P • A • R • T • 3

FUNDAMENTALS OF FOUNDATION CONSTRUCTION AND DESIGN

SECTION 3
CONCRETE

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Concrete is one of the basic construction materials; it finds a place in almost all structures. Even in such structures as steel bridges, the deck is quite often made of concrete. Concrete is the preferred and most widely used material for foundation construction. Even if the superstructure is made of steel or wood, the foundation is usually made of concrete. In the case of slab on grade floors, whether industrial, commercial, or residential, concrete is the preferred material.

This section deals with some of the fundamental aspects of concrete. Only the basic information considered necessary for the design and construction engineer is presented. The reader can refer to the literature for more details and in-depth information on a particular aspect. This section deals only with plain concrete. Reinforced concrete is discussed in Part 4.

3.1 CONSTITUENT MATERIALS

Concrete is a composite material made of portland cement (often simply called cement), aggregates, and water. In most cases, additional constituents, called admixtures, are used to improve the properties of fresh and hardened concrete. For example, water-reducing admixtures are often used to improve the workability of fresh concrete without increasing its water content, thus maintaining the strength and durability characteristics of the hardened concrete. The admixtures can be classified broadly as chemical and mineral admixtures.

This section presents basic information with regard to the various constituent materials used in concrete. They are grouped as (1) cement, (2) aggregates, (3) water and water-reducing admixtures, (4) chemical admixtures, and (5) mineral admixtures. Even though water-reducing admixtures are chemical admixtures, they are discussed together with water because of their direct impact on the quantity of water used in the mix and their widespread use in practice.

3.1.1 Cement

Cement, which is the binding ingredient of concrete, is produced by combining lime, silica, and alumina. A small amount of gypsum is added to control the setting time of the cement. Portland cement was first patented by Joseph Aspdin of England in 1824. David Saylor of Coplay, Pennsylvania, was the first to produce portland cement in the United States in 1871. He used vertical kilns that were similar to the ones used for burning lime. The rotary kiln was introduced in 1899. In the 1990s cement production in the United States was in the range of 800 million tons (725 million tonnes); worldwide it reached 5 billion tons (4.5×10^9 tonnes).

Manufacture of Cement

The raw materials for portland cement consist primarily of limestone or some other lime-containing material such as marl, chalk, or shells, and of clay or shale or some other clayey material such as ash or slag. Sometimes other ingredients, such as high-calcium limestone, sandstone, and iron ore, are added to control the chemical composition of the final product. The manufacturing process can be briefly described as follows.

The raw materials are ground into impalpable powder and thoroughly mixed. In the dry process, blending and grinding operations are done in the dry form, and the mixing is primarily accomplished during the grinding phase. In the wet process, water is used to form a slurry. The slurry is often mixed in large vats to obtain a thorough mixing, even though the ingredients have already been mixed during the grinding process. The wet process, which requires about 15% more energy than the dry process, is often chosen for environmental and technological reasons. Continuous quality control measures are used to ascertain the proper chemical composition of the raw material so that the chemical ingredients of the final product will be within the limits specified.

In most cases the slurry is fed into the upper end of a slightly inclined rotary kiln. In some instances part of the water is removed from the slurry before feeding it into the kiln. The length and the diameter of the kilns vary between 60 and 500 ft (18 and 150 m) and between 6 and 15 ft (1.8 and 4.5 in), respectively. The kilns, set at an inclination of about 0.5 in/ft (40 mm/m), rotate between 30 and 90 revolutions per hour, moving the material toward the lower (discharge) end. Heating is usually done by using powdered coal and air. In some instances oil or gas is used instead of coal. The temperature varies along the kiln, reaching a maximum in the range of 2300 to 3450°F (1250 to 1900°C).

As the mix passes through the kiln, various reactions take place, including (1) evaporation of free water, (2) dehydroxylation of clay minerals, (3) crystallization of the products of clay mineral dehydroxylation, (4) decomposition of $CaCO_3$, (5) reaction between $CaCO_3$ (CaO) and aluminosilicates, and (6) liquefaction and formation of cement compounds. The temperature variations are controlled in such a way as to keep the compounds in the molten stage to a minimum. The molten liquid agglomerates into nodules. The nodules, ranging in size from 0.125 to 2 in (3 to 50 mm), are called cement clinkers. These clinkers are dropped off from the kiln.

The clinkers are cooled and ground to a fine powder. About 3 to 5% of gypsum ($CaSO_4 \cdot 2H_2O$) is added during the grinding process to control the setting time of the cement. Addition of gypsum retards the hydration of cement, or increases its setting time. After grinding, the cement is stored in silos.

In the United States, cement can be bought in bulk or in bags containing 94 lb (42.5 kg). It is common to designate concrete mixes as 5, 6, or 7 bag mix, and hence it is useful to remember the weight of the cement in a bag.

Composition

Compounds of four oxides containing lime, silica, alumina, and iron constitute about 9S% of the portland cement clinkers. The other 5% could include magnesia, sodium and potassium oxide, titania, sulfur, phosphorous, and manganese oxide. The major components, namely, tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and tetracalcium aluminoferrate (C_4AF), play important roles in the rate of strength development, the heat of hydration, and the ultimate cementing value. For example, the early strength of hydrated portland cement is higher if the percentage of tricalcium silicate is higher, whereas long-term strengths will be higher with higher percentages of dicalcium silicate.

