CONCRETE *Information*

Types and Causes of Concrete Deterioration

The exceptional durability of portland cement concrete is a major reason why it is the world's most widely used construction material. But material limitations, design and construction practices, and severe exposure conditions can cause concrete to deteriorate, which may result in aesthetic, functional, or structural problems.

Concrete can deteriorate for a variety of reasons, and concrete damage is often the result of a combination of factors. The following summary discusses potential causes of concrete deterioration and the factors that influence them.

CORROSION OF EMBEDDED METALS

Corrosion of reinforcing steel and other embedded metals is the leading cause of deterioration in concrete. When steel corrodes, the resulting rust occupies a greater volume than the steel. This expansion creates tensile stresses in the concrete, which can eventually cause cracking, delamination, and spalling (Figs. 1 and 2).

Steel corrodes because it is not a naturally occurring material. Rather, iron ore is smelted and refined to produce steel. The production steps that transform iron ore into steel add energy to the metal.

Steel, like most metals except gold and platinum, is thermodynamically unstable under normal atmospheric conditions and will release energy and revert back to its natural state—iron oxide, or rust. This process is called corrosion.

For corrosion to occur, four elements must be present: There must be at least two metals (or two locations on a single metal) at different energy levels, an electrolyte, and a metallic connection. In

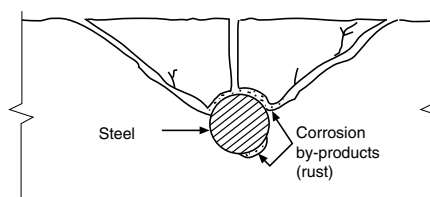Fig. 1. Corrosion of reinforcing steel is the most common cause of concrete deterioration. (46080)

Fig. 2. The expansion of corroding steel creates tensile stresses in the concrete, which can cause cracking, delamination, and spalling.

Corrosion is an electrochemical process involving the flow of charges (electrons and ions). Fig. 3 shows a corroding steel bar embedded in concrete. At active sites on the bar, called anodes, iron atoms lose electrons and move into the surrounding concrete as ferrous ions. This process is called a

reinforced concrete, the rebar may have many separate areas at different energy levels. Concrete acts as the electrolyte, and the metallic connection is provided by wire ties, chair supports, or the rebar itself.
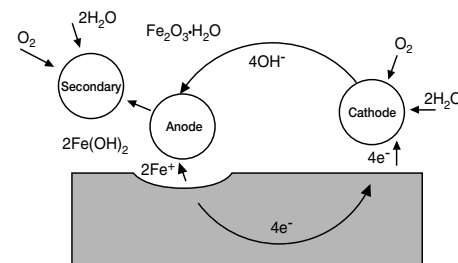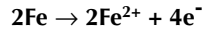

Fig. 3. When reinforcing steel corrodes, electrons flow through the bar and ions flow through the concrete.

half-cell oxidation reaction, or the anodic reaction, and is represented as:

$$2Fe \rightarrow 2Fe^{2+} + 4e^-$$

The electrons remain in the bar and flow to sites called cathodes, where they combine with water and oxygen in the concrete. The reaction at the cathode is called a reduction reaction. A common reduction reaction is:

$$2H_2O + O_2 + 4e^- \rightarrow 4OH^-$$
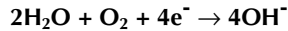
To maintain electrical neutrality, the ferrous ions migrate through the concrete pore water to these cathodic sites where they combine to form iron hydroxides, or rust:

$$2Fe^{2+} + 4OH^- \rightarrow 2Fe(OH)_2$$
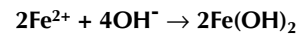
This initial precipitated hydroxide tends to react further with oxygen to form higher oxides. The increases in volume as the reaction products react further with dissolved oxygen leads to internal stress within the concrete that may be sufficient to cause cracking and spalling of the concrete cover.

Corrosion of embedded metals in concrete can be greatly reduced by placing crack-free concrete with low permeability and sufficient concrete cover. Table 1 shows the concrete cover requirements for different exposure conditions as set by ACI 318, *Building Code Requirements for Structural Concrete.*

Table 1. Minimum Concrete Cover Requirements (ACI 318)

Cast-In-Place Concrete	Min. cover, mm (in.)
Concrete cast against and permanently exposed to earth	75 (3)
Concrete exposed to earth or weather:	
No. 19 (No. 6) through No. 57 (No. 18) bars	50 (2)
No. 16 (No. 5) bar, MW200 (W31) or	
MD200 (D31) wire, and smaller	40 (1½)
Concrete not exposed to weather or in contact with ground:	
Slabs, Walls, Joists:	
No. 43 (No. 14) and No. 57 (No. 18) bars	40 (1½)
No. 36 (No. 11) bar and smaller	20 (¾)
Beams, columns:	
Primary reinforcement, ties, stirrups, spirals	40 (1½)
Shells, folded plate members:	
No. 19 (No. 6) bar and larger	20 (¾)
No. 16 (No. 5) bar, MW200 (W31) or	
MD200 (D31) wire, and smaller	15 (½)
Precast Concrete[1]	
Concrete exposed to earth or weather:	
Wall panels:	
No. 43 (No. 14) and No. 57 (No. 18) bars	40 (1½)
No. 36 (No. 11) bar and smaller	20 (¾)
Other members:	
No. 43 (No. 14) and No. 57 (No. 18) bars	50 (2)
No. 19 (No. 6) through No. 36 (No. 11) bars	40 (1½)
No. 16 (No. 5), MW200 (W31) or	30 (1¼)
MD200 (D31) wire, and smaller	
Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists:	
No. 43 (No. 14) and No. 57 (No. 18) bars	30 (1¼)
No. 36 (No. 11) bar and smaller	15 (⅝)
Beams, columns:	
Primary reinforcement	d_b but not less than 15 (⅝) and need not exceed 40 (1½)
Ties, stirrups, spirals	10 (⅜)
Shells, folded plate members:	
No. 19 (No. 6) bar and larger	15 (⅝)
No. 16 (No. 5) bar, MW200 (W31) or	10 (⅜)
MD200 (D31) wire, and smaller	
Prestressed Concrete[2]	
Concrete cast against and permanently exposed to earth	75 (3)
Concrete exposed to earth or weather:	
Wall panels, slabs, joists	25 (1)
Other members	40 (1½)
Concrete not exposed to weather or in contact with ground:	
Slabs, walls, joists	20 (¾)
Beams, columns:	
Primary reinforcement	40 (1½)
Ties, stirrups, spirals	25 (1)
Shells, folded plate members:	
No. 16 (No. 5) bar, MW200 (W31) or	10 (⅜)
MD200 (D31) wire, and smaller	
Other reinforcement	d_b but not less than 20 (¾)

[1]Manufactured under plant controlled conditions. [2]Modification to the cover requirements are possible depending on the manufacturing method and tensile stress in the member. See ACI 318.
d_b = diameter of reinforcing bar

Concrete and the Passivating Layer

Although steel's natural tendency is to undergo corrosion reactions, the alkaline environment of concrete (pH of 12 to 13) provides steel with corrosion protection. At the high pH, a thin oxide layer forms on the steel and prevents metal atoms from dissolving. This passive film does not actually stop corrosion; it reduces the corrosion rate to an insignificant level. For steel in concrete, the passive corrosion rate is typically 0.1 µm per year. Without the passive film, the steel would corrode at rates at least 1,000 times higher (ACI 222 2001).

