3.1 INTRODUCTION

Prior to the early 1930s, concrete was a relatively simple, straightforward material, consisting only of “pure” Portland cement, aggregate, and water. These were combined in varying proportions to produce concretes with a range of compressive strengths and workabilities. In North America, it was only then that the first concrete additives began to come into general use: air-entraining, set-controlling, and water-reducing admixtures, and fly ash as a cement replacement. In Europe, ground-granulated blast-furnace slag (GGBFS) had been used since the beginning of the twentieth century, but the chemical admixtures were still not available until the 1930s. More recently, the numbers and types of materials being added to Portland cement and concrete have been growing at an accelerating pace, driven by a combination of economic considerations and a desire to make the cement and concrete industries more sustainable. Of course, both durable and non-durable concretes are made using much the same materials. The object is to ensure that the right materials are chosen for any particular project, taking into consideration both the exposure conditions and any special requirements for the concrete. In what follows, the materials found in modern concretes will be examined primarily in terms of their impact on the durability of concrete. This chapter deals only with the concrete-making materials themselves; the corrosion of steel reinforcement is dealt with in Chapters 4 and 8, and other durability issues are described in greater detail elsewhere in this book.

3.2 PORTLAND CEMENTS AND OTHER BINDERS

Of all of the materials used to make concrete, it is the Portland cement-based binder that is the most unstable part of the system, and is responsible for most of the durability problems that may beset concrete. It may be subject to both chemical and physical attack, as shown in Table 3.1. (A much more detailed list of deterioration mechanisms is provided in Chapter 4,
The durability issues are inherent with the use of Portland cements; they are functions of the physical and chemical nature of these cements. Fortunately, we already know how to deal with most of these problems.

It should be noted that in North America, where the bulk of cement production falls within the parameters of the five basic ASTM cement types, long-term performance data are readily available for both strength and durability. However, in Europe, the classification of EN cements (BS EN 197-1, 2011) presents a bewildering array of possibilities, as already shown in Table 2.1. The vast majority of cements on the European market are blended, often heavily so. Except for strength, there is little performance data on these “newer” cements, and so their long-term durability remains somewhat uncertain.

Of course, the first line of defense is to keep the water/binder (w/b) ratio as low as possible (and to ensure that the concrete is properly cured). This will greatly reduce the permeability of the concrete, thus inhibiting the ingress of deleterious chemicals into the interior of the concrete. Further reductions in permeability can be achieved by adding supplementary cementing materials (SCMs). In particular, silica fume leads to a reduction in both total porosity and the size of the pores. While these measures cannot eliminate chemical attack, they can slow it down considerably, thereby increasing the effective life of the concrete.

### 3.2.1 Chemical attack

In most cases, not much can be done with the cementing materials themselves to eliminate leaching and efflorescence, or chemical attack from acids or from water containing significant amounts of industrial or agricultural wastes. These types of problems are governed by the chemistry
of the cements and of the deleterious chemicals. As stated above, these reactions can be delayed only by using a low w/b and the use of certain SCMs. However, there are some instances in which altering the cement (or binder) chemistry can be highly beneficial.

### 3.2.1.1 Alkali–aggregate reaction

For instance, where alkali–aggregate reaction (AAR) is likely to occur, reducing the alkali equivalent of the cement to below 0.6%, limiting the total alkali content of the concrete, and using a low w/b ratio will generally mitigate the problem (Blight and Alexander, 2011), or at least considerably diminish the rate of deterioration.

### 3.2.1.2 Sulfate attack

The mechanisms involved in sulfate attack are complex, and have been described in detail by Skalny et al. (2002). The requirements to mitigate this attack, however, are quite straightforward. Basically, where sulfate attack is expected, the use of a low C₃A cement is required. This must be used in combination with a low w/b ratio to be effective. Typical requirements for sulfate-resistant concrete based on the Canadian standard (CSA A23.1-14, 2014) are given in Chapter 9 (Table 9.1).