Types

Various types of cement are produced to suit the various applications. The American Society for Testing and Materials (ASTM) recognizes the following five main types:

Type I For general use in construction

Type II For use that requires moderate heat of hydration and exposure to moderate sulfate action

Type III For use where high early strength is needed

Type IV For use that requires low heat of hydration

Type V For use that requires high sulfate resistance

Types 1,11, and III can be obtained with air-entraining agents. These are then designated types IA, IIA, and IllA. Some standard blended portland cements that are available are called portland blast-furnace slag cement and portland pozzolan cement.

Typical composition values for the various compounds of the five cement types are shown in Table 3.1. These numbers are mean values, and there is a specified minimum and maximum for each compound.

Fineness

The term fineness refers to the average size of the cement particles. The fineness of the cement determines the rate of reaction because finer particles have more surface area and, hence, generate more reactivity when water is added. Type III high-early-strength cement has more fine particles than type I cement. Finer cement bleeds less than coarser cement. In addition, finer cement contributes to better workability and produces less autoclave expansion. But the finer cement is more

TABLE 3.1 Percentage Composition of Portland Cements

		Component, %							
Type of cement		C_3S	C_2S	C_3A	C_4AF	CaSO4	CaO	MgO	General characteristics
I	Normal	49	25	12	8	2.9	0.8	2.4	All-purpose cement
II	Modified	45	29	6	12	2.8	0.6	3.0	Comparative low heat liberation; used in large structures
III	High early strength	56	15	12	8	3.9	1.4	2.6	High strength in 3 days
IV	Low heat	30	46	5	13	2.9	0.3	2.7	Used in mass concrete dams
V	Sulfate resisting	43	36	4	12	2.7	0.4	1.6	Used in sewers and structures exposed to sulfates

expensive to produce, and if the particles are overly fine, they could lead to increased shrinkage, higher water demand, strong reaction with alkali-reactive aggregates, and poor stability.

The particle size usually varies from 1 to 200 ~m. Fineness of cement can be expressed using the Blame specific surface area. The cement is considered overly fine if the Blame specific surface area is greater than 2440 ft^2/lb (5000 cm^2/g). The specific surface areas, measured using the air permeability method, vary from 1220 to 1760 ft^2/lb (2500 to 3600 cm^2/g) and from 1760 to 2200 ft^2/lb (3600 to 4500 cm^2/g) for type I and III cements, respectively.

Cements with small particle sizes, known as microcements, are available for special purposes such as grouting. The fine particles facilitate the grouting of soils containing small pore sizes.

Testing Methods

Typically cement is tested at periodic intervals for chemical composition and certain physical properties to satisfy quality control requirements. Special tests may be needed for particular cases such as the determination of compatibility with certain admixtures. ASTM specifications exist for most of the tests. The following list contains commonly used tests and the corresponding ASTM specifications.

- *Chemical compositions:* Chemical analysis of portland cement (ASTM C14).
- *Fineness:* Sieve analysis (ASTM C 184), Wagner turbidimeter method (ASTM C115), Blame air-permeability method (ASTM C204).
- *Normal consistency* (ASTM C187): This test is used to determine the amount of water to be used in making samples to be tested for soundness, time of setting, and strength. The amount of water needed for normal consistency varies between 22 and 28% for portland cement.
- *Time of setting:* Time of setting includes both initial and final setting times. The time needed for the paste to start stiffening is called the initial setting time, whereas the final setting time represents the end of plasticity. The ASTM standards are C191 for Vicat apparatus and C266 for Gillmore needles. There is also a term called false set. This represents a stage in which the mix that is stiff can be remixed without adding water to restore plasticity. ASTM specifications require that initial setting time be at least 45 min by Vicat apparatus and 60 min by Gillmore needles. The corresponding final setting times are 8 and 10 h, respectively.
- *Soundness:* The test for soundness involves the measurement of the expansion of hardened cement paste. One of the popular tests is the autoclave test (ASTM C151).
- *Strength:* Strength is probably the most important property sought after. Both the magnitude of strength and the rate of strength development are important. The basic strength tests are compression (ASTM, C 109), tension (ASTM C190), and flexure (ASTM C348 or C349).
- *Heat of hydration (ASTM C186):* This property, which provides an indication of the amount of heat generated during hydration, is extremely important for most concrete construction such as dams, thick slabs, and pile caps.
- *Other tests:* Other tests that are used infrequently include shrinkage or expansion tests and tests for measuring specific gravity, alkali reactivity, sulfate resistance, air entrainment, bleeding, and efflorescence.

3.1.2 Aggregates

Aggregates are much less expensive than the cementing material and could constitute up to 90% of the volume of the concrete. They are typically considered as inert filler material, even though some aggregates do react minimally with the cement paste. Aggregates can be classified as coarse or fine based on their size; normal weight, lightweight, or heavyweight based on their bulk densities; and natural mineral or synthetic based on their type of production. Aggregate characteristics that affect the final product (namely, the concrete) include porosity, grading and size distribution, shape, surface texture, crushing strength, elastic modulus, moisture absorption, and type and amount of deleterious substances present. The primary concerns are the quality of the aggregate and its grading.

