amounts of water in aggregates with a very fine pore structure can disrupt concrete through ice formation. The size of the coarse aggregate has been shown to be an important factor in its frost resistance. Verbeck and Landgren (1960) have demonstrated that, when unconfined by cement paste, the ability of natural rock to withstand freezing and thawing without damage increases with a decrease in size, and that there is a critical size below which rocks can be frozen without damage. They showed that the critical size of some rocks can be as small as a 1/4 in. (6 mm). Some aggregates (such as granite, basalt, diabase, quartzite, and marble) capacities for freezable water is so low that they do not produce stress when freezing occurs under commonly experienced conditions, regardless of the particle size.

Various properties related to the pore structure within the aggregate particles, such as absorption, porosity, pore size, and pore distribution or permeability, can be indicators of potential durability problems when the aggregates are used in concrete that become saturated and freeze in service. Generally, it is the coarse aggregate particles with relatively high porosity or absorption values, caused principally by medium-sized pore spaces in the range of 0.1 to 5 μm , that are most easily saturated and contribute to deterioration of concrete individual popouts. Larger pores usually do not get completely filled with water, therefore, damage is not caused by freezing. Water in very fine pores may not freeze as readily (ACI 221R). Fine aggregate is generally not a problem, because the particles are small enough to be below the critical size for the rock type and the entrained air in the surrounding paste can provide an effective level of protection (Gaynor 1967).

The role of entrained air in alleviating the effect of freezing in coarse aggregate particles is minimal.

1.2.3. Overall effects in concrete—Without entrained air, the paste matrix surrounding the aggregate particles can fail when it becomes critically saturated and is frozen. If the matrix contains an appropriate distribution of entrained air voids characterized by a spacing factor less than about 0.008 in. (0.20 mm), freezing does not produce destructive stress (Verbeck 1978).

There are some rocks that contain practically no freezable water. Air-entrained concrete made with an aggregate composed entirely of such rocks will withstand freezing for a long time, even under continuously wet exposures. This time can be shortened if the air voids fill with water and solid matter.

If absorptive aggregates, such as certain cherts and light-weight aggregates, are used and the concrete is in a continuously wet environment, the concrete will probably fail if the coarse aggregate becomes saturated (Klieger and Hanson 1961). The internal pressure developed when the particles expel water during freezing ruptures the particles and the matrix. If the particle is near the concrete surface, a popout can result.

Normally, aggregate in concrete is not in a critical state of saturation near the end of the construction period because of desiccation produced by the chemical reaction during hardening (self-desiccation of the cement paste) and loss by evaporation. Therefore, if any of the aggregate ever becomes critically saturated, it will be by water obtained from an outside source. Structures so situated that all exposed surfaces are kept continuously wet, and yet are periodically subject to freezing, are uncommon. Usually concrete sections tend to dry out during dry seasons when at least one surface is exposed to the atmosphere. That is why air-entrained concrete generally is not damaged by frost action, even where absorptive aggregate is used.

Obviously, the drier the aggregate is at the time the concrete is cast, the more water it must receive to reach critical saturation and the longer it will take. This is an important consideration, because the length of the wet and cold season is limited. It can prove a disadvantage to use gravel directly from an underwater source, especially if the structure goes

into service during the wet season or shortly before the beginning of winter.

Some kinds of rock, when dried and then placed in water, are able to absorb water rapidly and reach saturation quickly; they are described as readily saturable. This type, even when dry at the start, can reach high levels of saturation while in a concrete mixer and might not become sufficiently dried by self-desiccation; hence, with such a material trouble is in prospect if there is not a sufficiently long dry period before the winter season sets in. A small percentage of readily saturable rocks in an aggregate can cause serious damage. Rocks that are difficult to saturate, which are generally coarse grained, are less likely to cause trouble. Obviously, data on the absorption characteristic of each kind of rock in an aggregate is useful.

1.3—Ice-removal agents

When the practice of removing ice from concrete pavements by means of salt (sodium chloride, calcium chloride, or both) became common, it was soon learned that these materials caused or accelerated surface disintegration in the form of pitting or scaling. (These chemicals also accelerate the corrosion of reinforcement, which can cause the concrete to spall, as described in [Chapter 4](#).)

The mechanism by which deicing agents damage concrete is fairly well understood and is primarily physical rather than chemical. The mechanism involves the development of disruptive osmotic and hydraulic pressures during freezing, principally in the paste, similar to ordinary frost action, which is described in [Section 1.2](#). It is, however, more severe.

The concentration of deicer in the concrete plays an important role in the development of these pressures. Verbeck and Klieger (1957) showed that scaling of the concrete is greatest when ponded with intermediate concentrations (3 to 4%) of deicing solutions. Similar behavior was observed for the four deicers tested: calcium chloride, sodium chloride, urea, and ethyl alcohol. Browne and Cady (1975) drew similar conclusions. Litvan's findings (1975, 1976) were consistent with the studies just mentioned. He further concluded that deicing agents cause a high degree of saturation in the concrete, and that this is mainly responsible for their detrimental effect. Salt solutions (at a given temperature) have a lower vapor pressure than water; therefore, little or no drying takes place between wetting (see [Section 1.2.3](#)) and cooling cycles. ASTM C 672 determines the resistance of a given concrete mixture to resist scaling in the presence of deicing chemicals.

The benefit from entrained air in concrete exposed to deicers is explained in the same way as for ordinary frost action. Laboratory tests and field experience have confirmed that air entrainment greatly improves resistance to deicers and is essential under severe conditions to consistently build scale-resistant pavements.

1.4—Recommendations for durable structures

Concrete that will be exposed to a combination of moisture and cyclic freezing requires the following:

- Design of the structure to minimize exposure to moisture;

- Low w/cm ;
- Appropriate air entrainment;
- Quality materials;
- Adequate curing before first freezing cycle; and
- Special attention to construction practices.

These items are described in detail in the following paragraphs.

1.4.1 Exposure to moisture—Because the vulnerability of concrete to cyclic freezing is greatly influenced by the degree of saturation of the concrete, precautions should be taken to minimize water uptake in the initial design of the structure.

The geometry of the structure should promote good drainage. Tops of walls and all outer surfaces should be sloped. Low spots conducive to the formation of puddles should be avoided. Weep holes should not discharge over the face of exposed concrete. Drainage from higher ground should not flow over the top or faces of concrete walls (Miesenhelder 1960).

Joints not related to volume change should be eliminated. Provisions for drainage, such as drip beads, can prevent water from running under edges of structural members. Water traps or reservoirs, which can result from extending diaphragms to the bent caps of bridges, should be avoided during design.

Even though it is seldom possible to keep moisture from the underside of slabs on grade, subbase foundations incorporating the features recommended in ACI 325.9R will minimize moisture buildup. Care should also be taken to minimize cracks that can collect or transmit water.

Extensive surveys of concrete bridges and other structures have shown a striking correlation between freezing and thawing damage of certain portions and excessive exposure to moisture of these portions due to the structural design (Callahan et al. 1970; Jackson 1946; Lewis 1956).

1.4.2 Water-cement ratio—Frost-resistant normalweight concrete should have a w/cm not exceeding the following: thin sections (bridge decks, railings, curbs, sills, ledges, and ornamental works) and any concrete exposed to deicing salts, w/cm of 0.45; all other structures, w/cm of 0.50.

Because the degree of absorption of some lightweight aggregates may be uncertain, it is impracticable to calculate the w/cm of concretes containing such aggregates. For these concretes, a 28 day compressive strength of at least 4000 psi (27.6 MPa) should be specified.

1.4.3 Entrained air—Too little entrained air will not protect cement paste against freezing and thawing. Too much air will penalize the strength. Recommended air contents of concrete are given in [Table 1.1](#).

Air contents are given for two conditions of exposure: severe and moderate. These values provide approximately 9% of air in the mortar fraction for severe exposure and approximately 7% for moderate exposure.

Air-entrained concrete is produced through the use of an air-entraining admixture added to the concrete mixer, air-entraining cement, or both. The resulting air content depends on many factors, including the properties of the materials being used (cement, chemical admixtures, aggregates, pozzolans), mixture proportions, types of mixer, mixing time, and temperature. Where an air-entraining admixture is used,

Table 1.1—Recommended air contents for frost-resistant concrete

Nominal maximum aggregate size, in. (mm)	Average air content, %*	
	Severe exposure [†]	Moderate exposure [‡]
3/8 (9.5)	7-1/2	6
1/2 (12.5)	7	5-1/2
3/4 (19.0)	6	5
1 (25.0)	6	5
1-1/2 (37.5)	5-1/2 [§]	4-1/2 [§]
3 (75)	4-1/2 [§]	3-1/2 [§]
6 (150)	4	3

*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.

[§]These air contents apply to the whole as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 1-1/2 in. (37.5 mm) is removed by handpicking or sieving and the air content is determined on the minus 1-1/2 in. (37.5 mm) fraction of the mixture. (The field tolerance applies to this value.) From this, the air content of the whole mixture is computed.

Note: There is conflicting opinion on whether air contents lower than those given in the table should be permitted for high-strength (approximately 5500 psi) (37.8 MPa) concrete. This 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 1-1/2 in. (37.5 mm), this reduction applies to the minus 1-1/2 in. (37.5 mm) fraction of the mixture.)

the dosage is varied as necessary to give the desired air content. This is not possible where an air-entraining cement alone is used, and occasionally the air content will be inadequate or excessive. Nevertheless, this is the most convenient method for providing some assurance of protection from cyclic freezing on small jobs where equipment to check the air content is not available. The preferred procedure is to use an air-entraining admixture.

Samples for air content determination should be taken as close to the point of placement as feasible. Frequency of sampling should be as specified in ASTM C 94. For normal-weight concrete, the following test methods may be used: volumetric method (ASTM C 173), pressure method (ASTM C 231), or the unit weight test (ASTM C 138). The unit weight test (ASTM C 138) can be used to check the other methods. For lightweight concrete, the volumetric method (ASTM C 173) should be used.

The air content and other characteristics of an air-void system in hardened concrete can be determined microscopically (ASTM C 457). ACI 212.3R lists the air-void characteristics required for durability. ASTM C 672 provides a method to assess the resistance of concrete to deicer scaling.

1.4.4 Materials

1.4.4.1 Cementitious materials—The different types of portland and blended hydraulic cements, when used in properly proportioned and manufactured air-entrained concrete, provide similar resistance to cyclic freezing. Cement should conform to ASTM C 150 or C 595.

Most fly ashes and natural pozzolans, when used as admixtures, have little effect on the durability of concrete, provided that the air content, strength, and moisture content of the concrete are similar. A suitable investigation, however,

should be made before using unproven materials. Fly ashes and natural pozzolans should conform to ASTM C 618. Ground-granulated blast-furnace slag should conform to ASTM C 989. In continental European countries (Belgium, the Netherlands, France, and Germany) blast-furnace-slag cements have been used successfully for over a century in concrete exposed to severe freezing and thawing environments, including marine exposures.

1.4.4.2 Aggregates—Natural aggregates should meet the requirements of ASTM C 33; although, this will not necessarily ensure their durability. Lightweight aggregates should meet the requirements of ASTM C 330. These specifications provide many requirements but leave the final selection of the aggregate largely up to the judgment of the engineer. If the engineer is familiar with the field performance of the proposed aggregate, his or her judgment should be adequate. In some situations, it is possible to carry out field service record studies to arrive at a basis for acceptance or rejection of the aggregate. When this is not feasible, heavy reliance must be placed on cautious interpretations of laboratory tests.

Laboratory tests on the aggregate include absorption, specific gravity, soundness, and determination of the pore structure. Descriptions of the tests and opinions on their usefulness have been published (Newlon 1978; Buth and Ledbetter 1970). Although these data are useful, and some organizations have felt justified in setting test limits on aggregates, it is generally agreed that principal reliance should be placed on tests on concrete made with the aggregate in question.

Petrographic studies of both the aggregate (Mielenz 1978) and concrete (Erlin 1966; Mather 1978a) are useful for evaluating the physical and chemical characteristics of the aggregate and concrete made with it.

Laboratory tests on concrete include the rapid freezing and thawing tests (ASTM C 666), in which the durability of the concrete is measured by the reduction in dynamic modulus of elasticity of the concrete. ASTM C 666 permits testing by either Procedure A, freezing and thawing in water, or Procedure B, freezing in air and thawing in water.

The results of tests using ASTM C 666 have been widely analyzed and discussed (Arni 1966; Buth and Ledbetter 1970; ACI 221R; Transportation Research Board 1959). These tests have been criticized because they are accelerated tests and do not duplicate conditions in the field. Test specimens are initially saturated, which is not normally the case for field concrete at the beginning of the winter season. Furthermore, the test methods do not realistically duplicate the actual moisture conditions of the aggregates in field concrete. The rapid methods have also been criticized because they require cooling rates greater than those encountered in the field. Also, the small test specimens used are unable to accommodate larger aggregate sizes proposed for use, which may be more vulnerable to popout and general deterioration than smaller sizes. The presence of a piece of popout producing aggregate in the central portion of the relatively small test specimens can cause some of these specimens to fail, whereas the popout material would only cause superficial surface defects in in-service concrete (Sturup et al. 1987).

It is generally conceded that while these various tests may classify aggregates from excellent to poor in approximately the correct order, they are unable to predict whether a marginal aggregate will give satisfactory performance when used in concrete at a particular moisture content and subjected to cyclic freezing exposure. The ability to make such a determination is of great economic importance in many areas where high-grade aggregates are in short supply, and local marginal aggregates can be permitted. Despite the shortcomings of ASTM C 666, many agencies believe that this is the most reliable indicator of the relative durability of an aggregate (Sturup et al. 1987).

Because of these objections to ASTM C 666, a dilation test was conceived by Powers (1954) and further developed by others (Harman et al. 1970; Tremper and Spellman 1961). ASTM C 671 requires that air-entrained concrete specimens be initially brought to the moisture condition expected for the concrete at the start of the winter season, with the moisture content preferably having been determined by field tests. The specimens are then immersed in water and periodically frozen at the rate to be expected in the field. The increase in length (dilation) of the specimen during the freezing portion of the cycle is measured. ASTM C 682 assists in interpreting the results.

Excessive length change in this test is an indication that the aggregate has become critically saturated and vulnerable to damage. If the time to reach critical saturation is less than the duration of the freezing season at the job site, the aggregate is judged unsuitable for use in that exposure. If it is more, it is judged that the concrete will not be vulnerable to cyclic freezing.