Because of concrete's inherent protection, reinforcing steel does not corrode in the majority of concrete elements and structures. However, corrosion can occur when the passivating layer is destroyed. The destruction of the passivating layer occurs when the alkalinity of the concrete is reduced or when the chloride concentration in concrete is increased to a certain level.

The Role of Chloride Ions

Exposure of reinforced concrete to chloride ions is the primary cause of premature corrosion of steel reinforcement. The intrusion of chloride ions, present in deicing salts and seawater, into reinforced concrete can cause steel corrosion if oxygen and moisture are also available to sustain

Fig. 4. Deicing salts are a major cause of corrosion of reinforcing steel in concrete. (55807)

the reaction (Fig. 4). Chlorides dissolved in water can permeate through sound concrete or reach the steel through cracks. Chloride-containing admixtures can also cause corrosion.

No other contaminant is documented as extensively in the literature as a cause of corrosion of metals in concrete than chloride ions. The mechanism by which chlorides promote corrosion is not entirely understood, but the most popular theory is that chloride ions penetrate the protective oxide film easier than do other ions, leaving the steel vulnerable to corrosion.

The risk of corrosion increases as the chloride content of concrete increases. When the chloride content at the surface of the steel exceeds a certain limit, called the threshold value, corrosion will occur if water and oxygen are also available. Federal Highway Administration (FHWA) studies found that a threshold limit of 0.20% total (acid-soluble) chloride by weight of cement could induce corrosion of reinforcing steel in bridge decks (Clear 1976). However, only water-soluble chlorides promote corrosion; some acid-soluble chlorides may be bound within aggregates and, therefore, unavailable to promote corrosion. Work at the FHWA (Clear 1973) found that the conversion factor from acid-soluble to water-soluble chlorides could range from 0.35 to 0.90, depending on the constituents and history of the concrete. Arbitrarily, 0.75 was chosen, resulting in a water-soluble chloride limit of 0.15 % by weight of cement. Table 2 shows the maximum permissible water-

soluble chloride-ion content for reinforced concrete in various exposure conditions (ACI 318 2002).

Although chlorides are directly responsible for the initiation of corrosion, they appear to play only an indirect role in the rate of corrosion after initiation. The primary rate-controlling factors are the availability of oxygen, the electrical resistivity and relative humidity of the concrete, and the pH and temperature.

Table 2. Maximum Chloride Ion Content of Concrete (ACI 318)

Type of Member	Maximum Cl-*
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

*Water-soluble chloride, percent by weight of cement.

Carbonation

Carbonation occurs when carbon dioxide from the air penetrates the concrete and reacts with hydroxides, such as calcium hydroxide, to form carbonates. In the reaction with calcium hydroxide, calcium carbonate is formed:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$
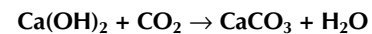
This reaction reduces the pH of the pore solution to as low as 8.5, at which level the passive film on the steel is not stable.

Carbonation is generally a slow process. In high-quality concrete, it has been estimated that carbonation will proceed at a rate up to 1.0 mm (0.04 in.) per year. The amount of carbonation is significantly increased in concrete with a high water-to-cement ratio, low cement content, short curing period, low strength, and highly permeable or porous paste.

Carbonation is highly dependent on the relative humidity of the concrete. The highest rates of carbonation occur when the relative humidity is maintained between 50% and 75%. Below 25% relative humidity, the degree of carbonation that takes place is considered insignificant. Above 75% relative humidity, moisture in the pores restricts CO_2 penetration (ACI 201 1992). Carbonation-induced corrosion often occurs on areas of building facades that are exposed to rainfall, shaded from sunlight, and have low concrete cover over the reinforcing steel (Fig. 5).

Fig. 5. Carbonation-induced corrosion often occurs on building facades with shallow concrete cover. (70157)

Carbonation of concrete also lowers the amount of chloride ions needed to promote corrosion. In new concrete with a pH of 12 to 13, about 7,000 to 8,000 ppm of chlorides are required to start corrosion of embedded steel. If, however, the pH is lowered to a range of 10 to 11, the chloride threshold for corrosion is significantly lower—at or below 100 ppm (Montani 1995). Like chloride ions, however, carbonation destroys the passive film of the reinforcement, but does not influence the rate of corrosion.

Dissimilar Metal Corrosion

When two different metals, such as aluminum and steel, are in contact within concrete, corrosion can occur because each metal has a unique electrochemical potential. A familiar type of dissimilar metal corrosion occurs in an ordinary flashlight battery. The zinc case and carbon rod are the two metals, and the moist paste acts as the electrolyte. When the carbon and zinc are connected by a wire, current flows. In reinforced concrete, dissimilar metal corrosion can occur in balconies where embedded aluminum railings are in contact with the reinforcing steel.

Below is a list of metals in order of electrochemical activity:

1. Zinc
2. Aluminum
3. Steel
4. Iron
5. Nickel
6. Tin
7. Lead
8. Brass
9. Copper
10. Bronze
11. Stainless Steel
12. Gold

When the metals are in contact in an active electrolyte, the less active metal (lower number) in the series corrodes.

FREEZE–THAW DETERIORATION

When water freezes, it expands about 9%. As the water in moist concrete freezes, it produces pressure in the capillaries and pores of the concrete. If the pressure exceeds the tensile strength of the concrete, the cavity will dilate and rupture. The accumulative effect of successive freeze-thaw cycles and disruption of paste and aggregate can eventually cause significant expansion and cracking, scaling, and crumbling of the concrete (Fig. 6). Fig. 7 shows the severity of freeze-thaw exposure typically encountered

in different areas of the United States. Local weather records can also be referenced to more precisely determine the severity of exposure.

The resistance of concrete to freezing and thawing in a moist condition is significantly improved by the use of intentionally entrained air. Entrained air voids act as empty chambers in the paste for the freezing and migrating water to enter, thus relieving the pressure in the capillaries and pores and preventing damage to the concrete. Concrete air content requirements for various exposure conditions are shown in Table 3.

Concrete with low permeability is also better able resist the penetration of water and, as a result, performs better when exposed to freeze-thaw cycles. The permeability of concrete is directly related to its water-to-cement ratio—the lower the water-to-cement ratio, the lower the permeability of the concrete. Table 4 shows the maximum water-to-cementitious materials ratio and minimum compressive strength required for different exposure conditions.
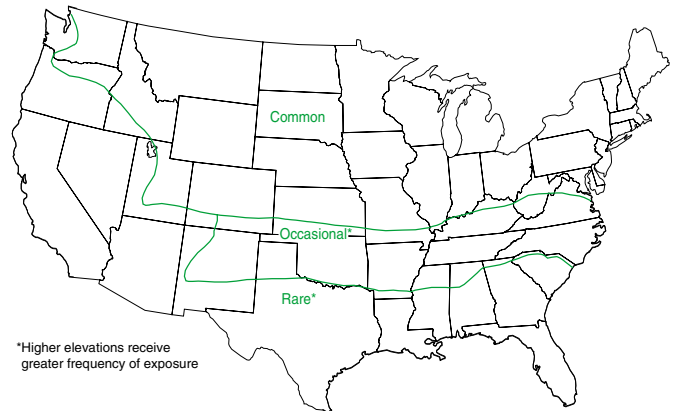

Fig. 7. The frequency of freeze-thaw exposure typically encountered in different areas of the United States.