Coarse Aggregates

If the particle size is greater than 0.25 in (6 mm), the aggregates are classified as coarse aggregates. A list of the common coarse aggregate types follows.

- *Natural gravel:* About 50% of the coarse aggregates used in the United States consists of gravel. Natural cobbles and gravel are produced by weathering action. They are usually round in shape with a smooth surface. Hence these aggregates provide better workability. When they are made of siliceous rocks and uncontaminated with clay or silt, they make strong and durable aggregates. Some of the very high strength concretes with a compressive strength in the range of 20,000 psi (140 MPa) are made using gravel aggregates.
- *Natural crushed stone:* Crushed stone aggregate is produced by crushing the rocks and grading them. About two-thirds of the crushed aggregate in the United States is made of carbonate rocks (limestone, dolomite). The remainder is made of sandstone, granite, diorite, gabbro, and basalt. Carbonate rocks are softer than siliceous sedimentary rocks. The characteristics such as strength, porosity, and durability could vary considerably. Hence care should be taken to avoid the rocks that are not suitable for aggregates. Crushed stone aggregates are typically angular in shape and thus less workable than gravel under similar conditions.
- *Lightweight aggregates:* Aggregates with a bulk density of less than 70 lb/ft^3 (1120 kg/m^3) are normally considered lightweight aggregates. However, there is a whole spectrum of lightweight aggregates weighing from 5 to 55 lb/ft^3 (80 to 900 kg/m^3), as shown in Fig. 3.1.[1] Natural lightweight aggregates are made by processing naturally occurring lightweight rock formations such as pumice, scoria, and tuff. But most of the lightweight aggregates used for structural concrete are made by expanding or thermally treating a variety of materials such as clay, shale, slate, diatomite, perlite, or vermiculite. Industrial by-products such as blast-furnace slag and fly ash are also used to manufacture lightweight aggregates. The lightweight aggregates can be grouped into

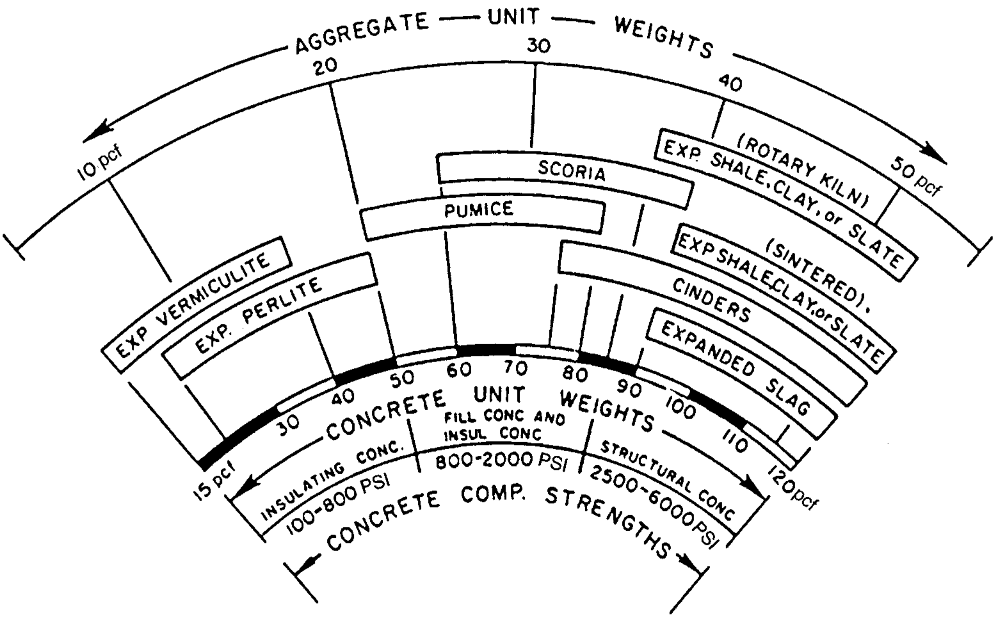


FIGURE 3.1 Lightweight aggregate spectrum. (*From Litvin and Fiorato.*[1])

three categories based on their end use: structural concrete, production of masonry units, and insulating concrete.

- *Heavyweight aggregates:* The bulk density of heavyweight aggregates ranges from 145 to 280 lb/ft^3 (2320 to 4480 kg/m^3). The primary use of these aggregates is for nuclear radiation shields. Natural rocks suitable for heavyweight aggregates may contain barium minerals, iron ores, or titanium ores. The aggregate types are witherite ($BaCO_3$), barite, ($BaSO_4$), magnetite (Fe_3O_4), hematite (Fe_2O_3), hydrous iron ores, ilinenite ($FeTiO_3$), ferrophosphorus (Fe_3P, Fe_2P, FeP), and steel aggregate (Fe). Ferrophosphorus aggregates when used with portland cement might generate flammable (and possibly) toxic gases and, hence, should be used with caution. Boron and hydrogen are very effective for neutron attenuation, and hence the aggregates containing these compounds are more useful for shielding. Heavyweight aggregates tend to produce more segregation. Sometimes preplaced-aggregate techniques are employed to make heavyweight concrete in order to avoid segregation.
- *Aggregates made from recycled concrete and other waste products:* Rubble from demolished buildings could be used as aggregates. The best source is the recycled concrete that contains less hydrolized cement paste and gypsum. Concrete made with these aggregates has about two-thirds of the compressive strength of concrete made with stone aggregates. One of the major projects built using recycled aggregate was a repavement project in Michigan. The work was carried out by the Michigan Department of Transportation and involved about 125,000 yd^3 (96,000 m^3) of concrete made using recycled concrete pavements.