The time required for conducting a dilation test may be greater than that required to perform a test by ASTM C 666. Also, the test results are very sensitive to the moisture content of the aggregate and concrete. Despite these shortcomings, most reported test results are fairly promising. Although most agencies are continuing to use ASTM C 666, results from ASTM C 671 may turn out to be more useful (Philleo 1986).

When a natural aggregate is found to be unacceptable by service records, tests, or both, it may be improved by removal of lightweight, soft, or otherwise inferior particles.

1.4.4.3 Admixtures — Air-entraining admixtures should conform to ASTM C 260. Chemical admixtures should conform to ASTM C 494. Admixtures for flowing concrete should conform to ASTM C 1017.

Some mineral admixtures, including pozzolans, and aggregates containing large amounts of fines may require a larger amount of air-entraining admixture to develop the required amount of entrained air. Detailed guidance on the use of admixtures is provided by ACI 212.3R.

1.4.5 Maturity—Air-entrained concrete should withstand the effects of freezing as soon as it attains a compressive strength of about 500 psi (3.45 MPa), provided that there is no external source of moisture. At a temperature of 50 F (10 C), most well-proportioned concrete will reach this strength some time during the second day.

Before being exposed to extended freezing while critically saturated (ASTM C 666), the concrete should attain a compressive strength of about 4000 psi (27.6 MPa). A period of drying following curing is advisable. For moderate exposure conditions, a strength of 3000 psi (20.7 MPa) should be attained (Kleiger 1956).

1.4.6 Construction practices—Good construction practices are essential when durable concrete is required. Particular attention should be given to the construction of pavement slabs that will be exposed to deicing chemicals because of the problems inherent in obtaining durable slab finishes and the severity of the exposure. The concrete in such slabs should be adequately consolidated; however, overworking the surface, overfinishing, and the addition of water to aid in finishing must be avoided. These activities bring excessive mortar or water to the surface, and the resulting laitance is particularly vulnerable to the action of deicing chemicals. These practices can also remove entrained air from the surface region. This is of little consequence if only the larger air bubbles are expelled, but durability can be seriously affected if the small bubbles are removed. Timing of finishing is critical (ACI 302.1R).

Before the application of any deicer, pavement concrete should have received some drying, and the strength level specified for the opening of traffic should be considered in the scheduling of late fall paving. In some cases, it may be possible to use methods other than ice-removal agents, such as abrasives, for control of slipperiness when the concrete is not sufficiently mature.

For lightweight concrete, do not wet the aggregate excessively before mixing. Saturation by vacuum or thermal means (for example, where necessary for pumping) can bring lightweight aggregates to a moisture level at which the absorbed water will cause concrete failure when it is cyclically frozen, unless the concrete has the opportunity to dry before freezing. Additional details and recommendations are given in a publication of the California Department of Transportation (1978).

CHAPTER 2—AGGRESSIVE CHEMICAL EXPOSURE

2.1—General

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 under which the useful life of even the best concrete will be short, 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.

Table 2.1—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) Sulfate liquor
Slow	Carbonic	—	Sodium hydroxide* 10 to 20%	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 chromate	Ammonia (liquid)

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

Table 2.2—Factors influencing chemical attack on concrete

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

Comprehensive tables have been prepared by ACI Committee 515 (515.1R) and the Portland Cement Association (1968) giving the effect of many chemicals on concrete. Biczok (1972) gives a detailed discussion of the deteriorating effect of chemicals on concrete, including data both from Europe and the U.S.

The effects of some common chemicals on the deterioration of concrete are summarized in Table 2.1. 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 2.2. Therefore, Table 2.1 should be considered as only a preliminary guide.

Major areas of concern are exposure to sulfates, seawater, salt from seawater, acids, and carbonation. These areas of concern are discussed in Sections 2.2 through 2.6.

2.2—Chemical sulfate attack by sulfate from sources external to the concrete

2.2.1 Occurrence — Naturally occurring sulfates of sodium, potassium, calcium, or magnesium,¹ that can attack hardened concrete, are sometimes found in soil or dissolved in ground-water 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; Reading 1982; USBR 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 build-up of sulfates due to evaporation, particularly where such systems use relatively small amounts of make-up water. Sulfate ions can also be present in fill containing industrial waste products, such as slags from iron processing, cinders, and groundwater leaching these materials.

¹Many 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 ₂ SO ₄
bassanite	CaSO ₄ · 1/2H ₂ O	mirabilite	Na ₂ SO ₄ · 10H ₂ O
gypsum	CaSO ₄ · 2H ₂ O	arcanite	K ₂ SO ₄
kieserite	MgSO ₄ · H ₂ O	glauberite	Na ₂ Ca(SO ₄) ₂
epsomite	MgSO ₄ · 7H ₂ O	langbeinite	K ₂ Mg ₂ (SO ₄) ₃
thaumasite	Ca ₃ Si(CO ₃)(SO ₄)(OH) ₁ · 12H ₂ O		

Seawater and coastal soil soaked with seawater constitute a special type of exposure. Recommendations for concrete exposed to seawater are in [Section 2.3](#).

2.2.2 Mechanisms—The two best recognized chemical consequences of sulfate attack on concrete components are the formation of ettringite (calcium aluminate trisulfate 32-hydrate, $\text{CaO}\cdot\text{Al}_2\text{O}_3\cdot 3\text{CaSO}_4\cdot 32\text{H}_2\text{O}$) and gypsum (calcium sulfate dihydrate, $\text{CaSO}_4\cdot 2\text{H}_2\text{O}$). 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 ($\text{Mg}(\text{OH})_2$, magnesium hydroxide) 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 diminish durability.

Publications discussing these mechanisms in detail include Lea (1971), Hewlett (1998), Mehta (1976, 1992), DePuy (1994), Taylor (1997), and Skalny et al. (1998). 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).

2.2.3 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 water to cementitious-materials ratio (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, ACI 302.1R, ACI 308.1, ACI 305R, and ACI 306R.

The sulfate resistance of portland cement generally decreases with an increase in its calculated tricalcium-aluminate (C_3A) content (Mather 1968). Accordingly, ASTM C 150 includes Type V sulfate-resisting cement for which a maximum of 5% calculated C_3A is permitted and Type II moderately sulfate-resisting cement for which the calculated C_3A is 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 C 150 provides that in Type V cement the $\text{C}_4\text{AF} + 2\text{C}_3\text{A}$ should not exceed 25%, unless the alternate requirement based on the use of the performance test (ASTM C 452) is invoked. In the case of Type V cement, the sulfate-expansion test (ASTM C 452) can be used in lieu of the chemical requirements (Mather 1978b). The use of ASTM C 1012 is discussed by Patzias (1991).

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 2.3](#). Both 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).

[Table 2.3](#) 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.

The field conditions of concrete exposed to sulfate are numerous and variable. The aggressiveness of the conditions depends, among others, on soil saturation, water movement, ambient temperature and humidity, concentration of sulfate, and type of sulfate or combination of sulfates involved. Depending on the above variables, solutions containing calcium sulfate are generally less aggressive than solutions of sodium sulfate, which is generally less aggressive than magnesium sulfate. [Table 2.3](#) provides criteria that should maximize the service life of concrete subjected to the more aggressive exposure conditions.

Portland-cement concrete can also be attacked by acidic solutions, such as sulfuric acid. Information on acid attack is provided in [Section 2.5](#).

2.2.4 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.²

2.2.5 Material qualification of pozzolans and slag for sulfate-resistance enhancement—Tests of one year's 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 Class 2 exposures using the 6-month criteria in [Sections 2.2.6](#) and [2.2.7](#).

Fly ashes, natural pozzolans, silica fumes, and slags may be qualified for sulfate resistance by demonstrating an expansion $\leq 0.10\%$ in one year when tested individually with portland cement by ASTM C 1012 in the following mixtures:

For fly ash or natural pozzolan, the portland-cement portion of the test mixture should consist of a cement with Bogue calculated C_3A ³ of not less than 7%. The fly ash or natural pozzolan proportion should be between 25 and

²If 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 either be 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.

³The C_3A 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 C 150.

Table 2.3—Requirements to protect against damage to concrete by sulfate attack from external sources of sulfate

Severity of potential exposure	Water-soluble soluble sulfate (SO ₄) [*]	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 [‡]	C 150 Type II or equivalent [§]
Class 2 exposure	0.20 to < 2.0	1500 to < 10,000	0.45 [‡]	C 150 Type V or equivalent [§]
Class 3 exposure	2.0 or greater	10,000 or greater	0.40 [‡]	C 150 Type V plus pozzolan or slag [§]
Seawater exposure	—	—	See Section 2.4	See Section 2.4

^{*}Sulfate expressed as SO₄ is related to sulfate expressed as SO₃, as given in reports of chemical analysis of portland cements as follows: SO₃% x 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 2.3.

[‡]These values are 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 2.2.5, 2.2.6, and 2.2.9. For Class 2 exposure, equivalents are described in Sections 2.2.5, 2.2.7, and 2.2.9. For Class 3 exposure, pozzolan and slag recommendations are described in Sections 2.2.5, 2.2.8, and 2.2.9.

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 C₃A³ 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 C₃A³ of not less than 7%. The slag proportion should be between 40 and 70% by mass, calculated as 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 one year from the date of test completion.

The reported calcium-oxide content⁴ 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 content⁴ 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.

2.2.6 Type II Equivalent for Class 1 Exposure

- A. ASTM C 150 Type III cement with the optional limit of 8% max. C₃A; C 595M Type IS(MS), Type IP(MS), Type IS-A(MS), Type IP-A(MS); C 1157 Type MS; or

³The 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 C 150.

⁴Analyzed in accordance with ASTM C 114.

- B. Any blend of portland cement of any type meeting ASTM C 150 or C 1157 with fly ash or natural pozzolan meeting ASTM C 618, silica fume meeting ASTM C 1240, or slag meeting ASTM C 989, that meets the following requirement when tested in accordance with ASTM C 1012. Any fly ash, natural pozzolan, silica fume, or slag used should have been previously qualified in accordance with Section 2.2.5.
- Expansion ≤ 0.10% at 6 months.

2.2.7 Type V Equivalent for Class 2 Exposure

- A. ASTM C 150 Type III cement with the optional limit of 5% max. C₃A; ASTM C 150 cement of any type having expansion at 14 days no greater than 0.040% when tested by ASTM C 452; ASTM C 1157 Type HS; or
- B. Any blend of portland cement of any type meeting ASTM C 150 or C 1157 with fly ash or natural pozzolan meeting ASTM C 618, silica fume meeting ASTM C 1240, or slag meeting ASTM C 989 that meets the following requirement when tested in accordance with ASTM C 1012:

Expansion ≤ 0.05% at 6 months. Any fly ash, natural pozzolan, silica fume, or slag used should have been previously qualified in accordance with Section 2.2.5 in order 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 2.2.5, then 1-year tests should be performed on the proposed combination and the expansion should comply with the following limit:

Expansion ≤ 0.10% at 1 year.

2.2.8 Class 3 Exposure—any blend of portland cement meeting ASTM C 150 Type V or C 1157 Type HS with fly ash or natural pozzolan meeting ASTM C 618, silica fume meeting ASTM C 1240, or slag meeting ASTM C 989, that

meets the following requirement when tested in accordance with ASTM C 1012:

Expansion $\leq 0.10\%$ at 18 months.

2.2.9 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 2.2.6, 2.2.7, or 2.2.8. 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 2.2.6, 2.2.7, or 2.2.8:

- *Fly ash*—reported calcium-oxide content⁵ no more than 2.0 percentage points higher than that of the fly ash used in the test mixture;
- *Slag*—reported aluminum-oxide content⁵ 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 2.2.6, 2.2.7, or 2.2.8.

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 2.3 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 Type I and Type II cement (ACI 232.2R; ACI 233R; ACI 234R). Some pozzolans, especially some Class C fly ashes, decrease the sulfate resistance of mortars in which they are used (Mather 1981b, 1982). Good results were obtained when the pozzolan was a

fly ash meeting the requirements of ASTM C 618 Class F (Dikeou 1975; Dunstan 1976). Slag should meet ASTM C 989.

In concrete made with nonsulfate-resisting cements, calcium chloride reduces resistance to attack by sulfate (USBR 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.

2.3—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), and Haynes and O'Neill (1994), Haynes et al. (1996), and Marchand and Skalny (1999). 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 the ASTM C 88. 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 is surface scaling 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 poor-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.

2.4—Seawater exposure

2.4.1 Seawater in various locations throughout the world has a range of concentration of total salts, it is less dilute 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 is especially high in shallow coastal areas with excessive daily evaporation rates.

⁵Analyzed in accordance with ASTM C 114.

Where concrete structures are placed on reclaimed coastal areas with the foundations below saline groundwater levels, capillary suction and evaporation may cause supersaturation and crystallization in the concrete above ground, resulting both in chemical attack on the cement paste (sulfate) and in aggravated corrosion of steel (chlorides).

In tropical climates these combined deleterious effects may cause severe defects in concrete in the course of a very few years.

2.4.2 The reaction of mature concrete with the sulfate ion in seawater is similar to that with sulfate ion in fresh water or leached from soils, but the effects are different (Mather 1966). The concentration of sulfate ions 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 given calculated C_3A content than would be expected of the same cement in a freshwater exposure where the water has the same sulfate-ion content. The performance of concretes continuously immersed in seawater made with ASTM C 150 cements having C_3A contents as high as 10% have proven satisfactory, provided the permeability of the concrete is low (Browne 1980). The Corps of Engineers (1994) permits, and the Portland Cement Association recommends, up to 10% calculated C_3A for concrete that will be permanently submerged in seawater if the w/c 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_4AF are concerned. Therefore, the interrelation between the measured C_3A content and the seawater resistance may be equally uncertain.

2.4.3 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/c , 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 1/10th or 1/100th that of comparable concrete of equal strength made without slag or pozzolan (Bakker 1980). The satisfactory performance of concretes containing ground slag in a marine environment has been described (Mather 1981a; Vanden Bosch 1980; and Lea 1971).

Concrete should be designed and constructed to minimize crack widths, therefore limiting seawater access to the reinforcement. Additionally, concrete should reach a maturity equivalent of not less than 5000 psi (35 MPa) at 28 days when fully exposed to seawater.

Conductive coatings applied at the time of construction as part of a cathodic-protection system can 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, that is, 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.

2.5—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.

2.5.1 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) which, upon oxidation, produces sulfuric acid. Further reaction can produce sulfate salts, which produce sulfate attack (Hagerman and Roosaar 1955; Lossing 1966; Bastiensen, Mourn, and Rosenquist 1957; Mourn and Rosenquist 1959).