Fig 6. Freeze-thaw cycles can cause scaling of concrete surfaces. (A5273)

Table 3. Target Air Contents for Frost-Resistant Concrete (ACI 318)

Nominal Maximum Aggregate Size[1], mm (in.)	Air Content, %[2]	
	Severe Exposure	Moderate Exposure
9 (³⁄₈)	7.5	6
13 (¹⁄₂)	7	5.5
19 (³⁄₄)	6	5
25 (1)	6	4.5
37.5 (1¹⁄₂)	5.5	4.5
50[3] (2)	5	4
75[3] (3)	4.5	3.5

[1] See ASTM C 33 for tolerance on oversize for various nominal maximum size designations.

[2] Concrete should have an air content within –1 to +2 percent of the target value.

[3] These air contents apply to total mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 37.5 mm (1¹⁄₂ in.) is removed by hand-picking or sieving, and air content is determined on the minus 37.5 mm (1¹⁄₂ in.) fraction of mix (tolerance on air content as delivered applies to this value). Air content of total mix is computed from value determined on the minus 37.5 mm (1¹⁄₂ in.) fraction.

Table 4. Requirements for Special Exposure Conditions (ACI 318)

Exposure Condition	Maximum w/cm[1]	Minimum Compressive Strength[2]
Concrete intended to have low permeability when exposed to water	0.50	4000
Concrete exposed to freezing and thawing in a moist condition or to deicing chemicals	0.45	4500
For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, saltwater, brackish water, seawater, or spray from these sources	0.40	5000

[1] Water-to-cementitious materials ratio, by weight, normal weight aggregate concrete.

[2] Normal-weight and lightweight aggregate concrete.

Deicer Scaling

Deicing chemicals used for snow and ice removal, such as sodium chloride, can aggravate freeze-thaw deterioration. The additional problem caused by deicers is believed to be a buildup of osmotic and hydraulic pressures in excess of the normal hydraulic pressures produced when water in concrete freezes. In addition, because salt absorbs moisture, it keeps the concrete more saturated, increasing the potential for freeze-thaw deterioration. However, properly designed and placed air-entrained concrete can withstand deicers for many years.

In the absence of freezing, sodium chloride has little to no chemical effect on concrete. Weak solutions of calcium chloride generally have little chemical effect on concrete, but studies have shown that concentrated calcium chloride solutions can chemically attack concrete. Magnesium chloride deicers have come under recent criticism for aggravating scaling. One study found that magnesium chloride, magnesium acetate, magnesium nitrate, and calcium chloride are more damaging to concrete than sodium chloride (Cody, Cody, Spry, and Gan 1996). Deicers containing ammonium nitrate and ammonium sulfate should be prohibited because they rapidly attack and disintegrate concrete (See section on "Chemical Attack," page 7).

Aggregate Expansion

Some aggregates may absorb so much water (to critical saturation) that they cannot accommodate the expansion and hydraulic pressure that occurs during the freezing of water. The result is expansion of the aggregate and possible disintegration of the concrete if enough of the offending particles are present. If a problem particle is near the surface of the concrete, it can cause a popout (Fig. 8).

D-cracking is a form of freeze-thaw deterioration that has been observed in some pavements after three or more years of service. Due to the natural accumulation of water in the base and sub-

Fig. 8. Some aggregates absorb water and, upon freezing, expand to produce a popout. (0113)

Freezing of Fresh Concrete

Concrete gains very little early strength at low temperatures. Freshly mixed concrete must be protected against freezing until the degree of saturation of the concrete has been sufficiently reduced by cement hydration (Fig. 9). The time at which this reduction is accomplished corresponds roughly to the time required for the concrete to attain a compressive strength of 3.5 MPa (500 psi). At normal temperatures and water-to-cement ratios less than 0.60, this typically occurs within the first 24 hours after placement. Significant ultimate strength reductions, up to about 50%, can occur if concrete is frozen within the first few hours after placement or before it attains a compressive strength of 3.5 MPa (500 psi). Concrete to be exposed to deicers should attain a strength of 28 MPa (4000 psi) prior to exposure to cycles of freezing and thawing.

Concrete that has been frozen just once at an early age can often be restored to nearly normal strength by providing favorable subsequent curing conditions. Such concrete, however, will not be as resistant to weathering or as watertight as concrete that had not been frozen. The critical period after which concrete is not seriously damaged by one or two freezing cycles depends on the concrete ingredients and conditions of mixing, placing, curing, and subsequent drying. For example, air-entrained concrete is less susceptible to damage by early freezing than non-air-entrained concrete.

Fig. 9. Ice crystals often form in concrete that is frozen before it hardens. The disruption of the paste matrix by freezing can cause reduced strength gain and increased porosity. (44047)

base of pavements, the aggregate may eventually become saturated. Then with freezing and thawing cycles, cracking of the concrete starts in the saturated aggregate at the bottom of the slab and progresses upward until it reaches the wearing surface (Fig. 10). D-cracking usually starts near pavement joints.

Aggregate freeze-thaw problems can often be reduced by either selecting aggregates that perform better in freeze-thaw cycles or, where marginal aggregates must be used, reducing the maximum particle size.
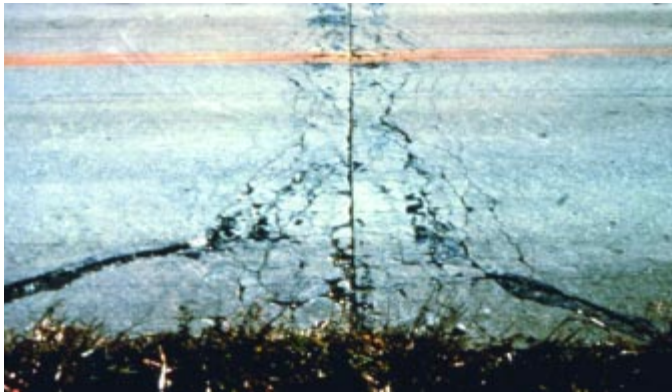

Fig. 10. D-cracking is a form of freeze-thaw deterioration that has been observed in some pavements after three or more years of service. (70155)

CHEMICAL ATTACK

Concrete performs well when exposed to various atmospheric conditions, water, soil, and many other chemical exposures. However, some chemical environments can deteriorate even high-quality concrete. The deleterious effects of some common chemicals on concrete are shown in the box on page 7.