Investigations have also been conducted for utilizing aggregates made with municipal wastes and incinerator ashes. The results are not very positive. The presence of glass particles tends to reduce workability and long-term durability. Metals such as aluminum react with alkaline materials in concrete and expand, causing deterioration. Organic wastes and paper interfere with the setting of concrete.

Fine Aggregates

Fine aggregates are usually made of natural or crushed sand. Their size ranges from 0.25 to 0.01 in (6 to 0.25 mm). The fine aggregates should be free of organic materials, clay, or other deleterious materials. Minimum particle sizes should be not less than 0.01 in (0.25 mm) because these fine particles tend to increase the water demand and reduce strength. For all-lightweight concrete, light-

TABLE 3.2 Grading Requirements for Coarse Aggregates

Size number	Nominal size (sieves with square openings)		Amounts finer than laboratory sieve (square openings), wt %				
			100 mm (4 in)	90 mm (3½ in)	75 mm (3 in)	63 mm (2½ in)	50 mm (2 in)
1	90–37.5 mm	(3½–1½ in)	100	90–100	—	25–60	—
2	63–37.5 mm	(2½–1½ in)	—	—	100	90–100	35–70
3	50–25.0mm	(2–1 in)	—	—	—	100	90–100
357	50–4.75 mm	(2 in–No. 4)	—	—	—	100	95–100
4	37.5–19.0mm	(1½–¾ in)	—	—	—	—	100
467	37.5–4.75 mm	(1½in–No. 4)	—	—	—	—	100
5	25.0–12.5 mm	(1–½ in)	—	—	—	—	—
56	25.0–9.5 mm	(l–⅜ in)	—	—	—	—	—
57	25.0–4.75 mm	(1 in–No. 4)	—	—	—	—	—
6	19.0–9.5 mm	(¾–⅜ in)	—	—	—	—	—
67	19.0–4.75 mm	(¾ in–No. 4)	—	—	—	—	—
7	12.5–4.75 mm	(½ in–No. 4)	—	—	—	—	—
8	9.5–2.36 mm	(⅜ in–No. 4)	—	—	—	—	—

Source: From *Annual Book of ASTM Standards*.[2]

weight sand made of expanded shale or clay materials is used. Similarly, for heavyweight concrete, regular sand is replaced with fine steel shot, iron ore, or other high-density material.

Production of Aggregates

The natural sand and gravel aggregates are the most economical aggregates to produce. They only need cleaning and grading. Cleaning can be accomplished by either screening or washing. Washing is more effective for removing clay and silt particles, but the process is more expensive.

When aggregates are produced by crushing rock, the type of crushing equipment depends on the type of rock to be crushed. The major sequential operations are crushing, cleaning, separation of various sizes, and blending of various sizes to obtain the required gradation.

In the case of synthetic lightweight aggregates, an extra step of expanding is needed. Generally, crushed or ground and pelletized raw material (clay, slate, or shale) is heated to about 2012°F (1100°C). At this temperature, the material expands because of the entrapped gases. The expanded material is processed to obtain the required grading. Lightweight aggregates are typically more expensive because of the extra effort and energy needed to produce them.

Size and Grading

The maximum size of the aggregates depends on the type of structure for which the concrete is to be used. For example, in dams and large foundations, maximum sizes greater than 2 in (50 mm) are very common, whereas for beams and columns containing extensive reinforcement, the maximum size is often restricted to 0.75 in (19 mm). The maximum size of the aggregate should be smaller than one-fifth of the narrowest dimension of the form and three-fourths of the maximum clear distance between reinforcements. For high-strength concrete the maximum size is generally limited to 0.75 in (19 mm).

As mentioned earlier, aggregates constitute a combination of different-size particles. The distribution of these particle sizes is called grading. The aim is to attain a grading (size distribution) that will produce a concrete with the required strength using minimum cement. At the same time, the mix should be workable in the plastic stage. Theoretically the best combination of particle sizes is the one that produces minimum voids and minimum total surface area. ASTM has established grading requirements for both coarse and fine aggregates so as to obtain workable concrete mixtures. Tables 3.2 and 3.3 present these requirements for normal-weight concrete. Similar requirements for

TABLE 3.2 (*Cont.*)