Mountain streams are sometimes mildly acidic due to dissolved free carbon dioxide. Usually these waters 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 or hydrogen sulfide, or both, can seriously damage any concrete (RILEM 1962; Thornton 1978). In the case of hydrogen sulfide, bacteria that converts 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.

2.5.2 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 2.2.2](#).

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 4](#)), which will in turn cause cracking and spalling of the concrete.

2.5.3 Recommendations—A dense concrete with a low w/c 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. **Chapter 7** discusses the general principles involved in the use of such systems.

2.6—Carbonation

2.6.1 When concrete or mortar is exposed to carbon dioxide, 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 to which they occur, and the environmental exposure. On the one hand, 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 carbon dioxide (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 CO_2 can be either the atmosphere or water carrying dissolved CO_2 .

2.6.2 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 . 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/c and good consolidation serve to reduce permeability and restrict carbonation to the surface. Industrial areas with higher concentrations of CO_2 in the air result in higher rates of carbonation.

2.6.3 Carbonation by groundwater— CO_2 absorbed by rain enters the groundwater as carbonic acid. Additional CO_2 , together with humic acid, can be dissolved from decaying vegetation, resulting in high levels of free CO_2 . While such waters are usually acidic, the aggressiveness cannot be determined by pH alone. Reaction with carbonates in the soil produce an equilibrium with calcium bicarbonate that can result

in solutions with a neutral pH, but containing significant amounts of aggressive CO_2 (Lea 1971).

The rate of attack, similar to that by CO_2 in the atmosphere, is dependent upon the properties of the concrete and concentration of the aggressive CO_2 . There is no consensus at this time as to limiting values because of widely varying conditions in underground construction. It has been concluded in some studies, however, that water containing more than 20 parts per million (ppm) of aggressive CO_2 can result in rapid carbonation of the hydrated cement paste. On the other hand, freely moving waters with 10 ppm or less of aggressive CO_2 can also result in significant carbonation (Terzaghi 1948, 1949).

CHAPTER 3—ABRASION

3.1—Introduction

The abrasion resistance of concrete is defined as the “ability of a surface to resist being worn away by rubbing and friction” (ACI 116R). Abrasion of floors and pavements can result from production operations, or foot or vehicular traffic; therefore, abrasion resistance is of concern in industrial floors (Lovell 1928). Wind or waterborne 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 quite objectionable in some kinds of service. Abrasion of concrete in hydraulic structures is discussed only briefly in this guide; the subject is treated in detail in ACI 210R.

3.2—Testing concrete for resistance to abrasion

Research to develop meaningful laboratory tests on concrete abrasion has been underway for more than a century. There are several different types of abrasion, and no single test method has been found that is adequate for all conditions. Four general areas should be considered (Prior 1966):

1. Floor and slab construction—Table 2.1 of ACI 302.1R, classes of wear are designated and special considerations required for good wear resistance. (Table 2.1 of ACI 302.1R is reproduced herein as **Table 3.1**);

2. Wear on concrete road surfaces is due to heavy trucks and automobiles with studded tires or chains (attrition, scraping, and percussion);

3. Erosion of hydraulic structures, such as dams, spillways, tunnels, bridge piers, and abutments, is 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 causes erosion where high velocities and negative pressures are present. This damage can best be corrected by changes in design that are not covered in this guide.

ASTM C 779 covers three operational procedures for evaluating floor surfaces: Procedure A, revolving discs (Schuman and Tucker 1939); Procedure B, dressing wheels; and Procedure C, ball bearings.

Each method has been used to develop information on wear resistance. Prior (1966) commented that the most reliable method uses revolving discs. Reproducibility of abrasion test-

Table 3.1—Floor classifications (Table 2.1 in ACI 302.1R)

	Class	Usual traffic	Use	Special considerations	Concrete finishing technique (Chapter 7)
Single course	1	Light foot	Residential or tile covered	Grade for drainage, make plane for tile	Medium steel trowel
	2	Foot	Offices, churches, schools, hospitals	Nonslip aggregate, mix in surface	Steel trowel; special finish for nonslip
			Ornamental residential	Color shake, special	Steel trowel, color, exposed aggregate; wash if aggregate is to be exposed
	3	Light foot and pneumatic wheels	Drives, garage floors, and side-walks for residences	Crown, pitch, joints, and air entrainment	Float, trowel, and broom
	4	Foot and pneumatic wheels*	Light industrial, commercial	Careful curing	Hard steel trowel and brush for nonslip
5	Foot and wheels—abrasive wear*	Single-course industrial, integral topping	Careful curing	Special metallic or mineral aggregate, float and trowel	
Two course	6	Foot and hard wheel vehicles—severe abrasion	Bonded two-course heavy industrial	<i>Base</i> —textured surface and bond	<i>Base</i> —Surface leveled by troweling
				<i>Topping</i> —special aggregate, mineral, or both, or metallic surface treatment	<i>Topping</i> —Special power floats with repeated steel troweling
	7	Classes 3, 4, 5, and 6	Unbonded toppings	Mesh reinforcing; bonded breaker on old concrete surface; minimum thickness 2-1/2 in. (nom. 64 mm)	—

*Under abrasive conditions on floor surface, the exposure will be much more severe and a higher quality surface will be required for Class 4 and 5 floors. Under these conditions a Class 6 two-course floor or a mineral or metallic aggregate monolithic surface treatment is recommended.

ing is an important factor in selecting the test method. Replication of results is necessary to avoid misleading data from single specimens.

The concrete surface condition, aggregates used that are abraded during the test procedure, and care and selection of representative samples will affect test results. Samples that are fabricated in the laboratory must be identical for proper comparison and the selection of sites for field testing to provide representative results.

To set limits for abrasion resistance of concrete, it is necessary to rely on relative values based on test results that will provide a prediction of service.

Underwater abrasion presents special demands for test procedures. ASTM C 1138 uses agitation of steel balls in water to determine abrasion resistance.

3.3—Factors affecting abrasion resistance of concrete

The abrasion resistance of concrete is a progressive phenomenon. Initially, resistance is closely related to compressive strength at the wearing surface, and floor wear is best judged on this basis. As the paste wears, the fine and coarse aggregates are exposed, and abrasion and impact will cause additional degradation that is related to aggregate-to-paste bond strength and hardness of the aggregate.

Tests (Scripture, Benedict, and Bryant 1953; Witte and Backstrom 1951) and field experience have generally shown that compressive strength is proportional to the abrasion resistance of concrete. Because abrasion occurs at the surface, it is critical that the surface strength be maximized. Resistance can be increased by the use of shakes and toppings, finishing techniques, and curing procedures.

Reliance should not be placed solely on test cylinder compressive strength results, but careful inspection should be given to the installation and finishing of the floor surface (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 (forbidding any water addition to the surface to aid 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 must be given to the quality of the aggregate in the surface region (Scripture, Benedict, and Bryant 1953; Smith 1958). The service life of some concrete, such as warehouse floors subjected to abrasion by steel or hard rubber wheeled traffic, is greatly lengthened by the use of a specially hard or tough aggregate.

Special aggregates can be used either by the dry-shake method 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. For additional abrasion resistance, a change to a blend of metallic aggregate and cement will increase the abrasion resistance further and provide additional surface life.

The use of two-course floors using a high-strength topping is generally limited to floors where both abrasion and impact are destructive effects at the surface. While providing excellent abrasion resistance, a two-course floor will generally be more expensive and is justified only when impact is a consideration. Additional impact resistance can be obtained by using a topping containing portland cement and metallic aggregate. A key element in production of a satisfactory floor surface is curing (Prior 1966; ACI 302.1R; ACI 308).

Because the uppermost part of the surface region is that portion abraded by traffic, maximum strength is most important at the surface. This is partially accomplished through proper timing of the finishing operation, hand troweling, and adequate curing.

3.4—Recommendations for obtaining abrasion-resistant concrete surfaces

3.4.1 The following measures will result in an appropriate concrete compressive strength, giving abrasion-resistant concrete surfaces (refer to ACI 302.1R, Table 6.2.1):

- A low w/cm at the surface—Use water-reducing admixtures, a mixture proportioned to eliminate bleeding, or timing of finishing to avoid the addition of water during troweling; vacuum-dewatering may be a good option;
- Proper grading of fine and coarse aggregate (meeting ASTM C 33)—The maximum size of coarse aggregate should be chosen for optimum workability and minimum water content;
- Use the lowest slump consistent with proper placement and consolidation as recommended in ACI 309R, and proportion the mixture for the desired slump and to achieve the required strength; and
- Air contents should be consistent with exposure conditions. For indoor floors not subjected to freezing and thawing, air contents of 3% or less are preferable.

In addition to a detrimental effect on strengths, high air contents can cause blistering if finishing is improperly timed. Entrained air should not be used when using dry shakes unless special precautions are followed.

3.4.2 *Two-course floors*—High-strength toppings in excess of 6000 psi (40 MPa) will provide increased abrasion resistance using locally available aggregate. Normally, the nominal maximum aggregate size in a topping is 12.5 mm (1/2 in.).

3.4.3 *Special concrete aggregates*—Selection of aggregates for improved strength at a given w/cm will also improve abrasion resistance. These are normally applied as a dry shake or in a high-strength topping.

3.4.4 *Proper finishing procedures*—Delay floating and troweling until the concrete has lost its surface water sheen. It may be necessary to remove free water from the surface to permit proper finishing before the base concrete hardens. Do not finish concrete with standing water because this will radically reduce the compressive strength at the surface. The delay period will vary greatly depending on temperature, humidity, and the movement of air. More complete finishing recommendations are included in ACI 302.1R

3.4.5 *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. Ensure that proper dewatering is accomplished at the edges of the vacuum mats. Improperly dewatered areas are less resistant to abrasion due to a higher w/cm .

3.4.6 *Special dry shakes and toppings*—When severe wear is anticipated, the use of special dry shakes or toppings

should be considered. For selection, the recommendations found in ACI 302.1R should be followed.

3.4.7 *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 a practical method. Curing compounds, which seal moisture in the concrete, are used as an alternative.

Water curing can be used by sprays, damp burlap, or cotton mats. Water-resistant paper or plastic sheets are satisfactory, provided the concrete is first sprayed with water and then immediately covered with the sheets, with the edges overlapped and sealed with water-resistant tape.

Curing compounds should meet ASTM C 309 at the coverage rate used and should be applied in a uniform coat immediately after concrete finishing. The compound should be covered with scuff-resistant paper if the floor is subjected to traffic before curing is completed. More information is found in ACI 308.

Wet curing is recommended for concrete with a low w/cm (to supply additional water for hydration), where cooling of the surface is desired, where concrete will later be bonded, or where liquid hardeners will be applied. It should also be required for areas to receive paint or floor tile, unless the curing compound is compatible with these materials. Curing methods are described in detail in ACI 308. Unvented salamander heaters or other fossil-burning fuel heaters that increase CO_2 levels during cold-weather concreting, finishing machines, vehicles, and welding machines should not be used unless the building is well ventilated. Under certain conditions, CO_2 will adversely affect the fresh concrete surface during the period between placement and the application of a curing compound. The severity of the effect is dependent on concentration of the CO_2 in the atmosphere, humidity, temperature, and length of exposure of the concrete surface to the air (Kauer and Freeman 1955). Carbonation will destroy the abrasion resistance of the surface to varying depths depending upon the depth of carbonation. The only resource is to grind the floor and remove the offending soft surface.

3.5—Improving wear resistance of existing floors

Liquid surface treatments (hardeners) are sometimes used to improve the wear resistance of floors (Smith 1956). Magnesium and sodium silicate are most commonly used. Their principal benefit is reduced dusting. They can also slightly resist deterioration by some oils and chemicals coming in contact with the concrete. Liquid hardeners are most useful on older floors that have started to abrade or dust as a result of poor-quality concrete or poor construction practices, such as finishing while bleedwater is on the surface, inadequate curing, or both. In such cases, they serve a useful purpose in prolonging the service life of the floor. Properly cured new floors should be of such quality that treatments with liquid hardeners should not be required, except where even slight dusting cannot be tolerated, that is, in powerhouse floors.

Liquid hardeners should not be applied to new floors until they are 28 days old to allow time for calcium hydroxide to

be deposited at the surface. Magnesium and sodium silicate liquid surface treatments react chemically with hydrated lime (calcium hydroxide), which is available at the surface of uncured concrete. Fluosilicates have toxic effects on workers and the environment, and must be handled with care. This lime is generated during cement hydration and, in inadequate curing conditions, is suspended in the pore water and is deposited on the concrete surface as the water evaporates. Proper curing reduces or eliminates these surface or near-surface lime deposits (National Bureau of Standards 1939). The floor should be moist-cured for 7 days and then allowed to air-dry during the balance of the period. Curing compounds should not be used if hardeners are to be applied because they reduce the penetration of the liquid into the concrete. The hardener should be applied in accordance with the manufacturer's instructions.

3.6—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 1/4 to 1/2 in. (6 to 12 mm) 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 U.S., to examine the properties of existing concretes 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 and polymer-fly ash concretes provide better resistance to wear, although at the sacrifice of skid resistance. Steel-fibrous concrete overlays were also tested and showed reduced wear. Although these results are fairly promising, no affordable concrete surface has yet been developed that will provide a wear life, when studded tires are used, approaching that of normal surfaces under rubber tire wear.

A report (Transportation Research Board 1975) summarizes available data on pavement wear and on the performance and winter accident record while studded tires have been in use.

3.7—Skid resistance of pavements

The skid resistance of concrete pavement depends on its surface texture. Two types of texture are involved:

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

The microtexture is the more important at speeds of less than approximately 50 mph (80 km/h) (Kummer and Meyer 1967; Murphy 1975; Wilk 1978). At speeds greater than 50 mph (80 km/h) the macrotexture becomes quite important, because it is relied on to prevent hydroplaning.

The skid resistance of concrete pavement initially depends on the texture built into the surface layer (Dahir 1981). In time, rubber-tired traffic abrades the immediate surface layer, removing the beneficial macrotexture and eventually exposing the coarse aggregate particles. The rate at which this will occur and the consequences on the skid resistance of the pavement depend on the depth and quality of the surface layer and the rock types in the fine and coarse aggregate.

Fine aggregates containing significant amounts of silica in the larger particle sizes will assist in slowing down the rate of wear and maintaining the microtexture necessary for satisfactory skid resistance at the lower speeds. 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 of this type, there may be a rapid polishing of the entire pavement surface with 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 be increased when it is exposed.