In North America, there are basically two common sulfate-resistant cements:

- ASTM Type II: Moderate sulfate resistance, with C₃A ≤ 8%
- ASTM Type V: High sulfate resistance, with C₃A ≤ 5%

These C₃A requirements can most often be met by using a blended cement, where the C₃A content is “diluted” by substituting an SCM for a portion of the Portland cement. Equivalence in sulfate resistance performance can be established between concretes made with sulfate-resistant cements and concretes made with normal Portland cement but with a lower w/b ratio, as shown for example in Chapter 2, Figure 2.3.

The situation is rather more complex in Europe, where blended cements are much more common than they are in North America. The current EU standard, EN-197-1 (2011) defines seven sulfate-resistant common cements (SR cements):

<table>
<thead>
<tr>
<th>Sulfate-resisting Portland cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I-SR0</td>
</tr>
<tr>
<td>C₃A content of clinker = 0%</td>
</tr>
<tr>
<td>CEM I-SR3</td>
</tr>
<tr>
<td>C₃A content of clinker ≤ 3%</td>
</tr>
<tr>
<td>CEM I-SR5</td>
</tr>
<tr>
<td>C₃A content of clinker ≤ 5%</td>
</tr>
</tbody>
</table>

Continued
The compositions of these SR cements are given in Table 3.2.

### 3.2.1.3 Acid attack

Concrete will be attacked by acids, the severity of the attack depending on the type of acid and its concentration. The Ca(OH)₂ in the hydrated cement is particularly susceptible to acid attack, but the C–S–H may also be attacked and, in some circumstances, the aggregate as well. The attack will occur when the pH of the liquid is below 6.5, becoming increasingly severe as the pH drops. The attack may be mitigated (but not eliminated) by blending the Portland cement with SCMs at replacement levels up to about 60% (Tamimi, 1997), such as fly ash, GGBFS, silica fume, or metakaolin, since this effectively reduces the amount of Ca(OH)₂ (Torii and Kawamura, 1994; de Belie et al., 1996; Kim et al., 2007).

### 3.2.1.4 Other considerations

It should be noted that modern concretes, and particularly high-performance concretes, are now much less “forgiving” than they used to be. That is, they
are much more sensitive to exactly which raw materials are used, how they are mixed together, and how the concrete is placed and cured. In part, this is related to changes in the chemistry and fineness of grinding of the Portland cement, which in turn are being driven by the desire of contractors and owners to accelerate the rate at which concrete gains strength (Aïtcin and Mindess, 2015). This is particularly true for binders containing large quantities of fly ash or similar pozzolanic materials, which would otherwise have a somewhat slower rate of strength gain because the pozzolanic reaction is relatively slow. There are two principal ways of “speeding up” the hydration reactions: finer grinding of the Portland cement and increasing its $C_3S$ and $C_3A$ contents.

### 3.2.1.5 Fineness

The fineness of Portland cement has steadily increased over the past 50 years (Bentz et al., 2011). They found that the average fineness increased from about 330 $m^2/kg$ in the early 1950s to about 380–400 $m^2/kg$ in 2010. This has a number of consequences. Of course, the hydration rate increases, leading to a higher rate of strength gain at early ages, but at the cost of a lower ultimate strength. As well, the higher reaction rate leads to a faster rate of heat release, which can lead to a higher temperature buildup in large sections, and the possibility of thermal cracking. The higher cement fineness can also lead to more shrinkage cracking, and a reduced capacity for self-healing. Combined, these effects will lead to potentially less durable concrete.

### 3.2.1.6 $C_3A$ content

From the point of view of the cement producer, producing a Portland cement with $C_3A$ and $C_4AF$ contents of about 8% each is close to ideal. At these contents, the clinker output of the kiln is maximized, and the kiln requires relatively little attention from the operator. Also, in the burning zone, the raw meal is easily transformed to clinker, because there is a good balance between the silicate phases and the interstitial (aluminate) phases. However, when producing a low $C_3A$ clinker, the interstitial phase becomes too fluid in the burning zone of the kiln, requiring a reduction in the speed of rotation of the kiln, and hence a reduced output. The kiln also becomes more difficult to operate.