Concrete is rarely, if ever, attacked by solid, dry chemicals. To produce significant attack on concrete, aggressive chemicals must be in solution and above some minimum concentration.

Acids

In general, portland cement concrete does not have good resistance to acids. In fact, no hydraulic cement concrete, regardless of its composition, will hold up for long if exposed to a solution with a pH of 3 or lower. However, some weak acids can be tolerated, particularly if the exposure is occasional.

Acids react with the calcium hydroxide of the hydrated portland cement. In most cases, the chemical reaction forms water-soluble calcium compounds, which are then leached away by aqueous solutions (ACI 201 1992).

The products of combustion of many fuels contain sulfurous gases which combine with moisture to form sulfuric acid. Also, certain bacteria convert sewage into sulfuric acid. Sulfuric acid is particularly aggressive to concrete because the calcium sulfate formed from the acid reaction will also deteriorate concrete via sulfate attack (Fig. 11).

Fig. 11. Bacteria in sewage systems can produce sulfuric acid, which aggressively attacks concrete. (70149)

In addition to individual organic and mineral acids which may attack concrete, acid-containing or acid-producing substances, such as acidic industrial wastes, silage, fruit juices, and sour milk, will also cause damage.

Animal wastes contain substances which may oxidize in air to form acids which attack concrete. The saponification reaction between animal fats and the hydration products of portland cement consumes these hydration products, producing salts and alcohols, in a reaction analogous to that of acids.

Acid rain, which often has a pH of 4 to 4.5, can slightly etch concrete, usually without affecting the performance of the exposed surface.

Any water that contains bicarbonate ion also contains free carbon dioxide, a part of which can dissolve calcium carbonate unless saturation already exists. This part is called the "aggressive carbon dioxide." Water with aggressive carbon dioxide acts by acid reaction and can attack concrete and other portland cement products whether or not they are carbonated.

Calcium-absorptive acidic soil can attack concrete, especially porous concrete. Even slightly acidic solutions that are lime-deficient can attack concrete by dissolving calcium from the paste, leaving behind a deteriorated paste consisting primarily of silica gel.

To prevent deterioration from acid attack, portland cement concrete generally must be protected from acidic environments with surface protective treatments. Unlike limestone and dolomitic aggregates, siliceous aggregates are acid-resistant and are sometimes specified to improve the chemical resistance of concrete, especially with the use of chemical-resistant cement. Properly cured concrete with reduced permeability experience a slightly lower rate of attack from acids.

Salts and Alkalis

The chlorides and nitrates of ammonium, magnesium, aluminum, and iron all cause concrete deterioration, with those of ammonium producing the most damage. Most ammonium salts are destructive because, in the alkaline environment of concrete, they release ammonia gas and hydrogen ions. These are replaced by dissolving calcium hydroxide from the concrete. The result is

a leaching action, much like acid attack. Strong alkalies (over 20 percent) can also cause concrete disintegration (ACI 515 1979).

Chemicals That Deteriorate Concrete

Promote rapid deterioration of concrete:
Aluminum Chloride
Calcium Bisulfite
Hydrochloric Acid (all concentrations)*
Hydrofluoric Acid (all concentrations)
Nitric Acid (all concentrations)
Sulfuric Acid, 10-80 percent*
Sulfurous Acid

Promote moderate deterioration of concrete:

Aluminum Sulfate*	Mustard Oil*
Ammonium Bisulfate	Perchloric Acid, 10%
Ammonium Nitrate	Potassium Dichromate
Ammonium Sulfate*	Potassium Hydroxide (>25%)
Ammonium Sulfide	Rapeseed Oil*
Ammonium Sulfite	Slaughterhouse Waste[2]
Ammonium Superphosphate	Sodium Bisulfate
Ammonium Thiosulfate	Sodium Bisulfite
Castor Oil	Sodium Hydroxide (>20%)
Cocoa Bean Oil*	Sulfite Liquor
Cocoa Butter*	Sulfuric Acid, 80% Oleum*
Coconut Oil*	Tanning Liquor (if acid)
Cottonseed Oil*	Zinc Refining Solutions[3]
Fish Liquor[1]	

* Sometimes used in food processing or as food or beverage ingredient. Ask for advisory opinion of Food and Drug Administration regarding coatings for use with food ingredients.
[1] Contains carbonic acid, fish oils, hydrogen sulfide, methyl amine, brine, other potentially active materials
[2] May contain various mixtures of blood, fats and oils, bile and other digestive juices, partially digested vegetable matter, urine, and manure, with varying amounts of water.
[3] Usually contains zinc sulfate in sulfuric acid. Sulfuric acid concentration may be low (about 6 percent in "low current density" process) or higher (about 22-28% in "high current density" process).

Sulfate Attack

Naturally occurring sulfates of sodium, potassium, calcium, or magnesium are sometimes found in soil or dissolved in groundwater. Sulfates can attack concrete by reacting with hydrated compounds in the hardened cement. These reactions can induce sufficient pressure to disrupt the cement paste, resulting in loss of cohesion and strength. Calcium sulfate attacks calcium aluminate hydrate and forms ettringite. Sodium sulfate reacts with calcium hydroxide and calcium aluminate hydrate forming ettringite and gypsum. Magnesium sulfate attacks in a manner similar to sodium sulfate and forms ettringite, gypsum, and brucite (magnesium hydroxide). Brucite forms primarily on the concrete surface, consumes calcium hydroxide, lowers the pH of the pore solution, and then decomposes the calcium silicate hydrates.

Environmental conditions have a great influence on sulfate attack. The attack is greater in concrete exposed to wet/dry cycling (Fig. 12). When water evaporates, sulfates can accumulate at the concrete surface, increasing in concentration and their potential for causing deterioration.

Porous concrete is susceptible to weathering caused by salt crystallization. Examples of salts known to cause weathering of field concrete include sodium carbonate and sodium sulfate (laboratory studies have also related saturated solutions of calcium chloride and other salts to concrete deterioration). Under drying conditions, salt solutions can rise to the surface by capillary action and, as a result of surface evaporation, the solution phase becomes supersaturated and salt crystallization occurs, sometimes generating pressures large enough to cause cracking and scaling (Mehta 2000).

Thaumasite may form during sulfate attack in moist conditions at temperatures usually between 0°C and 10°C (32°F to 50°F) and it occurs as a result of a reaction between calcium silicate hydrate, sulfate, calcium carbonate, and water. In concrete undergoing excessive thaumasite formation, cracks can be filled with thaumasite and haloes of white thaumasite surround aggregates. At the concrete/soil interface, the surface concrete layer can be "mushy" with complete replacement of the cement paste by thaumasite.

Sulfate attack is a particular problem in arid areas, such as the Northern Great Plains and parts of the Western United States. Seawater also contains sulfates but is not as severe an exposure as sulfates in groundwater.

Resistance to sulfates can best be achieved by using a low water-to-cement ratio and a cement with a limited amount of tricalcium aluminates. As outlined in ASTM C 150, Type II cement contains less than 8% C_3A, and Type V cement contains less than 5%. Cements meeting the ASTM C 1157 requirements of Type MS cement (moderate sulfate resistant) and Type HS cement (high sulfate resistant) can also be used to provide sulfate resistance, as well as moderate sulfate-resistant cements per ASTM C 595.