The macrotexture, quite important because it prevents hydroplaning, is accomplished by constructing grooves in the concrete—either before hardening or by sawing after the concrete has sufficient strength to provide channels for the escape of water otherwise trapped between the tire and pavement. It is vital that the island between the grooves be particularly resistant to abrasion and frost action. A high-quality concrete, properly finished and cured, possesses the required durability.

CHAPTER 4—CORROSION OF METALS AND OTHER MATERIALS EMBEDDED IN CONCRETE

4.1—Introduction

The avoidance of the conditions causing corrosion of reinforcing and prestressing steel is necessary if concrete containing steel is to have the intended longevity. This chapter summarizes the mechanisms and conditions of corrosion and methods and techniques for circumventing corrosion.

Table 4.1—Chloride data

Calcium-chloride compound	CaCl ₂ , %	Cl ⁻ , %
77 to 80% (flake)	78	50
90% CaCl ₂ (anhydrous)	91	58
94 to 97% CaCl ₂ (anhydrous)	95	61
29% CaCl ₂ solution	29	19

Concrete usually provides protection against the rusting of adequately embedded steel because of the highly alkaline environment of the portland-cement paste. The adequacy of that protection is dependent upon the amount of concrete cover, the properties of the concrete, the details of the construction, and the degree of exposure to chlorides from concrete-making components and external sources.

ACI 222R details the mechanisms of corrosion, protection against corrosion in new construction, methods for identifying corrosive environments, techniques for identifying steel undergoing active corrosion, and remedial measures and their limitations, and should be consulted for further information.

4.2—Principles of corrosion

4.2.1 Corrosion of steel in concrete is usually an electrochemical process that develops an anode where oxidations takes place and a cathode where reduction takes place. At the anode, electrons are liberated and ferrous ions are formed ($\text{Fe} \rightleftharpoons \text{Fe}^{++} + 2e^{-}$); at the cathode, hydroxyl ions are liberated ($1/2\text{H}_2\text{O} + 1/4\text{O}_2 + e^{-} \rightleftharpoons \text{OH}^{-}$). The ferrous ions subsequently combine with oxygen or the hydroxyl ions and produce various forms of rust.

Steel in concrete is usually protected against corrosion by the high pH of the surrounding portland-cement paste. Uncarbonated cement paste has a minimum pH of 12.5, and steel will not corrode at that pH. If the pH is lowered (for example, pH 10 or less), corrosion can occur. Carbonation of the portland-cement paste can lower the pH to levels of 8 to 9, and corrosion can ensue. When moisture and a supply of oxygen are present, the presence of water-soluble chloride ions, above threshold levels of 0.2% (0.4% calcium chloride) by mass of portland cement, can accelerate corrosion (ACI 222R). Chloride in concrete is frequently referred to as calcium chloride (dihydrate, anhydrous, and flake and pellet forms), or chloride (Cl⁻). The basic reference to chloride, particularly with respect to corrosion, is chloride as percent by mass of portland cement. For chloride used as an admixture, the usual references are to flake calcium chloride (contains 20 to 23% water) as a 1 or 2% addition by mass of portland cement. The amount of calcium chloride in different formulations is shown in [Table 4.1](#).

Corrosion can be induced if the concentration of oxygen, water, or chloride differs at various locations along a steel bar or electrically connected steel system. Other driving forces include couplings of different metals (galvanic corrosion) and stray electrical currents, such as caused by DC current of electric railways, electroplating plants, and cathodic systems used to protect other steel systems (such as pipe).

In each of the preceding situations, a strong electrolyte (such as chloride) and moisture are needed to promote the

corrosion or at least cause it to occur rapidly (in years instead of decades). If steel in contact with the concrete is not fully encased by it (for example, decking, door jams, and posts), even trace amounts of chloride can trigger and accelerate corrosion when moisture and oxygen are present.

There has been a great deal of discussion about the significance of chloride introduced into the concrete mixture versus chloride that enters the concrete from the environment. The former has been called domestic chloride, and the latter foreign chloride. Examples of domestic chloride include a chloride component of set-accelerating admixtures, water-reducing admixtures, aggregates, or cementitious materials.

If there is uniform distribution of chlorides, corrosion can be minimal. Even if there is a uniform distribution of chlorides, however, significant corrosion can result because of differences in oxygen and moisture contents or because of other factors. Further, in the case of a domestic chloride, even if the chloride is initially uniformly distributed, a non-uniform distribution can eventually result due to movement of water that contains chloride in solution. Additionally, some of the domestic chloride can become chemically fixed by reactions with calcium aluminate components of the portland cement, forming calcium chloroaluminate hydrates (or chloride), once chemically bound, can become unbound because of carbonation.

Based upon a review of literature on the relationship of chloride concentrations and corrosion of fully embedded steel, ACI Committee 222 recommends the following maximum acid-soluble chloride-ion contents, expressed as percent by mass of the cement, as a means of minimizing the risk of corrosion—prestressed concrete is 0.08%, and reinforced concrete is 0.20%.

Committee 222 also comments that because some of the concrete-making materials can contain chlorides that are not released into the concrete, documentation on the basis of past, good performance can provide a basis for permitting higher chloride levels. The suggested levels provide a conservative approach that is necessary because of the conflicting data on chloride threshold levels and the effect of different exposure environments. The conservative approach is also recommended because many exposure conditions, such as bridge decks, garages, and concretes in a marine environment, allow the intrusion of foreign chlorides. In instances where foreign chlorides are present, concrete should be made with admixtures and other concrete-making components that contain only trace amounts of chloride or none at all.

There have been instances of corrosion in relatively dry exposures, such as inside buildings, where the concrete was made with calcium chloride additions within the 1 to 2% levels usually deemed satisfactory for concrete that will stay dry (Erlin and Hime 1976). In these circumstances, concrete drying has been very slow because of thick sections or the use of tiles and other barriers to prevent loss of water by evaporation.

4.3—Effects of concrete-making components

4.3.1 *Portland cement, ground granulated blast-furnace slag, and pozzolans*—The high pH of concrete results largely from the presence of calcium hydroxide, liberated when the

portland cement hydrates, which constitutes approximately 15 to 25% of the portland-cement paste. Because the pH of a saturated solution of calcium hydroxide is 12.5, it is the minimum pH of uncarbonated paste. A higher pH can result because of the present of sodium and potassium hydroxide.

The tricalcium-aluminate component of portland cement can react with chloride to form calcium-chloroaluminate hydrates, which chemically tie up some of the chloride. Studies on the durability of concrete in a seawater exposure showed that when cement having 5 to 8% tricalcium aluminate (C_3A) was used, there was less cracking due to steel corrosion than when cement having a C_3A content less than 5% was used (Verbeck 1968). It is principally the domestic chloride that reacts, especially during the initial week or so of cement hydration. Subsequent carbonation of the paste (usually restricted to shallow surface regions and cracks) can result in the liberation of some of that chemically bound chloride.

The chloride content of portland cement, fly ash, and silica fume is typically very low. Slag, however, can have a significant chloride content if quenched with salt water.

4.3.2 Aggregates—Aggregates can contain chloride salts, particularly those aggregates associated with seawater or whose natural sites are in groundwater containing chloride. There have been reported instances (Gaynor 1985) where quarried stone, gravels, and natural sand contained small amounts of chloride that have provided concrete with chloride levels that exceed the permissible levels previously described. For example, coarse aggregate containing 0.06% chloride, when used in amounts of 1800 lb/yd³ (815 kg/m³) of concrete and with a cement content of 576 lb/yd³ (261 kg/m³), will result in 0.2% chloride by mass of cement. That level is the upper limit recommended in ACI 222R. Not all of that chloride will necessarily become available to the paste. Thus, ACI 222R indicates that higher levels are tolerable if past performance has shown that the higher chloride content has not caused corrosion.

4.3.3 Mixing water—Potable mixing water can contain small amounts of chloride, usually at levels from 20 to 100 ppm. Such amounts are considered insignificant. For example, for a concrete mixture containing 576 lb (261 kg) of portland cement per cubic yard and a w/cm of 0.5, the resulting chloride level would only be from 0.001 to 0.005% by mass of portland cement. Reclaimed wash water, however, can contain significant amounts of chloride, depending on the chloride content of the original concrete mixture and the water used for washing.

4.3.4 Admixtures other than those composed principally of calcium chloride and contributing less than 0.1% chloride ions by mass of cement—Some water-reducing admixtures contain chloride to improve admixture performance but contribute only small amounts of chloride to the concrete when they are added at recommended rates. Normal setting admixtures that contribute less than 0.1% chloride by mass of cement are most common and their use should be evaluated based on an application basis. If chloride ions in the admixture are less than 0.01% by mass of cementitious material, such contribution represents an insignificant amount and is considered innocuous.

Accelerating admixtures, other than those based on calcium chloride, have been used in concrete with varying success. Accelerators that do not contain chloride should not be assumed to be noncorrosive. Materials most commonly used are calcium formate, sodium thiocyanate, calcium nitrate, and calcium nitrite. It is generally accepted that formates (Holm 1987) are noncorrosive in concrete.

Calcium nitrite is the only accelerating chemical recommended by an admixture manufacturer 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 to nitrite ratios exceeding 1.5 to 1.0 by weight. Dosage rates of 40 to 170 fl oz per 100 lb (26 to 110 mL/kg) of cement are the most common. An extensive review of calcium nitrite's use in concrete was compiled by Berke and Rosenberg (1989). It documents the effectiveness of calcium nitrite as a corrosion inhibitor for steel, galvanized steel, and aluminum in concrete.

Structures subjected to deicing salt applications should be designed to limit penetration of chlorides to the reinforcing steel. If the accelerating effect from calcium nitrite is undesirable, use of a retarder is recommended. An increased air-entraining agent may be necessary when calcium nitrite is used.

At high dosages, sodium thiocyanate has been reported to promote corrosion (Manns and Eichler 1982). The threshold dosage at which it will initiate corrosion is between 0.75 and 1.0% by mass of cement (Manns and Eichler 1982; Nmai and Corbo 1989).

4.4—Concrete quality and cover over steel

One cause of chloride intrusion into concrete is cracks. These cracks allow infiltration by chlorides at a much faster rate than by the slower diffusion processes and establish chloride concentration cells that can initiate corrosion. To minimize crack formation, concrete should always be made with the lowest practical w/cm commensurate with workability requirements for proper consolidation. Quality concrete will have decreased water permeability and absorption, increased resistance to chloride intrusion, and reduced risk of corrosion.

When concrete is kept moderately dry, corrosion of steel can be minimized. For example, if concrete containing as much as 2% flake calcium chloride is allowed to dry to a maximum relative humidity of 50 to 60%, embedded steel should either not corrode or corrode at an inconsequential rate (Tutti 1982).

4.4.1 Cover over steel—Extensive tests (Clear 1976; Pfeifer, Landgren, and Zoob 1987; Marusin and Pfeifer 1985) have shown that 1 in. (25 mm) cover over bare steel bars is inadequate for 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 1/2 in. (12 mm) of concrete can be very high compared with those at depths of 1 to 2 in. (25 to 50 mm), even in concrete with a w/cm of 0.30. As a result, cover for

moderate-to-severe corrosion environments should be a minimum of 1-1/2 in. (38 mm) and preferably at least 2 in. (50 mm).

4.4.2 Concrete permeability and electrical resistivity—The permeability of concrete to water and chloride is the major factor affecting the process of corrosion of embedded metals.

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 interior of concrete usually requires extensive drying to achieve low electrical conductivity. Tests sponsored by the Federal Highway Administration (Pfeifer, Landgren, and Zoob 1987) show that 7 to 9 in. (178 to 220 mm)-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. Similar AC-resistance tests on concrete made with silica fume at water-cement-plus-silica-fume ratios of 0.20 show extremely high initial electrical-resistance values when compared with concretes having w/cm of 0.30 to 0.50. The high electrical-resistance values increased the resistance to steel corrosion. The high electrical resistance of silica-fume-concrete can be due to densification of the paste microstructure.

4.4.3 Water-cement ratio and concrete cover over steel—Generally, a low w/cm will produce less permeable concrete and provide greater protection against corrosion. In severe, long-term, accelerated salt-water exposure tests of reinforced concrete slabs with 1 in. (25 mm) of cover over the steel, concretes with w/cm of 0.30, 0.40, and 0.50 each developed corrosion activity, the concrete having the 0.50 w/cm developing the most severe corrosion currents and degree of rusting of the steel. These tests show that 1 in. (25 mm) of cover is inadequate for concrete made with commonly specified w/cm when exposure is to water that contains chlorides. These same laboratory tests show that 2 and 3 in. (50 and 75 mm) of cover provide additional corrosion protection, because chloride ions could not permeate the concrete in sufficient amounts to exceed the threshold value for triggering corrosion (Marusin and Pfeifer 1985). Long-term field studies, however, have shown that concretes made with a 0.5 w/cm , with 2 to 3 in. of concrete cover will not, under certain circumstances, protect steel from corroding.

Numerous test programs have shown that concrete made with a w/cm of 0.40 and adequate cover over the steel performs significantly better than concretes made with w/cm of 0.50 and 0.60; recent tests show that concrete having a w/cm of 0.32 and adequate cover over the steel will perform even better. Chloride-ion permeability to a 1 in. (25 mm) depth is about 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.

Based upon the preceding information, the w/cm of concrete that will be exposed to sea or brackish water or be in contact with more than moderate amounts of chlorides, should be as low as possible and preferably less than 0.40. If this w/cm cannot be achieved, a maximum w/cm of 0.45 can be used provided that the thickness of cover over the steel is increased. For severe marine exposure, a minimum concrete

cover of 3 in. (75 mm) should be used. AASHTO recommends 4 in. (100 mm) of cover for cast-in-place concrete, and 3 in. (75 mm) of cover for precast piles. These recommended w/cm apply for all types of portland cement.

For trial mixture purposes, ACI 211.1 can be used to determine the cement factor required for obtaining a given w/cm .

A low w/cm does not, by itself, ensure low-permeability concrete. For example, no-fines concrete can have a low w/cm and yet be highly permeable, as evidenced by the use of such concrete to produce porous pipe. Thus, in addition to the low w/cm , the concrete must be properly proportioned and well consolidated to produce a low-permeability concrete.

Salts applied in ice-control operations will be absorbed by the concrete. To reduce the likelihood of corrosion, a minimum cover of 2 in. (50 mm) and a low w/cm (0.40 maximum) are desirable. Because of construction tolerances, a design cover of at least 2.6 in. (65 mm) is needed to obtain a minimum cover of 2 in. (50 mm) over 90 to 95% of the reinforcing steel (Van Daveer and Sheret 1975). Nondestructive techniques, such as magnetic devices (pachometer) and radar, are available to determine the depth of cover over reinforcing steel in hardened concrete (Clear 1974a; Van Daveer and Sheret 1975).