Conversely, from the point of view of the concrete producer, the $C_3A$ is highly undesirable, because of its negative effects on the rheology and durability of the concrete. $C_3A$ is the most reactive of the cement minerals and combines with the gypsum ($CaSO_4 \cdot 2H_2O$) to form ettringite ($6CaO \cdot Al_2O_3 \cdot 3SO_3 \cdot 32H_2O$), mostly in the form of needles. These ettringite needles decrease the workability; the higher the $C_3A$ content, and the higher the fineness, the greater the loss in workability.
The C₃A may also lead to cement–superplasticizer incompatibility in low w/b ratio concretes. Finally, a high C₃A content may lead to durability problems, because ettringite is not stable in the high pH of the interstitial water in the capillary pores. After a few hours of hydration, some of the ettringite will be transformed into monosulfoaluminate (3CaO ⋅ Al₂O₃ ⋅ CaSO₄ ⋅ 12H₂O). If the concrete is subsequently exposed to sulfates, both the ettringite and the monosulfoaluminate are unstable; this can cause durability problems such as delayed ettringite formation (DEF), sulfate attack, and reduced resistance to freeze–thaw cycles. This suggests that to produce durable high-performance concretes, cements with a C₃A content of about 6%, not too finely ground (300–350 m²/kg), should ideally be used.

3.2.2 Physical attack

As with chemical attack, the first line of defense against physical attack is also to use as low a w/b ratio as is practicable for any individual structure. There are no particular “materials” solutions to problems arising from wetting and drying cycles. For abrasion and wear issues (see Chapter 9, Section 9.2.5), these are governed mostly by the aggregate, which is almost always harder and stronger than the cement paste. Stronger aggregates and a better paste–aggregate bond can only delay wear and abrasion, but cannot completely eliminate it (Mindess and Aïtcin, 2014).

3.2.2.1 Thermal effects

Mature hardened cement paste contains an appreciable amount of water: evaporable water in the larger capillary pores, the volume of this water depending upon the original water/cement (w/c) ratio and the degree of hydration; water physically adsorbed on the surface of the hydrated cement paste; and chemically combined water (i.e., the water involved in the hydration reactions). As the hydrated cement is heated from room temperature, the evaporable water begins to be driven off, with most of it lost by the time that the temperature reaches about 105°C. This is accompanied by considerable shrinkage of the paste. On further heating, by about 215°C, the remaining capillary water and some of the chemically bound water are lost (Phan et al., 2001). Between 300°C and 400°C, microcracking due to differential thermal deformations of the various phases in hydrated Portland cement occurs around the Ca(OH)₂ (portlandite) crystals, and around the larger still unhydrated cement grains (Piasta, 1984). Dehydration of the portlandite crystals begins at about 400°C, and decomposition is complete at about 600°C. The hardened cement paste continues to lose water continuously as the temperature increases, and is completely dehydrated by about 850°C. This is accompanied by a large increase in total porosity and
permeability (Piasta et al., 1984; Lee et al., 2008; Liu et al., 2008). Some of the cement begins to melt above 1200°C, and by 1300–1400°C, it is essentially entirely melted.

This thermal degradation of the cement paste is essentially independent of the w/c ratio and the type of Portland cement. For extended exposure to very high temperatures (>1000°C), calcium aluminate or high-alumina cements should be used. Generally, the effects of high temperature are governed mostly by the aggregate properties; for long-term exposure to high temperatures, it would be best to use refractory aggregates (Table 3.3).