Studies have shown that some pozzolans and ground-granulated blast-furnace slags increase the life expectancy of concrete exposed to sulfates. Good results have been obtained with fly ash

Fig 12. The bases of these concrete posts have suffered from sulfate attack. (66900)

Table 5. Type of Cement Required for Concrete Exposed to Sulfates in Soil or Water

Sulfate exposure	Water-soluble sulfate (SO_4) in soil, percent by mass	Sulfate (SO_4) in water, ppm	Cement type[2]	Maximum water-cementitious material ratio, by mass	Minimum design compressive strength, MPa (psi)
Negligible	Less than 0.10	Less than 150	No special type required	—	—
Moderate[1]	0.10 to 0.20	150 to 1500	II, MS, IP(MS), IS(MS), P(MS) I(PM)(MS), I(SM)(MS)	0.50	28 (4000)
Severe	0.20 to 2.00	1500 to 10,000	V, HS	0.45	31 (4500)
Very severe	Over 2.00	Over 10,000	V, HS	0.40	35 (5000)

[1] Seawater.
[2] Pozzolans or slags that have been determined by test or service record to improve sulfate resistance may also be used.
   Test method: *Method for Determining the Quantity of Soluble Sulfate in Solid (Soil or Rock) and Water Samples,* Bureau of Reclamation, 1977.
   Source: Adapted from Bureau of Reclamation 1981 and ACI 318.

meeting the requirements of ASTM C 618 Class F. Slags should conform to ASTM C 989. However, some pozzolans, especially some Class C fly ashes, decrease sulfate resistance. Therefore, pozzolans chosen to improve sulfate resistance should be tested to confirm their behavior.

Calcium chloride reduces sulfate resistance, so it should not be used as an accelerating admixture in concrete exposed to severe and very severe sulfate environments. Table 5 shows the requirements for concrete exposed to sulfates.

ALKALI-AGGREGATE REACTIVITY

In most concrete, aggregates are more or less chemically inert. However, some aggregates react with the alkali hydroxides in concrete, causing expansion and cracking over a period of years. This alkali-aggregate reactivity has two forms—alkali-silica reaction (ASR) and alkali-carbonate reaction (ACR). ASR is of more concern than ACR because aggregates containing reactive silica materials are more common.

Heat-Induced Delayed Expansion

Heat-induced delayed expansion (HIDE)—also called delayed ettringite formation (DEF)—is a rare condition of internal sulfate attack in which mature concretes undergo expansion and cracking (Fig. 13). Concretes may be affected when they have reached high temperatures (over 70°C [158°F]) depending on the concrete ingredients and the time the temperature is reached after casting), usually after the first few hours of placement.

The mechanism causing the expansion is not fully understood. At high temperatures, some of the initial ettringite in the cement paste may be converted to monosulfoaluminate and, upon cooling, revert back to ettringite. Because ettringite occupies more space than monosulfoaluminate, the reversion is expansive. The mechanism responsible for this concrete degradation is still being investigated.

Only concretes in massive elements that retain the heat of hydration or elements exposed to very high temperatures at an early age are at risk of HIDE; and of these, only a few have the chemical makeup or temperature profile to cause detrimental expansion. Normal-sized concrete elements cast and maintained near ambient temperatures cannot experience HIDE.

Fly ash and slag may help control heat-induced delayed expansion, along with control over early-age temperature development.
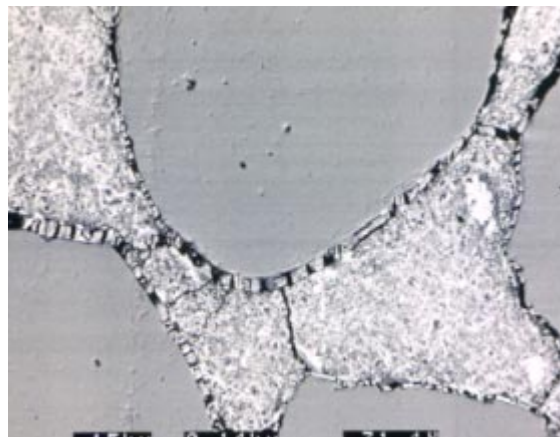

Fig. 13. Heat-induced delayed expansion is characterized by expanding paste that becomes detached from the aggregate. (69154)

Alkali-Silica Reactivity

Aggregates containing certain forms of silica will react with alkali hydroxide in concrete to form a gel that swells as it draws water from the surrounding cement paste or the environment. In absorbing water, these gels can swell and induce enough expansive pressure to damage concrete:

> **1. Alkalies + Reactive Silica → Gel Reaction Product**
>
> **2. Gel Reaction Product + Moisture → Expansion**

Typical indicators of alkali-silica reactivity are map (random pattern) cracking and, in advanced cases, closed joints and spalled concrete surfaces (Fig. 14). Cracking usually appears in areas with a frequent supply of moisture, such as close to the waterline in piers, from the ground behind retaining walls, near joints and free edges in pavements, or in piers or columns subject to wick action.

Because sufficient moisture is needed to promote destructive expansion, alkali-silica reactivity can be significantly reduced by keeping the concrete as dry as possible. The reactivity can be virtually stopped if the internal relative humidity of the concrete is kept below 80%. In most cases, however, this condition is difficult to achieve and maintain. Warm seawater, due to the presence of dissolved alkalies, can particularly aggravate alkali-silica reactivity.

Alkali-silica reactivity can be controlled using certain mineral admixtures. Silica fume, fly ash, and ground-granulated blast-furnace slag have significantly reduced alkali-silica reactivity. Class F fly ashes have reduced reactivity expansion up to 70% or more in some cases. In some cases, lithium compounds have been shown to effectively reduce ASR.

Although potentially reactive aggregates exist throughout North America, ASR distress in concrete is not common because of measures taken to control it. It is also important to note that not all ASR gel reaction products undergo destructive swelling.

Fig. 14. Typical indicators of alkali-silica reactivity are map cracking and, in advanced cases, closed joints and spalled concrete surfaces. (56586)
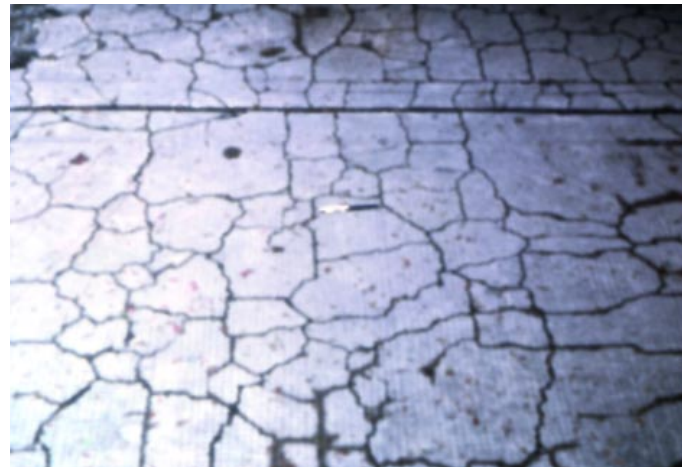

Fig. 15. Map cracking pattern caused by alkali-carbonate reactivity. (13664)

Alkali-Carbonate Reactivity

Reactions observed with certain dolomitic rocks are associated with alkali-carbonate reaction (ACR). Dedolomitization, or the breaking down of dolomite, is normally associated with expansive alkali-carbonate reactivity. This reaction and subsequent crystallization of brucite may cause considerable expansion. The deterioration caused by alkali-carbonate reaction is similar to that caused by alkali-silica reaction (Fig. 15); however, alkali-carbonate reaction is relatively rare because aggregates susceptible to this reaction are less common and are usually unsuitable for use in concrete for other reasons, such as strength potential.