4.4.4 Mixture proportions—Low w/cm decrease concrete permeability, which results in greater resistance to chloride intrusion. In seawater exposure studies of reinforced concrete where cover over the steel was nominally 1-1/2 in. (37 mm), a w/cm of 0.45 provided good corrosion protection, a w/cm of 0.53 provided an intermediate degree of protection, and a w/cm of 0.62 provided little protection (Verbeck 1968). Tests of concrete slabs at equal cement contents, which were salted daily, indicated that w/cm of 0.40 provided significantly better corrosion protection than w/cm of 0.50 and 0.60 (Clear and Hay 1973). Based on these studies, the w/cm for concrete exposed to brackish water or seawater, or in contact with chlorides from other sources, should not exceed 0.40. Any means of decreasing the permeability of concrete, such as by the use of high-range water reducers, pozzolans, and silica fume, will prolong the onset of corrosion.

Exposure of concrete at inland sites, that is, sites so far inland that no salt comes from the sea, has not been recognized as constituting a corrosion problem except where exposed to brackish water or where deicing salts are used. Severe corrosion of bridge and parking structures has occurred.

4.4.5 Workmanship—Good workmanship is vital for securing uniform concrete and concrete of low permeability. For low-slump concrete, segregation and honeycombing can be avoided by good consolidation practices. Because low-slump concrete is often difficult to consolidate, a density-monitoring device is helpful for insuring good consolidation (Honig 1984).

4.4.6 Curing—Permeability is reduced by good curing 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, Foucrier, and Malier 1986).

4.4.7 Drainage—Attention should be given to design details to ensure that water will drain and not pond on surfaces.

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

4.5—Positive protective systems

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; however, the most successful systems are listed as follows:

- Overlays and patches of very low w/cm (0.32) using conventional low-slump concrete, latex-modified concrete overlays (Clear and Hay 1973; Federal Highway Administration 1975c), concrete containing silica fume, and concrete containing high-range water-reducing admixtures;
- Epoxy-coated reinforcing steel (Clifton, Beeghly, and Mathey 1974; Federal Highway Administration 1975a);
- Waterproof membranes (Van Til, Carr, and Vallerga 1976);
- Surface protective-barrier systems produced from select silanes, siloxanes, epoxies, polyurethanes, and methacrylates (Van Daveer and Sheret 1975);
- Cathodic protection;
- Polymer impregnation (Smock 1975); and
- Replacement of the existing concrete with concrete containing a corrosion inhibitor.

General information on repairs of concrete and use of protective-barrier systems are given in Chapters 6 and 7.

4.6—Corrosion of materials other than steel

4.6.1 Aluminum—Corrosion of aluminum embedded in concrete can occur and cause cracking in 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 has a high alkali content (Woods 1968). Increasing ratios of steel area (when the metals are coupled), particularly in the presence of appreciable amounts of chloride, increases corrosion of the aluminum. Additionally, hydrogen gas evolution can occur when fresh concrete contacts aluminum. This can increase the porosity of the concrete and the penetration of future corrosive agents. Some aluminum alloys are more susceptible to this problem than others. Corrosion inhibitors (for example, calcium nitrite) have been shown to improve the corrosion resistance of aluminum in concrete (Berke and Rosenberg 1989).

4.6.2 Lead—Lead in damp concrete can be attacked by the calcium hydroxide in the concrete and can be destroyed in a few years. Contact of the lead with reinforcing steel can accelerate the attack. It is recommended that a protective

plastic or sleeves that are unaffected by damp concrete be used on lead to be embedded in concrete. Corrosion of embedded lead is not likely to damage the concrete.

4.6.3 Copper and copper alloys—Copper is not normally corroded by concrete, as 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 ammonium and possibly of nitrates can cause stress corrosion cracking of embedded copper. It should further be noted that unfavorable circumstances are created if the concrete also contains steel connected to the copper. In this case, the steel corrodes.

4.6.4 Zinc—Zinc reacts with alkaline materials 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 (Stark and Perenchio 1975; Hill, Spellman, and Stratfull 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. Be careful 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 theory indicates that 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).

There has been some difficulty 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.

4.6.5 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, however, can be adversely affected by the presence of soluble chlorides in seawater or deicing salts. Special circumstances might justify the use of Monel, or Type 316 stainless steel in marine locations, if data have documented their superior performance in concrete containing moisture and chlorides or other electrolytes. The 300 Series stainless steels, however, are susceptible to stress corrosion cracking when the temperature is over 140 F (60 C) and chloride solutions are in contact with the material. Embedded natural-weathering steels generally do not perform well in concrete containing moisture and chloride. Weathering steels adjoining concrete can discharge rust and cause staining of concrete surfaces.

4.6.6 Plastics—Plastics are increasingly being used in concrete as pipes, shields, waterstops, chairs, and reinforcement support as well as a component in the concrete mixture. Many plastics are resistant to strong alkalis and are 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. Special epoxies are successfully used as reinforcing bar coatings and are discussed in **Chapter 7**.

4.6.7 Wood—Wood has been widely used in or against mortars and concretes. Such use includes the incorporation of sawdust, wood pulp, and wood fibers in the concrete as well as 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 cement is usually effective in minimizing these effects. The further addition of up to 5% of calcium-chloride dihydrate by weight of cement has also helped to minimize these effects. Calcium chloride in such amounts, however, can cause corrosion of embedded metals 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 can lead to cracking and warping.

The embedment of lumber in concrete has sometimes resulted in leaching of the wood by calcium hydroxide with subsequent deterioration. Softwoods, preferably with a high resin content, are reported to be most suitable for such use.

4.7—Summary comments

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

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

CHAPTER 5—CHEMICAL REACTIONS OF AGGREGATES

5.1—Types of reactions

Chemical reactions of aggregates in concrete can affect the performance of concrete structures. Some reactions are beneficial; others may result in serious damage to the concrete by causing abnormal internal expansion that can produce cracking, displacement of elements within larger structural entities, and loss of strength (Woods 1968).

5.1.1 Alkali-silica reaction—The reaction that has received greatest attention and which was the first to be recognized involves a reaction between the OH^- ion associated with the alkalis (Na_2O and K_2O) from the cement and other sources, with certain siliceous constituents that can be present in the aggregate. This phenomenon was referred to as alkali-aggregate reaction but is more properly designated as alkali-silica reaction. The earliest paper discussing alkali-silica reaction is that by Stanton (1940).

Deterioration of concrete involving certain sand-gravel aggregates has occurred in Kansas, Nebraska, and eastern Wyoming (Gibson 1938; Lerch 1959). Because early studies

showed no consistent relationship between the distress and alkali content of the cement, this deterioration was called cement-aggregate reaction to differentiate it from alkali-silica reaction. Subsequent research indicated that this phenomenon is alkali-silica reaction (Hadley 1964).

There are reports mentioning structural repairs or replacements due to malfunction or concern for safety, for instance, at the Matilija dam of the U.S. Bureau of Reclamation (Coombes, Cole, and Clark 1975), and the Jersey New Waterworks Dam (Coombes 1976). Also, the bascule piers of the Oddeesund Bridge and the Vilsund Bridges, Jutland, Denmark, were thoroughly repaired due to malfunction in service (Danish National Institute of Building Research, 1956-65). Deterioration caused the misalignment of machinery and malfunction of operational structural elements in a dam in India (Visvesvaraya, Rajkumar, and Mullick 1987). Oberholster (1981) reported repairs to rather new highway bridges and pavements and at a nuclear power plant near Cape Town, South Africa. The combined effects of thermal expansion on a dam face and expansive alkali-silica reactions in the concrete mass necessitated repairs in Fontana Dam (Abraham and Sloan 1979).

In North Germany, the Lachwehrbrücke in Lubeck was removed in 1969, approximately 1 year after construction due to severe cracking caused by alkali-silica reaction. This case received intensive public interest; although, no technical report was issued. A comprehensive report on alkali-silica reaction in Germany was published in 1973 (Verein Deutscher Zementwerke 1973); other studies are described by Lenzner (1981).

In mass-concrete gravity dams, concern about the structural integrity can occur within the stipulated lifetime. Experience shows that the warning signals, cracking, and gross expansions, often allow for timely remedial work.

In reinforced concrete, the reinforcement contributes considerable resilience against decline of the structural safety, but it cannot prevent the effects of expansion and displacement of structural members.

In some structures, concrete spalling can cause safety risks, for instance, on airport runways and on bridges over highways. In other cases, the public attitude or esthetic concern can necessitate remedial work.

Hadley (1968) described a well-documented case of damage to concrete in service where low-alkali cement was used with alkali-reactive aggregate. It dealt with pavements in a region of very hot, dry summers where there was migration and concentration of alkalis as moisture moved through the pavement to evaporate at the top surface. Concern also applies where concrete structures are exposed to additional alkalis in a marine environment or where deicing salts based on sodium chloride are applied. Additional instances of damage to concrete by expansion due to alkali-silica reaction where the cement is believed to have had an alkali content below 0.60% Na_2O equivalent have been reported (Stark 1978).

The chemical reactions are accelerated by increased temperatures. At low temperatures, the reactions can become dormant.

5.1.2 Alkali-carbonate rock reaction—It has also been clearly demonstrated that certain carbonate rocks participate in reactions with alkalis which, in some instances, produce detrimental expansion and cracking. Detrimental reactions are usually associated with argillaceous dolomitic limestones that have somewhat unusual textural characteristics (Hadley 1964). This reaction is designated as the alkali-carbonate rock reaction. It has been extensively studied in Canada, where it was originally recognized (Swenson 1957; Swenson and Gillott 1960; Feldman and Sereda 1961; Gillott and Swenson 1969; Gillott 1963b; Swenson and Gillott 1967) and in the United States (Sherwood and Newlon 1964; Newlon and Sherwood 1964; Newlon, Ozol, and Sherwood 1972; Walker 1974; Ozol and Newlon 1974).

In addition to the detrimental expansive alkali-carbonate reaction, another phenomenon associated with some carbonate rocks occurs in which the peripheral zones of the aggregate particles in contact with cement paste are modified and develop prominent rims within the particle and extensive alteration of the surrounding paste (Hadley 1964; Newlon and Sherwood 1964; Bisque and Lemish 1960a, 1960b; Lemish and Moore 1964; Hiltrop and Lemish 1960). Some rims, when etched with dilute acid, appear in positive relief while others exhibit negative relief; hence, the terms positive rims and negative rims. As contrasted with alkali-carbonate reactions that cause detrimental expansion and cracking, it is doubtful that the rim-forming alkali-carbonate reaction is, by itself, a deleterious reaction (Buck and Dolch 1976).

Some recent cases of very large structural expansion and consequent distress were reported by Grattan-Bellew (1987).

5.1.3 Other reactions involving aggregate—Other damaging chemical reactions involving aggregates include the oxidation or hydration of certain unstable oxides, sulfates, or sulfides that occur after the aggregate is incorporated into the concrete. Examples include the hydration of anhydrous magnesium oxide, calcium oxide, or calcium sulfate, or the oxidation of pyrite (Mielenz 1964). Apparently, sound dolostone aggregate that has been found to be stable in concrete at normal temperatures can deteriorate due to oxidation of small amounts of pyrite when used at elevated temperatures (Soles 1982). Metallic iron can occur as a contaminant in aggregate and subsequently be oxidized. Still other reactions can result from organic impurities such as humus and sugar (Hansen 1964). Users of aggregate should be aware of these possibilities and use corrective measures where necessary. Careful testing and examination of the aggregates will usually indicate the presence of such reactive impurities and their use in concrete can be avoided.

The alkali-silica and alkali-carbonate reactions are more important than the others and will be discussed in detail in the following section.

5.2—Alkali-silica reaction

5.2.1 Occurrence—A map (Mielenz 1978) and data are available showing areas known to have natural aggregates suspected of or known to be capable of alkali-silica reaction (Meissner 1941; Hinds and Tuthill 1941; Kammer and Carlson 1941; Dolar-Mantuani 1969; Buck and Mather 1969;

Brown 1955; Mather 1973; Duncan, Swenson, and Gillott 1973a,b; Gogte 1973; Halldorsson 1975). Most of these refer to North America; however, the available evidence (Halldorsson 1975) suggests that similar considerations are applicable elsewhere. Cases have been reported from Denmark, Iceland, Sweden, Germany, France, Britain, Italy, Cyprus, Turkey, Chile, Argentina, Brazil, India, Japan, New Zealand, Australia, East, West, and South Africa, and other countries (Halldorsson 1975; Diamond 1978; Oberholster 1981; Idorn and Rostam 1983; Grattan-Bellew 1987).

At one time, it appeared that the greatest abundance of alkali-silica reactive rocks in the U.S. was in the western half of the country. This is probably still correct for the quickly developing alkali-silica reaction which was the first to be recognized (Stanton 1940; Meissner 1941; Hinds and Tuthill 1941; Transportation Research Board 1958); however, there is also a slowly developing type (Kammer and Carlson 1941).

The aggregate constituents recognized as reactive in 1958 are shown in **Table 5.1**. (Transportation Research Board 1958). Since 1958, other rocks have been recognized as reactive, including argillites, graywackes (Dolar-Mantuani 1969), quartzites (Duncan, Swenson, and Gillott 1973a,b), schists (Gogte 1973), as well as fractured and strained quartz, recognized as reactive by Brown (1955) and granite gneiss (Mather 1973). Such strained quartz is typically characterized by undulatory extinction (Gogte 1973). Several of these rocks, including granite gneisses, metamorphosed sub-graywackes, and some quartz and quartzite gravels, appear to react slowly even with high-alkali cement, the reactivity not having been recognized until the structures were over 20 years old (Buck and Mather 1969; Brown 1955; Mather 1973). Stark and Bhatti (1986) have shown that reactive aggregates can be caused to react by alkali derived from rocks and minerals that may not themselves be alkali-silica reactive but that can yield alkali by leaching.

In the evaluation of the ages at which reactivity has been recognized in structures, one must also recognize the uncertainty of the time of recognition of reaction, the influence of ambient temperatures and humidity, the alkali-silica ratio of the reacting system, and the concentrations of reactive aggregates.

In South Africa, deleterious reactions with graywacke have not been particularly slow (Oberholster 1981).