### 3.3 AGGREGATES

Generally, aggregates are harder and more durable than the matrix phase in concrete. However, there are some cases in which the aggregates either contribute directly to concrete deterioration or are susceptible to damage themselves under certain exposure conditions. Concrete deterioration mechanisms relating to aggregates have been reviewed in some detail by Alexander and Mindess (2005). They divide these mechanisms into two types: *intrinsic* mechanisms refer to processes that occur internally as a consequence of the nature of the constituent materials, and *extrinsic* mechanisms whereby concrete deteriorates from the actions of external agents. Some of the deterioration mechanisms relating to aggregates in concrete are given in Table 3.4 (taken from Alexander and Mindess, 2005), and are briefly discussed below.
### Table 3.4 Deterioration mechanisms relating to aggregates in concrete

<table>
<thead>
<tr>
<th>Physical mechanisms</th>
<th>Chemical mechanisms</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensional incompatibility</td>
<td>Alkali–aggregate reaction</td>
</tr>
<tr>
<td>• Thermal effects</td>
<td>Sulfides in aggregates</td>
</tr>
<tr>
<td>• Moisture effects</td>
<td>Thaumasite sulfate attack</td>
</tr>
<tr>
<td>Freeze–thaw</td>
<td>Acid attack</td>
</tr>
<tr>
<td>Surface wetting and drying (moisture cycles)</td>
<td>Alkali attack</td>
</tr>
<tr>
<td>Surface abrasion and erosion</td>
<td>Other aggressive chemicals, e.g., sulfates</td>
</tr>
</tbody>
</table>


### 3.3.1 Physical mechanisms

Since concrete is a composite material consisting of aggregate particles embedded in a binder, we must consider the possibility of dimensional incompatibility between these two phases, due to either thermal or moisture effects.

#### 3.3.1.1 Thermal effects

The coefficients of thermal expansion for aggregates and cements are quite different. For rocks (Alexander and Mindess, 2005), these range from about $1 \times 10^{-6}$ to $14 \times 10^{-6}/\degree C$; for cement, they typically range from about $11 \times 10^{-6}$ to $16 \times 10^{-6}/\degree C$. Siliceous minerals such as quartz ($12 \times 10^{-6}/\degree C$) have thermal coefficients about twice those of calcareous aggregates and certain granites (about $6 \times 10^{-6}/\degree C$). The coefficient of thermal expansion of concrete made with siliceous aggregates is thus significantly higher than that of concrete made with calcareous aggregates. Generally, this disparity in thermal expansion coefficients does not cause problems, but care should be taken when aggregates with low thermal coefficients are used in situations where fire resistance is required, or where there is cyclic freezing and thawing. In these cases, cracks may develop between the aggregate and the matrix, reducing the durability. It has been suggested (Callan, 1952) that the difference between the thermal coefficients of coarse aggregate and the mortar should not exceed $5 \times 10^{-6}/\degree C$.

The effect of temperature on aggregate properties becomes particularly important at high temperatures. Typical concrete aggregates are all thermally stable up to at least 350°C. At higher temperatures, their behaviors diverge. Aggregates that contain water will dehydrate, leading to a certain amount of shrinkage. The transformation of $\alpha$-quartz to $\beta$-quartz at 573°C is accompanied by a volume increase of about 5.7%. At temperatures above about 600°C, some calcareous aggregates (calcite, dolomite, magnesite) will dissociate to form CO$_2$ and an oxide. In the temperature range 1200–1300°C, some igneous rocks show degassing and expansion.
The temperatures at which aggregates melt also vary widely, from basalt at about 1060°C to quartzite at about 1700°C. Thus, the thermal behavior of concrete is very sensitive to the precise nature of the aggregate(s) used, particularly since aggregates make up about 70% of the volume of high-performance concretes.

The coefficient of thermal expansion of cement paste is strongly dependent upon its moisture content. On the other hand, the thermal coefficients of most rocks depend on the temperature, with the coefficients increasing significantly as the temperature increases. Thus, at lower temperatures, the coefficient of thermal expansion of cement paste is higher than that of most rocks; at high temperatures, the reverse is true. What is clear is that as the temperature increases from ambient to over 1200°C, the differences in thermal coefficients of the cement and the aggregates, and the other volume changes that take place, will lead to considerable cracking (both micro and macro cracks) in the concrete. Of course, nothing can be done about the differences in thermal expansion coefficients between cement and aggregate.