Amounts finer than laboratory sieve (square openings), wt %							
37.5 mm (1½ in)	25.0mm (1 in)	19.0mm (¾ in)	12.5 mm (½ in)	9.5 mm (⅜ in)	4.75 mm (No. 4)	2.36mm (No. 8)	1.18mm (No. 16)
0–15	—	0–5	—	—	—	—	—
0–15	—	0–5	—	—	—		
35–70	0–15	—	0–5	—	—	—	—
—	35–70	—	10–30	—	0–5	—	—
90–100	20–55	0–15	—	0–5	—	—	—
95–100	—	35–70	—	10–30	0–5	—	—
100	90–100	20–55	0–10	0–5	—	—	—
100	90–100	40–85	10–40	0–15	0–5	—	—
100	95–100	—	25–60	—	0–10	0–5	—
—	100	90–100	20–55	0–15	0–5	—	—
—	100	90–100	—	20–55	0–10	0–5	
—	—	100	90–100	40–70	0–15	0–5	—
—	—	—	100	85–100	10–30	0–10	0–5

TABLE 3.3 Grading Requirements for Fine Aggregates

Sieve (Specification E11)	Percent passing
9.5mm (3/8 in)	100
4.75 mm (No. 4)	95–100
2.36mm (No. 8)	80–100
1.18 mm (No.16)	50–85
600 µm (No. 30)	25–60
300 µm (No. 50)	10–30
150 µm (No. 100)	2–10

Source: From *Annual Book of ASTM Standards.*[2]

lightweight (C320) and heavyweight (C637) concrete can be found in the *Annual Book of ASTM Standards*, vol. 04.02.

The grading requirement for fine aggregate is straightforward (Table 3.3). The distribution of particles has to follow a certain pattern, with particles of different sizes limited to a certain range. For the coarse aggregate, first a maximum size should be chosen. The size distribution of the other particles depends on this maximum size. For example, if the maximum size is 1.0 in (25 mm), size numbers 5, 56, or 57 can be chosen (Table 3.2). If size 5 is chosen, 100% of the aggregates should pass through a 1.5-in (37.5-mm) sieve. Reading horizontally, 90–100, 20–55, 0–10, and 0–5% of material should pass through 1-, 0.75-, 0.5-, and 0.375-in (25-, 19-, 12.5-, and 9.5- mm) sieves, respectively. The sieve size represents the dimensions of the square openings. For example, a 1-in (25-mm) sieve has a mesh with a 1-in (25-mm) square opening. The reader should refer to ASTM C33 for other restrictions, such as fineness modulus.

Most of the time the aggregates have a uniform gradation. However, gap-graded aggregates are shown to provide better strength characteristics. In gap grading, the sizes of aggregates do not decrease uniformly. Certain size segments are left out in order to obtain a better packing and, hence, a more efficient utilization of cement.

Other Aggregate Characteristics

Grading is the most influential parameter that affects the behavior of concrete. Other factors influencing the quality of concrete are density and apparent specific gravity, absorption and surface moisture, crushing strength, elastic modulus, abrasion resistance, soundness, shape and surface texture, and the presence of deleterious substances. The significance of some of these parameters is listed in Table 3.4. The table also presents the ASTM test methods that can be used for the evaluation of these characteristics.

TABLE 3.4 Aggregate Characteristics and Their Significance

Characteristic	Effect on concrete	ASTM test method
Gradation	Economy, workability, long-term performance	Cl 17, C136
Bulk unit weight	Mix proportioning calculations	C29
Absorption and surface moisture	Quality control of concrete	C70, C 127, C 128, C566
Abrasion resistance	Wear resistance of floors and pavements	C131, C295, C535
Resistance to freezing and thawing	Surface scaling, durability	C295, C666, C682
Particle shape and surface texture	Workability of fresh concrete	C295, C398
Elastic modulus	Elastic modulus of concrete	C469

Selection of Aggregates

The ideal aggregate consists of particles that are strong, durable, clean, do not flake when wetted and dried, have somewhat rough surface texture, and contain no constituents that interfere with cement hydration. The grading of these particles should be done so that the concrete has good workability and the cement is utilized to its maximum efficiency. It is seldom possible to obtain ideal aggregates because of economical constraints.

In practical situations, aggregate selection should be made based on field conditions and end use. For example, a foundation built on sulfate-containing soil should not have aggregates vulnerable to sulfate attack. If the structure is going to be exposed to freezing and thawing cycles, durability of the aggregate plays a major role. The gradation requirements should be chosen not only for the maximum utilization of cement, but also to produce a workable concrete mixture. The availability of aggregates in close proximity plays an important economical role. If past records regarding the performance of the potential aggregate source based on existing structures are not available, suitable tests should be run to evaluate their properties. It should be noted that some of the disadvantages of the aggregate could be overcome by making minor modifications using admixtures. For example, if workability is the problem because of angular surfaces, water-reducing admixtures could be used to overcome this problem. The only constraint is the economy. The additional cost of the admixture should be justifiable.

3.1.3 Water and Water-Reducing Admixtures

Water is one of the primary ingredients of concrete. Water used during the mixing, called mixing water, performs the basic functions of hydration and lubrication. It also provides space for expanding hydration products. The lubrication action influences the workability of fresh concrete for placing, compaction, and finishing operations. The hydration, or the chemical reaction between water and cement, results in the hardening of concrete. Water is also needed for curing and sometimes for washing the aggregates. The amount of water needed for adequate workability is always greater than that needed for hydration. In addition, complete hydration of cement does not produce the highest strength. Therefore a number of admixtures were developed to improve the workability of concrete containing a limited amount of water. The most notable admixture is called high-range water-reducing admixture because of its very high efficiency. This section describes the requirements of water quality and the properties of water-reducing admixtures.