Lightweight aggregates, which often consist predominantly of amorphous silicates, would appear to have the potential for being reactive with cement alkalis; however, no case histories of distress of lightweight concrete caused by alkali reaction have been reported so far as is known to ACI Committee 213. An unpublished account of elongation of a lightweight concrete bridge deck has not been adequately documented.

5.2.2 Mechanisms — Alkali-silica reaction can cause expansion and cracking of concrete structures and pavements. The phenomenon is complex, and various theories have been advanced to explain field and laboratory evidence (Mather 1973; Gogte 1973; Hansen 1944; Powers and Steinour 1955; Diamond 1975, 1976); yet unanswered questions remain. Silica can be dissolved in solutions of high pH.

Table 5.1—Deleteriously reactive siliceous constituents that may be present in aggregates

Reactive substance	Chemical composition	Physical character
Opal	$\text{SiO}_2 \cdot n\text{H}_2\text{O}$	Amorphous
Chalcedony	SiO_2	Microcrystalline to cryptocrystalline; commonly fibrous
Certain forms of quartz	SiO_2	(a) Microcrystalline to cryptocrystalline; (b) Crystalline, but intensely fractured, strained, and/or inclusion-filled
Cristobalite	SiO_2	Crystalline
Tridymite	SiO_2	Crystalline
Rhyolitic, dacitic, latitic, or andesitic	Siliceous, with lesser proportions of Al_2O_3	Glass or cryptocrystalline
Glass or cryptocrystalline devitrification products	FeO_2 , alkaline earths, and alkalis	Material as the matrix of volcanic rock or fragments in tuffs
Synthetic siliceous glasses	Siliceous, with lesser proportions of alkalis, alumina, other substances, or all of the above	Glass

The most important deleteriously alkali-reactive rocks (that is, rocks containing excessive amounts of one or more of the substances listed above are as follows:

Opaline cherts
Chalcedonic cherts
Quartzose cherts
Siliceous limestones
Siliceous dolomites
Rhyolites and tuffs
Dacites and tuffs

Andesites and tuffs
Siliceous shales
Phyllites
Opaline concretions
Fractures, stained, and inclusion-filled quartz and quartzites

Note: A rock may be classified as, for example, a siliceous limestone and be innocuous if its siliceous constituents are other than those indicated above.

The initial reaction product at the surface will be a nonswelling calcium-alkali silica gel approaching C-S-H. For reaction to continue, the amount of reactive material must either be a negligible or more than a pessimum amount, depending on the amount of alkali and the fineness of reactive material. Formation of the nonexpansive product is desirable and will occur if the reactive particles are sufficiently numerous or sufficiently fine. Alkali-silica reactive materials of high fineness are pozzolanic materials and blast-furnace slag properly made and used can transform the reactions to become beneficial (Pepper and Mather 1959; Idorn and Roy 1986). If the amount of alkali is large with respect to the reactive aggregate surface, interior alkali-silica gel with unlimited expansive potential will form, imbibe water, and exert potentially destructive force.

5.2.3 Laboratory tests for alkali-silica reactivity—Laboratory tests should be made on aggregates from new sources and when service records indicate that reactivity is possible. The most useful laboratory tests are:

a. *Petrographic examination (ASTM C 295)*—This document provides a standard practice for the petrographic examination of aggregates (Mielenz 1978). The types of reactive aggregate constituents involved in alkali-aggregate reaction are listed in Table 5.1, and procedures for recognizing these constituents have been described (Kammer and Carlson 1941; Mather 1948; Brown 1955; Diamond 1975, 1976). Recommendations are available that show the amounts of reactive minerals, as determined petrographically, that can be tolerated (Mather 1948; Mielenz 1958; Corps of Engineers 1985). These procedures apply principally to the more extensively studied reactive constituents.

The reactive rocks and minerals that have been more frequently encountered in recent years appear to have larger pessimum proportions and are harder to recognize in petrographic examination. Highly deformed quartz with deformation lamellae appears characteristic of the reactive quartz-bearing rocks. Relatively coarse-pained micas (Duncan, Swenson, and Gillott 1973a,b) have also been regarded as reactive constituents; fine-grained micas are reactive in argillites (Dolar-Mantuani 1969). The pessimum proportion concept does not appear to apply for reactive coarse aggregates because the reactivity may be partial. In general, the concept is difficult to apply in engineering practice because mineral composition of aggregates of mixed rock types cannot be monitored practically and economically, and also because the effects of the particle size and alkali concentration are inseparably governing parameters.

b. *Mortar-bar test for potential reactivity (ASTM C 227)*—This method is the one most generally relied on to indicate potential alkali reactivity. Acceptance criteria are given in the appendix to ASTM C 33 for evaluating results of tests made using ASTM C 227. The procedure is useful not only for the evaluation of aggregates, but also for the evaluation of specific aggregate-cement combinations. Care should be taken to ensure that the bars are never allowed to lose moisture. From the results of Duncan, Swenson, and Gillott (1973a,b), it may be expected that certain metamorphic siliceous rocks will not reliably develop an expansive reaction in storage at 100 F (38 C). More elevated temperatures, longer test periods of probably 1 to 3 years, or both, will be required to develop evidence of reactivity. This prolongation of testing time makes it particularly desirable to use petrographic

criteria that will allow identification of these rocks. Studies in recent years suggest that the mortar-bar test is not always able to ensure safe determination of the expansive reactivity of aggregates in field concrete (Oberholster and Davies 1986).

Variations on ASTM C 227 have been tried in several different laboratories in different countries with the intent of inducing meaningful results more quickly, especially for aggregates that are slowly reactive. One such procedure involves storage in sodium chloride solution (Chatterji 1978).

c. *Chemical test for potential reactivity (ASTM C 289)*—This method is used primarily for a quick evaluation with results being obtainable in a few days as compared with three to 12 months with the mortar-bar test. Care should be exercised in interpreting the results of this test. Criteria for interpretation are given in the Appendix to ASTM C 33. Transportation Research Board *Special Report* No. 31 (1958) and Chaiken and Halstead (1960) give more details concerning interpretation of the results. Some of the more recently studied reactive rocks fall into a region below the end of the curve (Fig. 2, ASTM C 289) so that the results cannot be easily interpreted using the criteria given in the standard.

The test, in effect, measures the pozzolanic reactivity of the suspected aggregate at about the maximum temperature found in most concrete during the initial curing phase. It emphasizes the essential identity of the alkali-silica reaction and the pozzolanic reaction. This test method has given questionable results when evaluating lightweight aggregates, and it is not recommended for this purpose (Ledbetter 1973).

5.2.4 Criteria for judging reactivity—The field-performance record of a particular aggregate, if it has been used with cement of high-alkali content, is the best means for judging its reactivity (Mielenz 1958). If such records are not available, the most reliable criteria are petrographic examination with corroborating evidence from the mortar-bar test (Corps of Engineers 1985), sometimes supplemented by tests on concrete. The chemical test results should also be used in conjunction with results of the petrographic examination and mortar-bar test. It is strongly recommended that reliance not be placed upon the results of only one kind of test in any evaluation (Corps of Engineers 1985).

5.2.5 Recommended procedures to be used with alkali-reactive aggregates—If aggregates are shown by service records or laboratory examination to be potentially reactive, they should not be used when the concrete is to be exposed to seawater or other environments where alkali is available to enter the concrete in solution from an external source (Transportation Research Board 1958). When reactive aggregates must be used, satisfactory service is possible. This should be done only after thorough testing, preferably after service records have established that with appropriate limits on the alkali content of the cement or with the use of appropriate amounts of an effective pozzolan or slag, or both (Pepper and Mather 1959). In cases where alkali from the environment is not involved and there are no economical nonreactive materials available, reactive materials can be used provided the following safeguards are used:

a. *Low-alkali cement*—Specify a low-alkali cement (maximum of 0.60% equivalent Na_2O). Prohibit the use of seawater

or alkali soil water as mixing water, and avoid the addition of sodium or potassium chloride. Beware of the risk of migration of alkalis by diffusion in concrete.

b. *Pozzolan or slag*—Alternatively, use a suitable pozzolanic material meeting the relevant requirements of ASTM C 618, or blast-furnace slag meeting the requirements of ASTM C 989. Pozzolans should be tested in accordance with ASTM C 441 to determine their effectiveness in preventing excessive expansion due to the alkali-aggregate reaction. The criterion of 75% reduction in mortar-bar expansion, based on an arbitrary cement-pozzolan ratio, merely provides a basis for comparison. Pepper and Mather (1959) showed that many pozzolans would need to be used at higher than typical proportions to achieve 75% reduction in an expansion of a Pyrex mixture with a cement having a 1.0% Na_2O equivalent. Pozzolans (natural, fly ash, silica fume) when tested in a similar manner must show mortar-bar expansions less than 0.020% at 14 days. Fortunately, most reactive aggregates are less reactive than Pyrex.

Whenever the use of pozzolanic materials is considered, remember that these materials increase water demand and may cause increased drying shrinkage in concrete exposed to drying. Increased water demand results from high fineness and poor particle shape. Usually, well-granulated and ground blast-furnace slag will improve the workability of concrete. The rate of strength development in correctly proportioned concrete made with a pozzolan or slag can equal or exceed that of portland cement concretes at 28 days.

5.2.6 Cement-aggregate reaction—Sand-gravel aggregates in Kansas, Nebraska, and Wyoming have been involved in concrete deterioration due to cement-aggregate reaction (Gibson 1938; Lerch 1959; Hadley 1968) and differentiated from alkali-silica reaction because of lack of clear-cut dependence on level of alkali content of cement. It is now known (Hadley 1968) that the reaction is an alkali-silica reaction. Evaporation at the surface of the concrete causes an increase in alkali concentration in the pore fluids near the drying surface. Under these and other comparable conditions, even a low-alkali cement can cause objectionable deterioration, particularly near the surface. Special tests, such as ASTM C 342, have been devised to indicate potential damage from this phenomenon. Petrographic examination (ASTM C 295) and mortar bars (ASTM C 227), with results interpreted as described by Hadley (1968), are regarded as more reliable.

The use of these potentially deleteriously reactive sand-gravel aggregates should be avoided where possible. If they must be used, however, a suitable pozzolan or blast-furnace slag that does not increase drying shrinkage and 30% or more (by mass) of nonreactive limestone coarse aggregate should be used. Concrete tests should be used to determine whether the resulting combination is satisfactory (Transportation Research Board 1958; Powers and Steinour 1955) and whether the limestone is frost resistant in air-entrained concrete in the grading in which it is used.

5.3—Alkali-carbonate reaction

5.3.1 Occurrence—Certain carbonate-rock aggregates, usually dolomitic, have been found to be reactive in concrete structures in Canada (Ontario) and in the United States (Illinois, Indiana, Iowa, Michigan, Missouri, New York, South Dakota, Virginia, Tennessee, and Wisconsin). Both quarried aggregates and gravels containing particles from the same formation can be reactive.

5.3.2 Mechanism—More than one mechanism to explain alkali-carbonate reactivity has been proposed (Hadley 1964; Gillott and Swenson 1969; Gillott 1963a; Sherwood and Newlon 1964; Newlon, Ozol, and Sherwood 1972). It is clear that when dedolomitization leading to the formation of brucite $[\text{Mg}(\text{OH})_2]$ occurs, there is a regeneration of the alkali. This is a feature that is different from alkali-silica reactivity, in which the alkali is combined in the reaction product as the reaction proceeds. The presence of clay minerals appears significant in some cases and their swelling, when opened to moisture by dedolomitization, is the basis for one of the possible explanations of the reaction (Gillott 1963a).

Rim growth is not unusual in many carbonate rocks, and it has been associated with distress in pavements in Iowa (Welp and De Young 1964); however, this is not always the case. The nature of rim formation is not fully understood (Hadley 1964). It is, however, associated with a change in the distribution of silica and carbonate between the aggregate particle and the surrounding cement paste. The rims appear to extend concentrically deeper into the aggregate with time.

The affected concrete is characterized by a network of pattern or map cracks, usually most strongly developed in areas of the structure where the concrete has a constantly renewable supply of moisture, such as close to the waterline in piers, from earth behind retaining walls, from beneath road or sidewalk slabs, or by wick action in posts or columns. A feature of the alkali-carbonate reaction that distinguishes it from the alkali-silica reaction is the general absence of silica-gel exuding from cracks. Additional signs of the severity of the reaction are closed expansion joints with possible crushing of the adjacent concrete (Hadley 1964; Swenson and Gillott 1964).

5.3.3 Identification by laboratory tests

a. *Petrographic examination of aggregate (ASTM C 295)*—Such examination is used to identify the features of the rock as listed by Hadley (1964), and modified by Buck and Mather (1969) and Dolar-Mantuani (1964, 1971). While it is generally true that reactive rocks can be characterized as having dolomite rhombs from 1 to 200 μm in maximum dimension in a background of finer calcite and insoluble residue, the presence of all or any dolomite in a fine-grained carbonate rock makes it desirable to conduct the rock-cylinder test (ASTM C 586). This is recommended whether or not the texture is believed to be typical and whether or not insoluble residue, including clay, amounts to a substantial portion of the aggregate. As expansive rocks are recognized from more areas, the more variable the textures and composition appear to be.

b. *Rock-cylinder test (ASTM C 586)*—The rock-cylinder test was first adopted by ASTM in 1966 based on work by

Hadley (1964). It is discussed by Walker (1978). It should be used as a screening test.

c. *Expansion of concrete prisms*—The prisms are made with job materials and stored at 100% relative humidity at 73 F (23 C) (Swenson and Gillott 1964), or to accelerate the reaction, they may be made with additional alkali or stored at elevated temperature or both (Smith 1964, 1974; Gillott 1963a; Rogers 1986). Swenson and Gillott (1964) reported that such tests showed that expansion of concrete with highly reactive carbonate rock could be reduced to safe values only if the alkali content of the cement is below 0.45 or 0.40% as Na_2O equivalent. They state, “the normally accepted maximum of 0.60% alkali in low-alkali cement is not adequate.”

Comparison is usually made with the expansion of prisms containing a nonreactive control aggregate. ASTM C 1105 to measure length change of concrete due to alkali-carbonate rock reaction was adopted in 1989, and a Canadian standard (CSA A23.2-14A) using concrete specimens is available.

d. *Petrographic examination of the concrete*—This can confirm the types of aggregate constituents present and their characteristics. Distress that has occurred in the aggregate and surrounding matrix, such as micro- and macrocracking, can be observed. Reaction rims can be observed in certain aggregate particles and are identified as negative or positive by acid etching. Their presence does not necessarily signify harmful results. Secondary deposits of calcium carbonate, calcium hydroxide, and ettringite can be found in voids within the concrete. Deposits of silica, hardened or in gel form, associated with the suspect aggregate particles will not usually be found (Hadley 1964).

e. *Other laboratory tests*—An alkali-carbonate reaction can be identified by visual observation of sawed or ground surfaces. X-ray examination of reaction products is also sometimes useful. ASTM C 227, C 289, and C 342, which are applicable to alkali-silica reaction, are not applicable to alkali-carbonate reactivity.