3.3.1.2 Moisture effects

In concrete mix design, it is assumed that the aggregates are dimensionally stable. However, this is not always the case; there are aggregates that exhibit shrinking and swelling behavior on drying and wetting, and this can lead to very high values of concrete drying shrinkage, typically in excess of $1000 \times 10^{-6}$. This may lead to excessive deflections and cracking in structural elements. This phenomenon is well known for lightweight aggregates, but is also found with certain basic igneous rocks, some metamorphosed shales and slates, certain mudstones and sandstones, and other rock types as well. If these rocks are weathered, they should be considered as highly shrinkable, unless they are proven to be otherwise, either by tests or by a satisfactory service record. The best predictor of aggregates causing excess concrete shrinkage is aggregate absorption, with values above 0.8% indicating a much increased risk of excess drying shrinkage (Edwards, 1966).

3.3.1.3 Freezing and thawing

Certain types of aggregate may also suffer freeze–thaw damage; such aggregates typically have a total pore volume (or total absorption) greater than 1.5%. The rocks most susceptible to this type of damage are those with fine pores, a high total porosity, and a low permeability; this can occur in some cherts and shales.

3.3.1.4 Abrasion and erosion

The abrasion resistance of aggregates themselves is of little practical importance for most concrete construction. The exception is for those concretes
exposed to surface wear, as in pavements, airport runways, dam spillways, concrete canals carrying silt or gravel, or abrasion-resistant floor toppings. Abrasion of the aggregate in concrete can occur only if the aggregate particles become exposed due to wearing away of the original concrete surface. In all cases, it is best to use aggregates that are hard, strong, and free of soft or friable particles; a low w/c ratio, high strength concrete is also useful. However, whatever the case, it is more useful to assess the concrete itself for its wear resistance since this takes into account both the aggregate properties and the paste–aggregate bond.

The type of abrasive wear does impose somewhat different aggregate requirements. For erosion due to suspended solids in flowing water, larger aggregate particles work better, while for concrete subjected to cavitation, smaller (<20 mm) particles are preferred. For abrasion, aggregate behavior becomes increasingly important as strengths fall further and further below about 40 MPa.

There are no very good tests for the abrasion resistance of aggregates. Probably, the most common test is the Los Angeles abrasion test (ASTM C131 and C535), which involves ball milling a sample of aggregate and measuring the mass loss passing a 1.70-mm sieve. This test, however, does not correlate well with concrete behavior in the field; it is a better measure of the tendency of aggregates to break down during handling.

3.3.1.5 Maximum aggregate size

Although the maximum size of the coarse aggregate is not in itself a durability problem, it should be noted that there has been a tendency to go to smaller maximum aggregate sizes, to improve the handling, placing, and finishing of the concrete. This is particularly true for self-consolidating concrete and for some pumped concretes. This tends to lead to a higher water demand in the concrete and to higher potential shrinkage. If this shrinkage is not properly taken into account, excessive shrinkage cracking and a consequent loss in durability may occur.

3.3.1.6 Recycled concrete aggregates

There is increasing interest in using recycled concrete as coarse aggregate for new concrete construction. This interest derives from economic considerations and from a desire to make the concrete industry more sustainable. Note that the discussion here is limited to coarse aggregate; the fine aggregate resulting from the demolition of building wastes contains some old cement paste and mortar, which will in turn lead to problems with strength, and increases in the drying shrinkage and creep (Alexander and Mindess, 2005). Indeed, there is a RILEM report (Hansen, 1990) that recommends that all material less than 2 mm not be used.
Inevitably, even properly recycled concrete will contain traces of dirt, metals, glass, plastic, and so on. Thus, apart from generally leading to lower strengths in concrete made with recycled concrete aggregate, there is a greater likelihood of durability problems because of these impurities. Such aggregates should be used with caution in concretes that are to be subjected to severe environmental exposures.