ABRASION/EROSION

Abrasion damage occurs when the surface of concrete is unable to resist wear caused by rubbing and friction. As the outer paste of concrete wears, the fine and coarse aggregate are exposed and abrasion and impact will cause additional degradation that is related to aggregate-to-paste bond strength and hardness of the aggregate.

Although wind-borne particles can cause abrasion of concrete, the two most damaging forms of abrasion occur on vehicular traffic surfaces and in hydraulic structures, such as dams, spillways, and tunnels.

Traffic Surfaces

Abrasion of floors and pavements may result from production operations or vehicular traffic. Many industrial floors are subjected to abrasion by steel or hard rubber wheeled traffic, which can cause significant rutting.

Tire chains and studded snow tires cause considerable wear to concrete surfaces (Fig. 16). In the case of tire chains, wear is caused by flailing and scuffing as the rotating tire brings the metal in contact with the concrete surface.

Fig 16. Tire chains and studded snow tires can cause considerable wear to concrete surfaces. (70156)

Fig 17. The abrasion of debris in rushing water can produce significant wear on concrete surfaces. (70150)

In some areas, abrasive materials such as sand are applied to pavements to improve traction, but experience has shown that this causes little wear if the concrete is of good quality and the aggregates are wear resistant.

Compressive strength is the most important factor controlling the abrasion resistance of concrete, with abrasion resistance increasing with increase in compressive strength. The service life of some concrete, such as warehouse floors subjected to abrasion by steel or hard rubber wheels, may be greatly increased by the use of specially hard or tough aggregate.

Hydraulic Structures

Abrasion damage in hydraulic structures is caused by the abrasive effects of waterborne silt, sand, gravel, rocks, ice, and other debris impinging on the concrete surface. Although high-quality concrete can resist high water velocities for many years with little or no damage, the concrete may not withstand the abrasive action of debris grinding or repeatedly impacting on its surface (Fig. 17). In such cases, abrasion erosion ranging from a few millimeters (inches) to several meters (feet) can result, depending on flow conditions. Spillway aprons, stilling basins, sluiceways, drainage conduits or culverts, and tunnel linings are particularly susceptible to abrasion erosion. Abrasion erosion is readily recognized by its smooth, worn appearance, which is distinguished from the small holes and pits formed by cavitation erosion.

As is the case with traffic wear, abrasion damage in hydraulic structures can be reduced by using strong concrete with hard aggregates.

Cavitation is the formation of bubbles or cavities in a liquid. In hydraulic structures, the liquid is water and the cavities are filled with water vapor and air. The cavities form where the local pressure drops to a value that will cause the water to vaporize at the prevailing fluid temperature. Cavitation damage is produced when the vapor cavities collapse, causing very high instantaneous pressures that impact on the concrete surfaces, causing pitting, noise, and vibration.

Once cavitation damage has substantially altered water flow, other deterioration mechanisms come into play. Fatigue due to vibration, rushing water striking irregular surfaces, and mechanical failure due to vibrating reinforcing steel can cause significant concrete damage.

Pitting by cavitation is readily distinguished from the worn appearance caused by abrasion because cavitation pits cut around the harder coarse aggregate particles and have irregular and rough edges. Severe cavitation damage will typically form a "Christmas tree" configuration on spillway chute surfaces downstream from the point of origin.

Although proper material selection can increase the cavitation resistance of concrete, the only completely effective solution is to design hydraulic structures to reduce or eliminate the factors that trigger cavitation. Even the strongest materials cannot withstand the forces of cavitation indefinitely.

FIRE/HEAT

Concrete performs exceptionally well at the temperatures encountered in almost all applications. But when exposed to fire or unusually high temperatures, concrete can lose strength and stiffness (Fig. 18).

Fig. 19 shows the effect of high temperatures on the compressive strength, flexural strength, and modulus of elasticity of cured concrete, as determined by various investigators (Lankard 1968). As shown in the graphs, modulus of elasticity is the most sensitive to elevated temperature, followed by flexural strength and compressive strength.

The graphs represent a wide range of values because several factors influence the performance of concrete at elevated temperatures. Numerous studies have found the following general trends:

• Concrete that undergoes thermal cycling suffers greater loss of strength than concrete that is held at a constant temperature, although much of the strength loss occurs in the first few cycles. This is attributed to incompatible dimensional changes between the cement paste and the aggregate.

• Concrete that is under design load while heated loses less strength than unloaded concrete, the theory being that

Fig. 18. When exposed to fire or unusually high temperatures, concrete can lose strength and stiffness. (55707)

imposed compressive stresses inhibit development of cracks that would be free to develop in unrestrained concrete.

- Concrete that is allowed to cool before testing loses more compressive strength than concrete that is tested hot. Concrete loses more strength when quickly cooled (quenched) from high temperatures than when it is allowed to cool gradually.

- Concrete containing limestone and calcareous aggregates performs better at high temperatures than concrete containing siliceous aggregates (Abrams 1956). One study showed no difference in the performance of dolostone and limestone (Carette 1982). Another study showed the following relative aggregate performance, from best to worst: firebrick, expanded shale, limestone, gravel, sandstone and expanded slag.

- Proportional strength loss is independent of compressive strength of concrete.

- Concrete with a higher aggregate-cement ratio suffers less reduction in compressive strength; however, the opposite is true for modulus of elasticity. The lower the water-cement ratio, the less loss of elastic modulus.