5.3.4 Criteria for judging reactivity—Definitive correlations between expansions occurring in the laboratory in rock cylinders or concrete prisms and deleterious field performance have not yet been established. The factors involved are complex and include the heterogeneity of the rock, coarse aggregate size, permeability of the concrete, and seasonal changes in environmental conditions in service. The principal environmental conditions include availability of moisture, level of temperature, and possibly the use of sodium chloride as a deicing chemical.

Cracking is usually observed in concrete prisms at an expansion of about 0.05%. Experience in Ontario (Rogers 1986) indicates that if concrete prisms made according to the Canadian Standards Association Test Method (CSA A23.2-14A) do not show expansion greater than 0.02% after 1 year, harmful reactivity is unlikely. Slightly less restrictive criteria has been suggested elsewhere (Swenson and Gillott 1964; Smith 1974).

It is not certain that rapid determination of potential reactivity can always be made by using the rock-cylinder test, because some rocks showing an initial contraction develop considerable expansion later (Dolar-Mantuani 1964; Missouri

Highway Department 1967a). No universal correlation exists between the expansion of rock cylinders and concrete in service, but possibly between the expansion of rock cylinders and the expansion of concrete prisms stored in the laboratory (Hadley 1964; Newlon and Sherwood 1964; Missouri Highway Department 1967a; Rogers 1986).

Expansions greater than 0.10% in the rock cylinders are usually taken as a warning that further tests should be undertaken to determine expansion of the aggregate in concrete. Fortunately, many carbonate rocks that expand in rock cylinders do not expand in concrete.

5.3.5 Recommended procedures to minimize alkali-carbonate reactivity—Procedures that can be used to minimize the effects of the reaction include:

- Avoiding reactive rocks by selective quarrying (Bisque and Lemish 1960a; Smith 1964; Gillott 1963a);
- Dilution with nonreactive aggregates or use of a smaller nominal maximum size (Newlon and Sherwood 1964; Swenson and Gillott 1964); and
- Use of very low-alkali cement (less than 0.6% Na₂O equivalent [Section 5.3.3(c)])—this will prevent harmful expansions in most cases (Swenson and Gillott 1964; Missouri Highway Department 1967b); however, in pavements where sodium chloride is used as a deicing chemical, this cannot be taken as certain (Smith 1964; Missouri Highway Department 1967b).

Avoiding reactive rocks by selective quarrying is the safest and usually the most economical procedure to minimize alkali-carbonate reactivity. Pozzolans serve only as a diluent and are not effective in mitigating alkali-carbonate reactions.

5.4—Preservation of concrete containing reactive aggregate

There are no known methods of adequately preserving existing concrete that contains the elements that contribute to the potentially deleterious chemical reactions. Water or moisture is partly involved in at least two of these reactions. The destructive effects of freezing and thawing are more pronounced after the initial stages of destruction by these chemical reactions. Therefore, any practicable means of decreasing the exposure of such concrete to water may extend its useful life. It has been reported that in Iceland, treatment of vertical concrete surfaces with monosilanes is beneficial.

5.5—Recommendations for future studies

Current criteria used in the United States that provide a basis for separating aggregates into reactive and nonreactive, while generally effective in preventing recurrences of catastrophic destruction of concrete structures, are now seen to be inefficient in two ways. First, they have often caused more severe precautions to have been taken than were justified. One example is limiting the calculated cement alkalis to 0.60% Na₂O equivalent when a higher maximum would surely have been safe in some cases. Second, they have sometimes permitted alkali-silica reaction to occur to a degree causing notable cracking when aggregates erroneously classed as nonreactive were used with cements containing more than

0.60% Na₂O equivalent. It is concluded that new research (Mather 1975), or a reinterpretation of the results of previous research, is needed to better characterize the following relevant parameters:

- Degree and rate of aggregate reactivity;
- Influence of concrete mixture proportions, especially unit cement content;
- Influence of environment on the concrete, especially temperature and humidity; and
- Influence of dimensions of structures, the structural features, and the stress transfer system on cracking developed by alkali-silica reactions.

Additional future research should address optimization of the use of pozzolans and slag and methods of decreasing exposure to water of concrete made with reactive aggregate.

CHAPTER 6—REPAIR OF CONCRETE

Detailed coverage of concrete repairs falls within the mission of ACI Committee 546; therefore, this chapter will give only a brief, general coverage of the subject, with emphasis on the durability aspect. See also ACI 224.1R.

6.1—Evaluation of damage and selection of repair method

To evaluate objectively the damage to a structure, it is necessary to determine what caused the damage. The damage can be the result of poor design, faulty workmanship, mechanical abrasive action, cavitation or erosion from hydraulic action, leaching, chemical attack, chemical reaction inherent in the concrete mixture, exposure to deicing agents, corrosion of embedded metal, or another lengthy exposure to an unfavorable environment. Guidance for examining and sampling hardened concrete in construction is found in ASTM C 823.

Whatever the cause, it is essential to establish the extent of the damage, and determine if the major portion of the structure is of suitable quality on which to build a sound repair. Based on this information, the type and extent of the repair are chosen. This is the most difficult step—one that requires a knowledge of the subject and judgment by the engineer. If damage is the result of moderate exposure of what was an inferior concrete in the first place, then replacement by quality concrete should ensure lasting results. On the other hand, if quality concrete was destroyed, the problem becomes more complex. In that case, a superior quality of concrete is required, or the exposure conditions must be altered.

The repair of spalls from reinforcing bar corrosion (Section 4) requires a detailed study. Simply replacing the deteriorated concrete and restoring the original cover over the steel will not solve the problem. Also, if the structure is contaminated with salt, the electrolytic conditions will be changed by the application of new concrete, and the consequences of these changed conditions must be considered before any repairs are undertaken.

6.2—Types of repairs

6.2.1 Concrete replacement—The concrete replacement method consists of replacing defective concrete with concrete

of suitable proportions and consistency, so that it will become integral with the base concrete.

Concrete replacement is the desired method if there is honeycomb or deterioration in the existing construction that goes entirely through the wall or beyond the reinforcement, or if the quantity is large. For new work, the repairs should be made immediately after stripping the forms (Tuthill 1960; USBR 1975). Considerable concrete removal is required for this type of repair. Excavation of affected areas should continue until sound concrete has been reached. Additional chipping is sometimes necessary to accommodate the repair method selected and shape the cavity properly.

Concrete for the repair should generally be similar to the old concrete in nominal maximum size of aggregate and w/cm , provided durability is not sacrificed. Color is important in some exposed concrete.

Forming will usually be required for large repairs in vertical surfaces.

6.2.2 Dry pack—The dry pack method consists of ramming a very stiff mixture into place in thin layers. It is suitable for filling form tie-rod holes and narrow slots and for repairing any cavity that has a relatively high ratio of depth of area. Practically no shrinkage will occur with very stiff mixtures, and they develop strength equalling or exceeding that of the parent concrete. The method does not require any special equipment, but cement finishers must be trained in this type of repair if the results are to be satisfactory (USBR 1975).

6.2.3 Preplaced-aggregate concrete—Preplaced-aggregate concrete is advantageously used for certain types of repairs. It bonds well to concrete and has low drying shrinkage. It is also well adapted to underwater repairs. This is a specialized process that is described in ACI 304R.

6.2.4 Shotcrete—Properly applied shotcrete has excellent bond with new or old concrete, and is frequently the most satisfactory and economical method of making shallow repairs. It is particularly adapted to vertical or overhead surfaces where it is capable of supporting itself without a form, and without sagging or sloughing. Shotcrete repairs generally perform satisfactorily where recommended procedures of ACI 506R are followed. Simplified equipment has been developed for use in small repairs (USBR 1975).

6.2.5 Repair of scaled areas and spalls in slabs—Scaling of concrete pavement surfaces subject to deicing salts is not unusual, particularly if the concrete is not adequately air-entrained. Such areas can be satisfactorily repaired by a thin concrete overlay, provided the surface of the old concrete is sound, durable, and clean (Felt 1960). A minimum overlay thickness of approximately 1-1/2 in. (38 mm) is needed for good performance (ACI 325.9R). The temperature of the underlying slab should be as close as possible to that of the new concrete.

Spalls can occur adjacent to pavement joints or cracks. Spalls usually are several inches in depth, and even deeper excavation can be required to remove all concrete that has undergone some slight degree of deterioration. They are repaired by methods similar to those used for scaled areas.

Numerous quick-setting patching materials, some of which are proprietary, are available. Information on the field

performance of these materials is given by the Federal Highway Administration (1975c).

6.3—Preparations for repair

Sawcuts around the perimeter of a repair are usually advisable, particularly in the case of slabs, to eliminate feather edges. It is not possible to have acceptable results when the concrete is brought to a feather edge. Better results are obtained if a 3/4 in. (20 mm) minimum thickness is maintained. If practicable, the sawcuts should be made at a slight angle so that the width at the base of the patch is greater than at the surface, thereby providing some keying action.

All deteriorated or defective concrete should be removed; in the case of slabs, suitable mechanical or hydraulic scarification equipment should be used. Next, the surfaces of the concrete should be thoroughly cleaned, preferably by wet sandblasting.

Where chlorides are a factor in the deterioration, refer to [Chapter 4](#).

The bonding surface should be wet down and be damp at the time of patching. The surface should be coated with a layer of mortar approximately 1/8 in. (3 mm) thick or with another suitable bonding agent (Section 6.4). If an epoxy bonding agent is used, the repair surface should be dried before the epoxy is applied. The repair should proceed immediately after application of the bonding agent, unless directed otherwise by the manufacturer of a proprietary material.

6.4—Bonding agents

Bonding layers are generally used to establish unity between fresh concrete or mortar and the parent concrete. Sand-cement mortar or neat-cement paste has most commonly been used in the past. Many reports in the literature testify to the success of these treatments where recommended practices have been followed. Bonding agents are also used for additional insurance.

Epoxy resins are sometimes used as bonding agents (ACI 503.2). These materials develop a bond having greater tensile and shear strength than concrete. They are resistant to most chemicals and some formulations are highly water-resistant. These are some disadvantages in using epoxy resin, such as toxicity and short pot life. A number of failures of epoxy coatings have been reported. They have been ascribed to differences in thermal and tensile properties and moduli of elasticity of the two materials. These studies are continuing. For most effective results, epoxy bonding agents should not be applied in layers thicker than 3/16 in. (5 mm). Birdbaths and puddles should be avoided (ACI 503.2). Types and grades of epoxies for various uses are given in ASTM C 881.

Other types of bonding agents are available. Certain latexes, supplied as emulsions or dispersions, improve the bond and have good crack resistance. Polyvinyl acetates, styrene butadienes, and acrylics are among those used. Polyvinyl acetates should not be used, however, except in dry service conditions (ASTM 1059). Latexes can be used either as a bonding layer or added to the concrete or mortar during mixing. The substrate should be wet down with water before placing latex-modified concrete.

6.5—Appearance

Unless attention is given to all of the factors influencing the appearance of concrete repairs, they are likely to be unsightly. For concrete where appearance is important, ensure that the texture and color of the repair will match the surrounding concrete. A blend of white cement with the job cement or the careful use of pigments will enable the patch to come close to matching the color of the original concrete. A patch on a formed concrete surface should never be finished with a steel trowel because this results in a dark color that is impossible to remove.

6.6—Curing

All conventional concrete or mortar for repairs must be moist cured according to the recommendations of ACI 308. Latexes can require special curing. Epoxy resins require no moist curing.

6.7—Treatment of cracks

The decision of whether a crack should be repaired to restore structural integrity or merely sealed is dependent on the nature of the structure, the cause of the crack, and its location and extent. If the stresses that caused the crack have been relieved by its occurrence, the structural integrity can be restored with some expectation of permanence. In the case of working cracks (such as cracks caused by foundation movements, or cracks which open and close from temperature changes), however, the only satisfactory solution is to seal them with a flexible or extensible material.

Thorough cleaning of the crack is essential before any treatment takes place. All loose concrete, old joint sealant, and other foreign material should be removed. The method of cleaning is dependent upon the size of the crack and the nature of the contaminants and can include any combination of the following: compressed air, wire brushing, sandblasting, routing, or the use of picks or similar tools.

Restoration of structural integrity across a crack has been successfully accomplished using pressure injection of low-viscosity epoxies (Chung 1975; Stratton and McCollum 1974) and other monomers (Kukacka et al. 1974) that polymerize in place and rebond the parent concrete.

Sealing of cracks without restoration of structural integrity requires the use of materials and techniques similar to those used in sealing joints. A detailed discussion of the types of joint sealants available and methods of installation is contained in ACI 504R. Because cracks are generally narrower than joints, some modification in procedure, such as widening the crack with a mechanical router or the use of a low-viscosity material, is often necessary.

CHAPTER 7—USE OF PROTECTIVE-BARRIER SYSTEMS TO ENHANCE CONCRETE DURABILITY

7.1—Characteristics of a protective barrier system

Protective-barrier systems are used to protect concrete from degradation by chemicals and subsequent loss of structural integrity, to prevent staining of concrete, or to protect liquids from being contaminated by the concrete.

A protective-barrier system consists of the barrier material, the concrete surface it is to protect, the concrete structure, and the foundation. The quality of the concrete, especially at and

near the surface, will influence performance of the system because it affects the ability of the barrier material to perform as expected. The important elements of a protective-barrier system are discussed in Section 7.2. An understanding of these elements is essential to obtain optimum performance from protective-barrier systems.

7.2—Elements of a protective-barrier system

7.2.1 Barrier material—To be effective in protecting concrete, a barrier material should have certain basic properties as follows:

- When the barrier material is exposed to chemicals from the environment, the chemicals should not cause swelling, dissolution, cracking, or embrittlement of the barrier material; also, the chemicals should not permeate or diffuse through the barrier to destroy the adhesion between it and concrete;
- The abrasion resistance should be adequate to prevent the barrier material from being removed during normal service; and
- The adhesive bond strength of a nonbituminous barrier to the concrete should be at least equal to the tensile strength of the concrete at the surface; this bond is affected by the cleanliness of the interface when the barrier material is being applied.