### 3.3.2 Chemical mechanisms

Chemical durability problems may result from interactions between certain types of aggregates and the hydrated cement paste. The most common of these is AAR (see Chapter 9) but adverse reactions involving sulfates also occur. Aggregate sources may also occasionally contain organic or inorganic materials, which affect the setting of the fresh concrete.

Most aggregates, particularly siliceous ones, are immune from acid attack, but calcareous aggregates will be attacked by acids. This does not generally pose a problem, however, and it may be beneficial in helping to reduce the rate or severity of acid attack on the concrete. On the other hand, siliceous aggregates may be susceptible to attack by strong alkalis, as in the AARs discussed in Chapter 9.

However, it should be emphasized that attack by acids, alkalis, or other aggressive chemical agents are much more likely to damage the cementitious phases than the aggregate. Dense, sound aggregates almost always are sufficiently resistant to chemical attack, particularly when embedded in concrete.

### 3.4 WATER

In general, any water that is drinkable may be used to make concrete. However, some waters that are not drinkable may also be used satisfactorily. Excessive impurities may cause durability problems, and so limits on impurities are given in Table 3.5 (adapted from data in Mindess et al. [2003] and Kosmatka et al. [2002]).

As a further check on the quality of the mixing water, compressive strength of cubes or cylinders tested at 7 days should be at least 90% of the strength of control specimens cast with potable water. In some jurisdictions, such as South Africa, a value of 85% has been adopted.

Seawater contains, on average, about 34,000 ppm dissolved salts, mostly sodium chloride, plus magnesium chloride, magnesium sulfate, calcium sulfate, and calcium chloride. When used to produce plain (unreinforced) concrete, a strength loss of 10–20% can be expected. It should not be used for reinforced concrete, and particularly not for prestressed concrete, as it increases considerably the risk of corrosion of the steel. See Chapter 9 (Section 9.2.6) for a more detailed discussion of the effects of seawater.
3.5 ADMIXTURES

Most modern concretes contain some combination of both SCMs and chemical admixtures, as described in detail in Aïtcin and Flatt (2016). However, while these materials may not always lead to the desired outcomes, in themselves they do not cause durability problems. An improper choice of admixtures may lead to rheology problems, alterations of the time of set, delays in the rate of strength gain, and so on, but these are not durability problems per se.

<table>
<thead>
<tr>
<th><strong>Impurity</strong></th>
<th><strong>Maximum concentrations, ppm (mg/L)</strong></th>
<th><strong>Remarks</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Dissolved solids</td>
<td>2000</td>
<td>If alkali carbonate or bicarbonate is present</td>
</tr>
<tr>
<td>Suspended solids (turbidity)</td>
<td>2000</td>
<td>Silt, clay, organic matter</td>
</tr>
<tr>
<td>Algae</td>
<td>1000</td>
<td>Entrain air</td>
</tr>
<tr>
<td>$\text{Na}_2\text{CO}_3 + \text{NaHCO}_3$</td>
<td>2000</td>
<td>Affect setting</td>
</tr>
<tr>
<td>$\text{Ca(HCO}_3\text{)}_2; \text{Mg(HCO}_3\text{)}_2$</td>
<td>400</td>
<td>Affect setting and may reduce strength</td>
</tr>
<tr>
<td>Inorganic acids</td>
<td>10,000</td>
<td>pH not less than 3.0</td>
</tr>
<tr>
<td>Sugar</td>
<td>500</td>
<td>Affects setting behavior</td>
</tr>
<tr>
<td>$\text{Na}_2\text{SO}_4; \text{MgCl}_2; \text{MgSO}_4; \text{CaCl}_2$</td>
<td>10,000</td>
<td>May decrease setting time; reduce ultimate strength</td>
</tr>
<tr>
<td>$\text{NaCl}$ (in reinforced concrete)</td>
<td>1500</td>
<td>May decrease setting time; may increase the risk of corrosion in reinforced concrete</td>
</tr>
<tr>
<td>$\text{NaOH}$</td>
<td>500</td>
<td>May reduce strength</td>
</tr>
<tr>
<td>Sodium sulfide</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Iron salts</td>
<td>40,000</td>
<td></td>
</tr>
<tr>
<td>Salts of Zn, Ca, Pb, Mn, Sn</td>
<td>500</td>
<td>Retard set</td>
</tr>
<tr>
<td>Phosphates, arsenates, borates</td>
<td>500</td>
<td>Retard set</td>
</tr>
<tr>
<td>Chlorides (by mass of cement), as per ACI 318 Building Code (2002)</td>
<td>0.06%</td>
<td>Prestressed concrete</td>
</tr>
<tr>
<td></td>
<td>0.15%</td>
<td>Reinforced concrete exposed to chlorides in service</td>
</tr>
<tr>
<td></td>
<td>1.0%</td>
<td>Reinforced concrete kept dry in service</td>
</tr>
<tr>
<td></td>
<td>0.3%</td>
<td>Other reinforced concrete</td>
</tr>
</tbody>
</table>