- If residual water in the concrete is not allowed to evaporate, compressive strength is greatly reduced. If heated too quickly, concrete can spall as the moisture tries to escape.
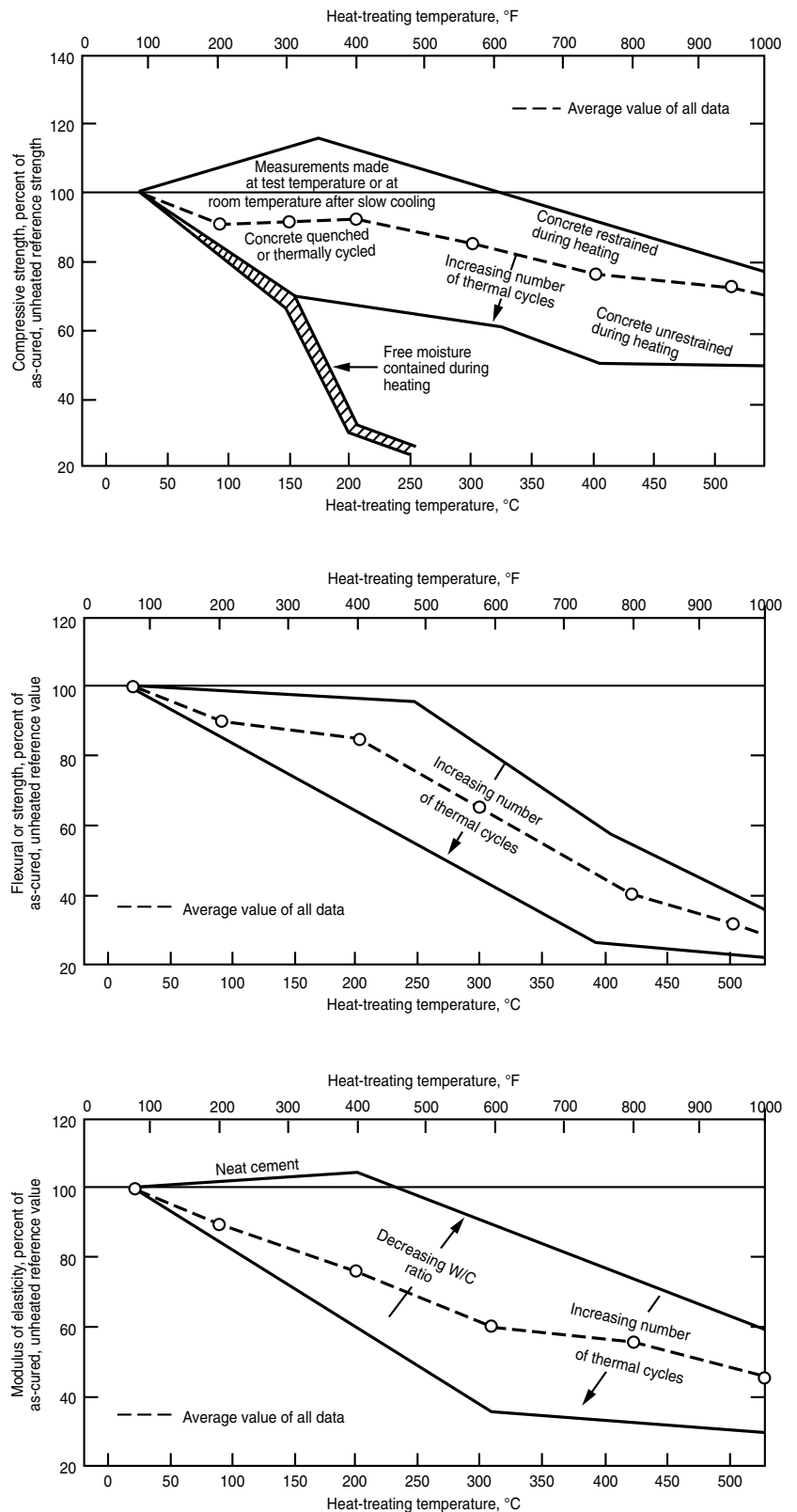

Fig. 19. The effect of elevated temperatures on the compressive strength (top), flexural strength (middle), and modulus of elasticity (bottom) of concrete (Lankard 1968).

RESTRAINT TO VOLUME CHANGES

Concrete changes slightly in volume for various reasons, the most common causes being fluctuations in moisture content and temperature. Restraint to volume changes, especially contraction, can cause cracking if the tensile stresses that develop exceed the tensile strength of the concrete.

Plastic Shrinkage Cracking

When water evaporates from the surface of freshly placed concrete faster than it is replaced by bleed water, the surface concrete shrinks. Due to the restraint provided by the concrete below the drying surface layer, tensile stresses develop in the weak, stiffening plastic concrete, resulting in shallow cracks of varying depth (Fig. 20). These cracks are often fairly wide at the surface.

Plastic shrinkage cracks can be prevented by taking measures to prevent rapid water loss from the concrete surface. Fog nozzles, plastic sheeting, windbreaks, and sunshades can all be used to prevent excessive evaporation.

Fig. 20. Plastic shrinkage cracks can occur when water evaporates from the surface faster than it is replaced by bleedwater. (1311)

Drying Shrinkage Cracking

Because almost all concrete is mixed with more water than is needed to hydrate the cement, much of the remaining water evaporates, causing the concrete to shrink. Restraint to shrinkage, provided by the subgrade, reinforcement, or another part of the structure, causes tensile stresses to develop in the hardened concrete. Restraint to drying shrinkage is the most common cause of concrete cracking (Fig. 21).

In many applications, drying shrinkage cracking is inevitable. Therefore, control joints are placed in concrete to predetermine the location of drying shrinkage cracks. Drying shrinkage can be limited by keeping the water content of concrete as low as possible and maximizing the coarse aggregate content.

Fig 21. Restraint to drying shrinkage is the most common cause of concrete cracking. (A5271)

Thermal Cracking

Concrete expands when heated and contracts when cooled. An average value for the thermal expansion of concrete is about 10 millionths per degree Celcius (5.5 millionths per degree Fahrenheit). This amounts to a length change of 5 mm for 10 m of concrete (⅔ in. for 100 ft of concrete) subjected to a rise or fall of 50°C (90°F).

Thermal expansion and contraction of concrete varies with factors such as aggregate type, cement content, water-cement ratio, temperature range, concrete age, and relative humidity. Of these, aggregate type has the greatest influence.

Designers should give special consideration to structures in which some portions of the structure are exposed to temperature changes, while other portions are partially or completely protected. Allowing for movement by using properly designed expansion or isolation joints and correct detailing will help minimize the effects of temperature variations.

OVERLOAD AND IMPACT

Properly designed and constructed concrete members are usually strong enough to support the loads for which they are intended. But overloading can occur for a variety of reasons — a change in use of a structure without proper structural upgrades, unintentional overloading, and other unusual circumstances. **Earthquake damage** is a classic example of the overloading of concrete structures.

Overload damage can occur during construction when concrete has not yet reached design strength. Early removal of formwork or the storage of heavy materials or operation of equipment on and around the structure can result in the overloading of certain concrete members. A common error occurs when precast members are not properly supported during transport and erection. Errors in post-tensioned construction, such as improperly timed or sequenced strand release, can also cause overload cracking.

Damage caused by impact is another form of overload. A common form of impact overload occurs at slab edges of joints on vehicular traffic surfaces (Fig. 22).

Fig. 22. A common form of impact overload occurs at slab edges of joints on vehicular traffic surfaces. (70151)

Even in properly designed reinforced concrete, load-induced tensile stresses can occur. This point is readily acknowledged and accepted in concrete design. Current design procedures use reinforcing steel to not only carry tensile loads, but to obtain both an adequate distribution of cracks and a reasonable limit on crack width.