Water

Typically the water that is good for drinking is good for making concrete. Certain mineral waters that are potable may not be suitable for concrete. The water should be free of a particular taste, color, and odor, and should not foam or fizz when shaken. If in doubt, the water should be tested for suitability by evaluating the setting time of the cement paste, compressive strength, and durability. In most cases the setting time test alone may be sufficient.

The harmful contaminates not permitted in the water used for concrete are sugar, tannic acid, vegetable matter, oil, humic acid, alkali salts, free carbonic acid, sulfates, and water containing effluents from paint and fertilizer factories and sewage treatment plants. The following can be considered the general maximum limits for impurities:

Acidity	$0.1N$ NaOH; 2 mL maximum to neutralize 200-mL sample
Alkalinity	$0.1N$ HCl; 10 mL maximum to neutralize 200-mL sample; pH in the range of 6 to 9
Organic solids	$\ngtr 0.02\%$
Inorganic solids	$\ngtr 0.30\%$
Sulfuric anhydride	$\ngtr 0.04\%$
Sodium chloride	$\ngtr 0.10\%$
Turbidity	$\ngtr 2000$ ppm

Water containing more impurities than the limits mentioned, such as seawater, has been used successfully. However, special tests should be conducted for the particular application prior to approval. In most cases a strength reduction of 15% can be expected if the mixing water contains salts. Decreased durability and corrosion of reinforcement can also be anticipated.

The requirements for curing water are less stringent because it comes in contact with the concrete on the surfaces only and that for too short a duration. Nevertheless, water with excessive impurities should not be used because it could cause surface discoloration. Under the worst conditions curing water could cause surface deterioration.

With regard to the water used for washing the aggregates and for concrete mixing and placing equipment the primary concern is the deposit of minerals on aggregate particles and equipment. The water should be clean enough not to leave any deposit. Washing the aggregate is not usually recommended because the disposal of contaminated water presents a problem. It should be noted that the disposal of contaminated water into natural streams and drains is not permitted in the United States.

Water-Reducing Admixtures

The water–cement ratio is the most influential parameter controlling the strength. Typically, a lower water content results in higher strength. However, a certain amount of water is needed to obtain workability so that the fresh concrete can be placed in position and compacted. Water-reducing admixtures improve the workability, and thus workable concrete can be obtained without increasing the water content. These admixtures can be used either to improve workability for the same water content, as the reduction of water without losing workability results in higher compressive strength, or to reduce the cement content, maintaining workability and strength.

For example, consider a reference concrete with 500 lb/yd^3 (300 kg/m^3) cement and a water–cement ratio of 0.62. This concrete had a slump of 2 in (50 mm) and a 28-day compressive strength of 5.3 ksi (37 MPa). Addition of the admixture resulted in an increase of slump (workability) to 4 in (100 mm). The compressive strength was 5.4 ksi (38 MPa).

When the admixture was used to reduce the water–cement ratio to 0.56, maintaining a slump of 2 in (50 mm), the compressive strength increased to 6.8 ksi (46 MPa). If the slump and strength levels were kept at the reference levels of 2 in (50 mm) and 5.3 ksi (37 MPa), the cement content could be reduced to 450 lb/yd^3 (270 kg/m^3). The 10% reduction in cement provides not only economy but also other benefits such as low heat of hydration and reduced shrinkage.

The principal active ingredients in water-reducing admixtures are salts, modifications and derivatives of lignosulfonic acid, hydroxylated carboxylic acids, and polyhydroxy compounds. Typically these admixtures provide better dispersion of cement particles and, hence, could provide an increase in the early-age strengths. Larger amounts of admixtures could retard the setting time by preventing the flocculation of hydrated particles. Some commercial formulations may contain accelerating admixtures to overcome this effect. Admixtures derived from lignin products also tend to entrain considerable air. This effect is nullified by adding air-detraining agents. The period of effectiveness varies with the formulations. However, in all cases the improved workability will be at least partially lost as the cement starts to hydrate.

A special type of water-reducing admixture called high-range water-reducing admixture or superplasticizer was developed in the 1970s and is commonly used now. As the name implies, this admixture provides substantial improvement in the workability. A water reduction of 20 to 25% is possible, as compared to S to 10% for normal water-reducing admixtures. Introduction of the superplasticizer is also responsible for the use of high-volume fly ash and silica fume in concrete because these mineral admixtures (at high volume fractions) have a high water demand and could not be used without the aid of a superplasticizer.

Superplasticizers consist of long-chain, high-molecular-weight anionic surfactants with a large number of polar groups in the hydrocarbon chain. These compounds are adsorbed on the cement particles during the mixing and impart a strong negative charge, helping to lower the surface tension of the surrounding water. This results in a uniform distribution of the cement particles and increased fluidity. Because of the better dispersion of cement particles, concrete made with superplasticizers tends to have higher 1-, 3-, and 7-day strengths as compared to reference concrete having the same water–cement ratio. The negative effect of better cement distribution is a rapid loss of workability because of accelerated setting. Hence set-retarding admixtures are typically added to control the setting time.