3.5.1 Supplementary cementing materials

At one time, the two main SCMs were fly ash (whose use goes back to the 1930s) and GGBFS (whose use in Europe goes back to the early 1900s). Originally, these were used primarily for economic reasons, as they were considerably cheaper than Portland cement. However, it was soon recognized that they could considerably improve the long-term properties of concrete through their pozzolanic reaction with the lime liberated by the hydration of Portland cement, though sometimes at the cost of a rather slower rate of strength gain. Of course, depending upon their source, these materials are sometimes of variable quality and precise chemical composition. However, in and of themselves, they do not cause durability problems for concrete.

Over time, driven by economic considerations (as fly ash and slag became more expensive), a large number of other materials (either naturally occurring or “waste” materials as byproducts of other industries) with pozzolanic properties have come into use. The most common of these are silica fume and metakaolin; various other sources of pozzolanic materials, such as rice husk ash, are also sometimes exploited in specific local markets. While the long-term effects of these materials in concrete are still unknown, there is as yet no evidence to suggest that they themselves lead to durability problems. The same may be said of the much less reactive filler materials, such as ground limestone, which are now also commonly added to Portland cement in relatively small quantities (<10% in North America, but up to 20% or even more in Europe).

3.5.2 Chemical admixtures

The picture is somewhat less clear when we look at chemical admixtures. These now include air-entraining admixtures, water reducers, superplasticizers, corrosion inhibitors, rheology modifiers, set-controlling admixtures (retarders or accelerators), water-proofing agents, internal curing materials, and so on. Again, these materials by themselves are not known to create durability problems. However, modern high-performance concretes, which almost always contain a variety of both mineral and chemical admixtures, are becoming increasingly complex. Inevitably, there are chemical interactions among the Portland cement and the various admixtures. This often makes it difficult to find (empirically) the right combination of binder materials and admixtures for any particular project. It is still unclear whether these combinations of materials will be durable in the long run, particularly in very aggressive environments.

3.6 FIBERS

Over the last 50 years, there has been a steady increase in the use of fibers in concrete, for both cast-in-place and shotcrete applications. The fibers are
not added to increase strength, though modest improvements in strength may occur. Rather, the fibers are added to help control cracking, and more particularly to impart some postcracking ductility to the concrete (Bentur and Mindess, 2007). A number of different fiber types are currently being used, including steel, polypropylene, polyethylene, carbon, aramid, glass, cellulose, and natural organic fibers such as sisal and jute. (Asbestos fibers were once common, but are not now used because of the health risks associated with the production and handling of these fibers.) Some of the fibers such as carbon, aramid, polyethylene, and polypropylene are completely inert in concrete. Others may react either with the cement or the environment: steel may rust, natural organic fibers may deteriorate over time in damp or alkaline environments, and glass may be attacked by the alkaline environment in concrete.

The fibers themselves do not have any direct effect on the durability of the concrete. However, those fibers that do interact with the surrounding concrete may lose their effectiveness over time.

**REFERENCES**


CSA A23.1-14, 2014, *Concrete Materials and Methods of Concrete Construction*, CSA Group, Toronto, Canada.