LOSS OF SUPPORT

Loss of support beneath concrete structures, usually caused by settling or washout of soils and subbase materials, can cause a variety of problems in concrete structures, from cracking and performance problems to structural failure (Fig 23). Loss of support can also occur during construction due to inadequate formwork support or premature removal of forms.

A common problem related to loss of support is **slab curling**. Curling is the rise of a slab's edges and corners caused by differences in moisture content or temperature between the top and bottom of a slab. The top dries out or cools, and contracts more than the wetter, warmer bottom. Curling results in a loss of contact between the slab and the subbase, which can lead to cracking, slab deflection, and joint deterioration when vehicular traffic crosses the joints.

Fig. 23. Settlement can cause a variety of problems in concrete structures, from cracking and performance problems to structural failure. (56521)

SURFACE DEFECTS

Various defects can occur on the surface of formed or finished concrete. Many of these defects are avoidable by using proper materials and construction practices; others are difficult or impossible to eliminate completely.

Formed Surfaces

Surface air voids, also known as **bugholes**, are small cavities that are the result of entrapped air bubbles in the surface of formed concrete during placement and consolidation (Fig. 24). They can be up to 25 mm (1 in.) wide, but are usually no more than 15 mm (9/16 in.) wide. Bug holes on vertical surfaces are more likely to occur in sticky or stiff concrete mixes of low workability that may have an excessive fine aggregate content, entrapped air content, or both. Also, the use of vibrators with too high of an amplitude, or the lack of complete insertion of the vibrator head may result in an increased quantity of bug holes. Bugholes are also aggravated by the heavy application of form release agent.

Fig. 24. Surface air voids, or bugholes, are small cavities of entrapped air bubbles in the surface of formed concrete. (70154)

Honeycomb occurs when mortar fails to fill the spaces between coarse aggregates (Fig. 25). Congested reinforcement, segregation, and insufficient fine aggregate contents can contribute to honeycombing. Higher concrete slumps and vibration may assist in preventing honeycombing by increasing the flowability of the concrete.

Form tie holes are voids intentionally cast into concrete surfaces. A common type of form tie uses plastic cones at the form surface. The cones act as a spreader for the forms, aid in reducing mortar leaks, and make breaking back snap ties easier. After removal, a typical cone leaves a hole in the concrete surface about 25 mm (1 in.) wide and 38 mm (1½ in.) deep. Form tie holes may need to be filled to prevent leakage or to increase concrete cover over embedded steel.

Cold joints are discontinuities in concrete members resulting from a delay in placement of sufficient time to prevent a union of the material in two successive lifts. Cold joints often result in visible lines indicating the presence of a joint where one layer of concrete had hardened before subsequent concrete was placed (Fig. 26). Aside from their appearance, cold joints can be a

Fig. 25. Honeycomb occurs when mortar fails to fill the spaces between coarse aggregates. (70152)

concern if they allow moisture penetration or if the loss of tensile strength of the concrete across the joint is deemed detrimental to the performance of the structure.

Form streaks are areas of fine or coarse aggregate left on the concrete surface after mortar leaks through form joints and tie holes. Overvibration and the use of excessively wet or high-slump concrete increases the chances of form streaking, as do unsealed formwork joints.

Sand streaking is a streak of fine aggregate in the concrete surface caused by heavy bleeding along the form. It often results from the use of harsh, wet mixes, particularly those deficient in aggregate sizes of 300 μm (No. 50) and smaller.

Form offsets are abrupt to gradual surface irregularities that are usually caused by inadequate stiffness or anchorage of the forms and can be aggravated by an excessive rate of placement or by an excessively powerful vibrator.

Fig. 26. Cold joints often result in visible lines where one layer of concrete had hardened before subsequent concrete was placed, as shown here in a sloping horizontal line. (70153)

Finished Surfaces

Delaminations occur when air and bleed water become trapped under a prematurely closed (densified) mortar surface. The trapped air and bleed water separate the upper 3- to 6-mm (¹⁄₈- to ¹⁄₄-inch) layer of mortar from the underlying concrete.

Delaminations are very difficult to detect during finishing and become apparent after the concrete surface has dried and the delaminated area is crushed under traffic. A smaller and more noticeable form of delamination is a **blister**, a bump that forms at the concrete surface from trapped air and bleed water (Fig. 27).

Fig. 27. Delaminations and blisters occur when air and bleed water become trapped under a prematurely closed (densified) mortar surface. (Photo courtesy of NRMCA.) (A5272, 49411)

The primary cause of delaminations is finishing the surface before bleeding is complete. Delaminations are more likely to occur when factors that extend the bleeding time of concrete are combined with factors that accelerate surface setting.

Dusting is the development of a fine, powdery material that easily rubs off the surface of hardened concrete (Fig. 28). It is the result of a thin, weak surface layer, called laitance, which is composed of water, cement, and fine particles.

Floating and troweling bleed water back into the concrete surface can cause dusting. Other causes include using too wet a mix, spreading dry cement over the surface to accelerate finishing, and allowing rapid drying of the surface. Unvented heaters can also contribute to dusting by supplying carbon dioxide, which reacts with calcium hydroxide in fresh concrete to form a weak layer of calcium carbonate on the surface.

A **popout** is a fragment that breaks out of the surface of concrete, leaving a hole that is usually 6 to 50 mm (¹⁄₄ to 2 in.) in diameter

Fig. 28. Dusting is the development of a fine, powdery material that easily rubs off the surface of hardened concrete. (1297)

(see Fig. 8). The cause of a popout usually is a piece of porous rock having a high rate of absorption and relatively low specific gravity. As the offending aggregate absorbs moisture or freezing occurs under moist conditions, its swelling creates internal pressures sufficient to rupture the concrete surface. Pyrite, hard-burned dolomite, coal, shale, soft fine grained limestone, or chert commonly cause popouts.

Some popouts are caused by alkali-silica reactivity. In some areas, sand-sized ASR-induced popouts are a common problem on floors that receive impermeable coverings.

Subsidence cracks may develop over embedded items, such as reinforcing steel, or adjacent to forms or hardened concrete as the concrete settles or subsides (Fig. 29). Subsidence cracking results from insufficient consolidation (vibration), high slumps (overly wet concrete), or a lack of adequate cover over embedded items.

Crazing is a pattern of fine cracks that do not penetrate much below the surface and are usually a cosmetic problem only (Fig. 30). They are barely visible, except when the concrete is drying after the surface has been wet. Preventing excessive evaporation during placement and proper curing can prevent crazing.
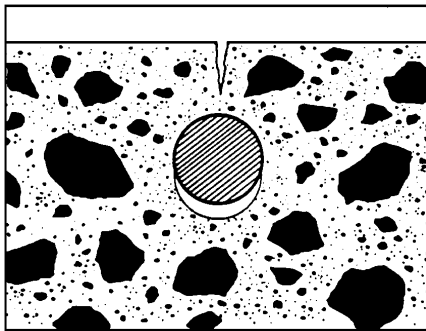

Fig. 29. Subsidence cracks can develop over reinforcing steel as the concrete settles or subsides.

Fig. 30. Crazing typically does not penetrate much below the surface and is usually a cosmetic problem only. (4099)

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

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