7.2.2 Concrete-barrier interface—Most nonbituminous barrier materials specifically formulated for use over concrete develop and maintain an adhesive bond strength greater than the tensile strength of the concrete, provided that the surface is properly prepared. The surface should be free of loose particles, dirt, dust, oil, waxes, and other chemicals that prevent adhesion. Moisture within the concrete can affect the ability of a barrier system to adhere to the surface if water vapor diffusing out of the concrete condenses at the concrete-barrier interface before the barrier has had an opportunity to cure. This problem is discussed in detail in [Section 7.4](#).

7.2.3 Concrete to a depth of 1/4 in. (6 mm) — Perhaps the most critical part of a nonbituminous barrier system is the first 1/4 in. (6 mm) of concrete. When a failure occurs, a thin layer of concrete up to 1/4 in. (6 mm), but usually less than 1/8 in. (3 mm) thick, generally adheres to the underside of the barrier material. This means that the concrete failed because the internal stresses in the barrier material were greater than the tensile strength of the concrete near the interface. These stresses derive from two ways. First, shrinkage and polymerization develop stresses when the barrier material is cured. This is common to all two-component polymeric materials cured by a chemical reaction between the resin and curing agent. Second, differential volume changes in the concrete and the barrier due to a difference in linear coefficient of thermal expansion coupled with a change in temperature creates internal stresses. All polymeric barriers have a much higher linear coefficient of thermal expansion than concrete. A granular filler is usually added to the barrier material so that its thermal coefficient will be closer to that of concrete.

A barrier system should have a low modulus of elasticity to prevent stresses from being greater than the tensile strength of concrete over the range of temperature expected for its use.

Table 7.1—Protective barrier systems—General categories (from ACI 515.1R)

Severity of chemical environment	Total nominal thickness range	Typical protective barrier systems	Typical but not exclusive uses of protective systems in order of severity
Mild	Under 40 mil (1 mm)	Polyvinyl butyral, polyurethane, epoxy, acrylic, styrene-acrylic copolymer asphalt, coal tar, chlorinated rubber, vinyl, neoprene, coal-tar epoxy, coal-tar urethane	<ul style="list-style-type: none"> • Protection against deicing salts • Improve freezing-thawing resistance • Prevent staining of concrete • Use for high-purity water service • Protect concrete in contact with chemical solutions having a pH as low as 4, depending on the chemical
Intermediate	125 to 375 mil (3 to 9 mm)	Sand-filled epoxy, sand-filled polyester, sand-filled polyurethane, bituminous materials	<ul style="list-style-type: none"> • Protect concrete from abrasion and intermittent exposure to dilute acids in chemical, dairy, and food-processing plants
Severe	20 to 250 mil (1/2 to 6 mm)	Glass-reinforced epoxy, glass-reinforced polyester, procured neoprene sheet, plasticized polyvinyl chloride sheet	<ul style="list-style-type: none"> • Protect concrete tanks and floors during continuous exposure to dilute material (pH is below 3), organic acids, salt solutions, strong alkalies
Severe	20 to 280 mil (1/2 to 7 mm) Over 250 mil (6 mm)	Composite systems: (a) Sand-filled epoxy system topcoated with a pigmented but unfilled epoxy (b) Asphalt membrane covered with acid-proof brick using chemical-resistant mortar	<ul style="list-style-type: none"> • Protect concrete tanks during continuous or intermittent immersion, exposure to water, dilute acids, strong alkalies, and salt solutions • Protect concrete from concentrated acids or combinations of acids and solvents.

Weak surface concrete can result from use of too high a w/cm , overworking during finishing, the presence of laitance on the surface, or improper curing. As a result, the concrete can fail due to the stresses imposed on it even by a low-modulus barrier system. Removal of weak surface material is essential for satisfactory performance of these barrier systems. Procedures to accomplish this are given in ACI 515.1R.

7.2.4 Concrete structures—Any cracks in the concrete that occur before and after application of the barrier will reflect through the barrier if the concrete is subjected to movement from temperature changes or from load application. This concrete movement can destroy the ability of the barrier to provide protection for the concrete. A poor-quality concrete slab with high permeability can allow ground water to travel through the concrete so rapidly that the surface will never dry sufficiently to allow the barrier to develop good adhesion, or the water can push the barrier material away from the concrete.

7.2.5 Foundation conditions—A dimensionally unstable base or one that does not have sufficient supporting strength can cause cracks in the concrete that are detrimental to these barriers, as discussed previously. Also, the availability and amount of ground water is a major factor in the success of a barrier. The use of a barrier to water on the exterior surfaces of tanks and tunnels, for example, will retard the entry of water into the concrete and is required when an interior protective-barrier system is to be applied.

7.3—Guide for selection of protective-barrier systems

7.3.1 Categories of service—Selection of a barrier system to provide optimum performance at the lowest cost (on cost-per-year basis) is complicated because there are so many systems available. To help in the selection process, protective-barrier systems are divided into three general categories according to the severity of the chemical service conditions: mild, intermediate, and severe; see Table 7.1.

7.3.2 Factors affecting selection—Selection of a barrier to protect concrete for a specific chemical service requires an

awareness of several items. First, the barrier material should be resistant to deterioration or degradation by the chemicals to which it will be exposed at the operating temperature. Second, the barrier material should resist the diffusion or permeation of the chemical through it; adhesion of the barrier material to the concrete surface can be adversely affected by this phenomenon, especially when the diffusing material is acidic. Chemical resistance and permeation resistance are two separate properties. A chemical such as hydrochloric acid can permeate through various plastic and rubber barrier materials to cause loss of adhesion without any indication that the chemical has degraded the barrier materials. Third, the temperature of the chemicals contacting the barrier material affect the performance of the barrier. Each material has its own characteristic maximum operating temperature for a given chemical environment. Thermal shock caused by rapid changes in temperature can crack some barrier materials or result in loss of bond between the barrier and the concrete.

7.3.3 Selection and testing of barrier material—There is no guarantee that materials made by different manufacturers will perform similarly, even when classified as the same generic type. They vary in the types and amounts of ingredients, so their performance will also vary. In addition, the application characteristics, such as ease of applying the material to concrete, sensitivity to moisture on a concrete surface, or a very limited temperature application range, will affect performance.

The thickness of the barrier required will depend on the severity of the environment. Barrier selection should be based on testing or past experience. If tests are to be conducted, the entire barrier system should be applied to concrete specimens before exposing them to the actual environment or one that simulates as closely as possible this environment. If a selection must be made before tests of sufficient duration (as agreed between manufacturer and user) can be conducted, the barrier supplier should be asked to supply fully documented case histories where his or her system has protected

concrete under the same or similar environmental conditions. The selection of a reliable barrier manufacturer and applicator is as important as the selection of the barrier itself. ACI 515.1R provides background information on this subject.

7.4—Moisture in concrete and effect on barrier adhesion

Concrete should be dry before the barrier material is applied. Not only is surface moisture objectionable, but moisture within the concrete can also affect the ability of a coating to adhere to the surface. There are no precise guidelines to indicate when moisture will be a problem although a qualitative test is described in Section 7.4.1.

Poor barrier adhesion to the concrete can result if water vapor diffuses out to the concrete surface. A surface that is too damp can produce voids in the barrier material and lead to blistering or peeling after it has cured to a hard film. The following factors should be considered in determining whether or not this will be a problem:

- The rate of vapor transmission through and from the concrete;
- The amount of moisture remaining in the concrete at any given stage;
- The ability of the coating to breathe, and therefore, allow moisture to pass through itself;
- The temperature differential between the concrete surface and ambient air temperature while the coating is curing; if the concrete temperature is below that of the dewpoint of the surrounding air, moisture will condense on the surface; and
- The ability of the material to displace moisture from the surface.

7.4.1 Dryness of surface-test method — For some barrier systems, a qualitative moisture test for normalweight concrete, as recommended by ACI Committee 503, is helpful. Moisture content is considered excessive if moisture collects at the bond line between the concrete and the barrier material before the barrier has cured. This is evaluated by taping a 4 x 4 ft (1.2 x 1.2 m) clear polyethylene sheet to the concrete surface and determining the time required for moisture to collect on the underside of the sheet. The time for moisture to collect should be compared with the time required for the barrier material to cure—a value that should be supplied by the material manufacturer. If it cures in a time that is less than that required for moisture to collect, it can be concluded that the concrete is adequately dry. Also the ambient conditions, that is sunlight, temperature, and humidity, during the test should simulate, as much as practicable, the conditions existing during application and curing of the barrier.

7.5—Influence of ambient conditions on adhesion

For concrete surfaces exposed to the sun, there is better adhesion between the concrete surface and the barrier material when it is applied in the afternoon. Exposure to sun and air for at least 6 h results in a lower surface moisture condition. A secondary benefit of applying a barrier material in the afternoon is that the surface will normally have reached its maximum temperature so that no further expansion of air in

the concrete pores and outgassing will occur. This will eliminate the tendency for expanding air to cause blistering of the applied barrier material while it is curing.

7.6—Encapsulation of concrete

Encapsulation of concrete is a special problem. This can occur when a concrete slab-on-grade receives a vapor barrier on the underside and is subsequently covered on top with a barrier system. Water can be trapped in the concrete, making it more susceptible to damage by freezing and thawing. In addition, if the concrete is encapsulated during a relatively cool day and then is subjected to higher ambient temperatures, the increased vapor pressure of the trapped water could cause loss of adhesion of the barrier material. The use of a breathing barrier can minimize the problem.

CHAPTER 8—REFERENCES

8.1—Referenced standards and reports

The documents of the various standards-producing organizations referred to in this document are listed below with their serial designations.

American Concrete Institute

- | | |
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| 116R | Cement and Concrete Terminology |
| 201.1R | Guide for Making a Condition Survey of Concrete in Service |
| 207.1R | Mass Concrete |
| 207.2R | Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete |
| 210R | Erosion of Concrete in Hydraulic Structures |
| 211.1 | Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete |
| 212.3R | Chemical Admixtures for Concrete |
| 216R | Guide for Determining the Fire Endurance of Concrete Elements |
| 221R | Guide for Use of Normal Weight Aggregates in Concrete |
| 222R | Corrosion of Metals in Concrete |
| 224R | Control of Cracking in Concrete Structures |
| 224.1R | Causes, Evaluation, and Repair of Cracks in Concrete Structures |
| 232.2R | Use of Fly Ash in Concrete |
| 233R | Ground Granulated Blast-Furnace Slag as a Cementitious Component in Concrete |
| 234R | Guide for the Use of Silica Fume in Concrete |
| 302.1R | Guide for Concrete Floor and Slab Construction |
| 304R | Guide for Measuring, Mixing, Transporting, and Placing Concrete |
| 305R | Hot Weather Concreting |
| 306R | Cold Weather Concreting |
| 306.1 | Standard Specifications for Cold Weather Concreting |
| 308.1 | Standard Practice for Curing Concrete |
| 309R | Guide for Consolidation of Concrete |
| 311.1R | ACI Manual of Concrete Inspection |
| 318/318R | Building Code Requirements for Structural Concrete, and Commentary |
| 325.9R | Recommendations for Construction of Concrete Pavements and Concrete Bases |

330R	Guide for the Design and Construction of Concrete Parking Lots	C 457	Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete
332R	Guide to Residential Cast-in-Place Concrete Construction	C 494	Specification for Chemical Admixtures for Concrete
345R	Standard Practice for Concrete Highway Bridge Deck Construction	C 586	Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock-Cylinder Method)
345.1R	Routine Maintenance of Concrete Bridges	C 595	Specification for Blended Hydraulic Cements
357.1R	State-of-the-Art Report on Offshore Concrete Structures for the Arctic	C 618	Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
503R	Use of Epoxy Compounds with Concrete	C 666	Test Method for Resistance of Concrete to Rapid Freezing and Thawing
503.2	Standard Specification for Bonding Plastic Concrete to Hardened Concrete with a Multi-Component Epoxy Adhesive	C 671	Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing
504R	Guide to Sealing Joints in Concrete Structures	C 672	Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
506R	Guide to Shotcrete	C 682	Practice for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures
515.1R	Guide to the Use of Waterproofing, Damp-proofing, Protective, and Decorative Barrier Systems for Concrete	C 779	Test Method for Abrasion Resistance of Horizontal Concrete Surfaces
546.1R	Guide for Repair of Concrete Bridge Superstructures	C 823	Practice for Examination and Sampling of Hardened Concrete in Constructions
<i>ASTM</i>		C 881	Specification for Epoxy-Resin-Base Bonding Systems for Concrete
C 33	Specification for Concrete Aggregates	C 989	Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars
C 88	Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate	C 1012	Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution
C 94	Specification for Ready-Mixed Concrete	C 1017	Specification for Chemical Admixtures for Use in Producing Flowing Concrete
C 114	Test Methods for Chemical Analysis of Hydraulic Cement	C 1059	Specification for Latex Agents for Bonding Fresh to Hardened Concrete
C 138	Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete	C 1105	Test Method for Length Change of Concrete Due to Alkali Carbonate Rock Reaction
C 150	Specification for Portland Cement	C 1138	Test Method for Abrasion of Concrete (Underwater Method)
C 173	Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method	C 1157	Performance Specification for Blended Hydraulic Cement
C 227	Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)	C 1240	Specification for Silica Fume for Use in Hydraulic-Cement Concrete and Mortar
C 231	Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method		
C 260	Specification for Air-Entraining Admixtures for Concrete		
C 289	Test Method for Potential Reactivity of Aggregates (Chemical Method)		
C 295	Guide for Petrographic Examination of Aggregates for Concrete		
C 309	Specification for Liquid Membrane-Forming Compounds for Curing Concrete		
C 330	Specification for Lightweight Aggregates for Structural Concrete		
C 342	Test Method for Potential Volume Change of Cement-Aggregate Combinations		
C 441	Test Method for Effectiveness of Mineral Admixtures or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction		
C 452	Test Method for Potential Expansion of Portland Cement Mortars Exposed to Sulfate		
			<i>Canadian Standards Association</i>
			CSA A23.2-14A Test for Alkali-Aggregate Reaction
			These publications may be obtained from the following organizations:
			American Concrete Institute
			P.O. Box 9094
			Farmington Hills, Mich. 48333-9094
			ASTM
			100 Barr Harbor Drive
			West Conshohocken, Pa. 19428

Canadian Standards Association
178 Rexdale Blvd.
Etobicoke (Toronto), Ont.
M9W 1R3
Canada

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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:

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

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 h, 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 solution is cloudy, refilter through Whatman 42 or Millipore filter. Transfer without rinses to a 250 mL Nalgene bottle.