The four basic types of superplasticizers available in the market are as follows:

1. Sulfonated melamine formaldehyde condensates
2. Sulfonated naphthalene formaldehyde condensates
3. Modified lignosulfonates
4. Sulfonic acid esters or other carbohydrate esters

Typical dosage of superplasticizers is in the range of 1 to 2.S%, even though dosages as high as 4% have been used successfully. One of the major problems encountered in using superplasticizers is the loss of workability with time. Some of the original versions lost their effectiveness in less than 1 h. The currently available formulations are effective for longer durations. The individual commercial product should be checked for its effective duration. Multiple dosages or retempering with superplasticizer was also found to be effective. It was found that the mix can be retempered three times without affecting the mechanical properties adversely.

The water-reducing admixtures are covered in ASTM C494. High-range water-reducing admixlures are called type F, and the regular admixtures are called type A water-reducing admixtures.

3.1.4 Chemical Admixtures

As mentioned earlier, the primary ingredients of concrete are cement, aggregates, and water. Any other ingredient added to concrete can be classified as an admixture. The functions performed by admixtures include improved workability, acceleration or retardation of setting time, control of strength development, improved freeze-thaw durability, and enhanced resistance to water permeation, frost action, thermal cracking, aggressive chemicals, and alkali-aggregate expansion. The admixtures are also used to improve economy and save energy. In some countries up to 80% of all concrete produced contains some kind of admixtures.

The admixtures can be broadly classified as chemical and mineral admixtures. The mineral admixtures are discussed in Sec. 3.l.5. The most frequently used chemical admixtures are (1) accelerators or retarders, (2) water reducers, and (3) air-entraining admixtures. The water-reducing admixtures were discussed in Sec. 3.l.3. This section deals with the other admixtures.

Accelerating Admixtures

Accelerating admixtures, or accelerators, classified as type C admixtures in ASTM C494, are used in concrete to reduce the time of setting or to enhance early strength development, or both. It should be noted that increased early-strength development could lead to a reduction in strength at later ages. In any case, improvement in long-term strength should not be expected. The chemical components used in accelerating admixtures include soluble chlorides, carbonates, silicates, fluorosilicates, hydroxides, bromides, and organic compounds.

The higher early strength can be used to achieve the following benefits in construction:

- Early finishing of surfaces
- Early removal of forms
- Early opening of construction for service
- More efficient plugging of leaks against hydraulic pressure
- Partial or complete compensation for effects of low temperature
- Reduction of the time required for curing and protection against cold weather

The most common accelerator used for concrete is calcium chloride ($CaCl_2$). Calcium chloride can be safely used up to 2% by weight of cement. The influence of this chemical on setting time and strength development is presented in Fig. 3.2. From this figure it can be seen that (1) the initial and final setting times can be reduced by as much as 50 and 70%, respectively, (2) 1- and 3-day strengths can be increased significantly, and (3) as the curing temperature decreases, the effectiveness of the admixture increases.

The influence of calcium chloride on various properties of concrete is shown in Table 3.5. Almost all of the mechanical properties at early age are improved. The most detrimental effect is on the corrosion of metals. Corrosion becomes a major problem only in locations where there is a steep gradient in the chloride ion concentration. However, even a uniform concentration beyond 2% is viewed with suspicion. The addition of calcium chloride also increases creep and shrinkage and aggravates alkali-aggregate reaction.

A number of nonchloride accelerating admixtures have been developed during the late 1980s. Some of these admixtures are almost as effective as calcium chloride.

Air-Entraining Admixtures

Air-entraining admixtures are used to entrain small spherical air bubbles about 10 to 1000 μm in diameter. Entrained air significantly increases the frost resistance and durability under freezing and thawing conditions. Most specifications mandate air entrainment for exposed structures. About 9% by volume of mortar is recommended for proper freeze-thaw durability. The air voids should be distributed uniformly with low spacing factors (distance between bubbles). Smaller bubbles are more

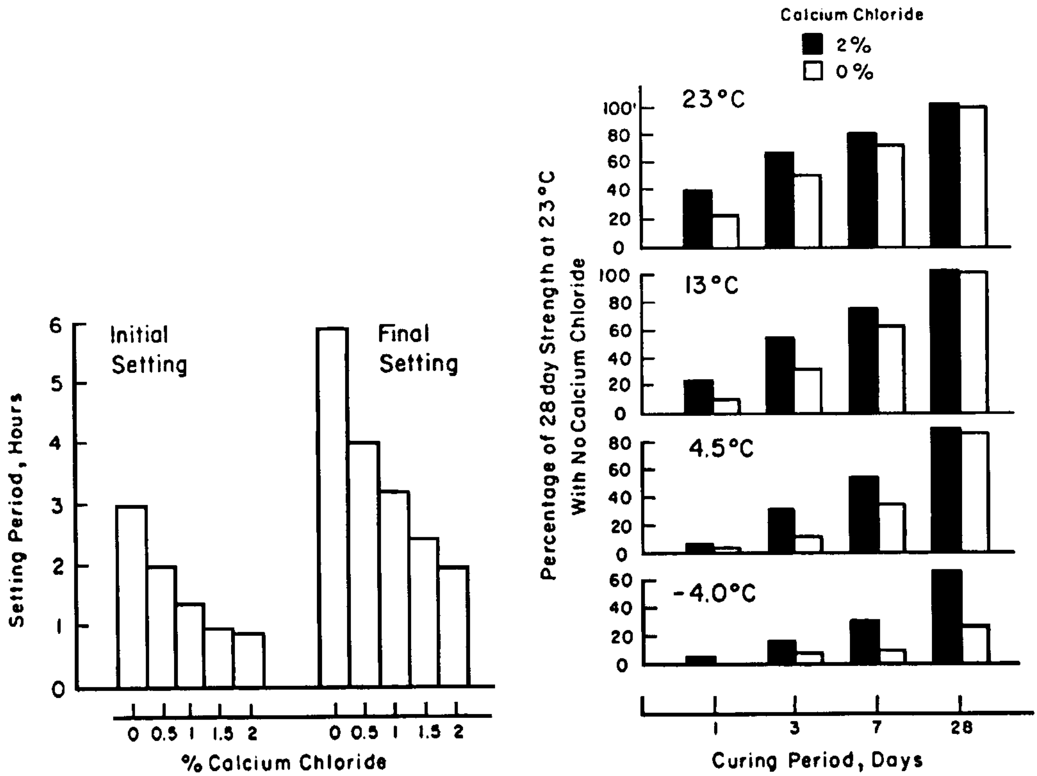


FIGURE 3.2 (a) Effect of calcium chloride addition on setting time of portland cement. (b) Effect of calcium chloride addition on strength at various curing temperatures. (*From Ramachandran.*[3])

TABLE 3.5 Some Properties Influenced by Calcium Chloride Admixture in Concrete

Property	Effect	Remarks
Setting	Reduces both initial and final setting times	ASTM standard requires that initial and final setting times occur at least 1 h earlier with respect to reference concrete.
Compressive strength	Significant increase at 3 days (gain may be about 30–100%)	ASTM requires an increase of at least 125% over control concrete at 3 days.
Tensile strength	Slight decrease at 28 days	
Flexural strength	Decrease of about 10% at 7 days	This figure may vary depending on starting materials and method of curing. The decrease may be more at 28 days.
Heat of hydration	Increase of about 30% in 24 h	Total amount of heat at longer times is almost the same as that evolved by reference concrete.
Resistance to sulfate attack	Reduced	Can be overcome by using type V cement with adequate air entrainment.
Alkali-aggregate reaction	Aggravated	Can be controlled by using low-alkali cement or pozzolana.
Corrosion	Causes no problems in normal reinforced concrete if adequate precautions taken (dosage should not exceed 1.5% $CaCl_2$ and adequate cover given)	Calcium chloride admixture should not be used in prestressed concrete or in a concrete containing a combination of dissimilar metals. Some specifications do not allow use of $CaCl_2$ in reinforced concrete.
Shrinkage and creep	Increased	
Volume change	Increase of 0-15% reported	
Resistance to damage by freezing and thawing	Early resistance improved	At later ages may be less resistant to frost attack.
Watertightness	Improved at early ages	
Modulus of elasticity	Increased at early ages	At longer periods almost same with respect to reference concrete.
Bleeding	Reduced	

Source: From Ramachandran.[3]

effective than larger ones. Entrapped air, which is the result of incomplete compaction, is not effective in improving durability.

The usual dosage of air-entraining mixtures is in the range of 0.02 to 0.06% by weight of cement. Higher dosages may be required for mixes containing type III cement, pozzolan cements, fly ash, or other finely divided powders. The volumetric air content in typical air-entrained concrete varies from 4 to 10%. The presence of air improves the workability of fresh concrete because the bubbles increase the spacing of the solids, resulting in decreased dilatancy. The air also reduces segregation and bleeding. Entrained air is particularly helpful in lightweight concrete because of the unfavorable shape and surface texture of the fine fraction of most light-weight aggregates. The air reduces the harshness of the mix and the bleeding rate in addition to providing improved workability. Entrained air typically reduces the compressive strength. Concrete containing 8 vol % air can be expected to register about 15% reduction in compressive strength as compared to control concrete with no entrained air and comparable workability. Note that to obtain comparable workability, the water content of the control concrete has to be increased.

Most of the commercially available air-entraining admixtures are in liquid form; a few are available in powder, flake, or semisolid form. The ingredients used for manufacturing admixtures include salts of wood resins, synthetic detergents, salts of sulfonated lignin, salts of petroleum acids, salts of proteinaceous materials, fatty and resinous acids and their salts, and organic salts of sulfonated hydrocarbons. The air-entraining admixture is typically added to the mixing water. Air-entrained concrete can also be made using air-entraining portland cement. The former method is preferable because the amount of air can be controlled easily.

The amount of air, or air content, is the primary factor that influences freeze-thaw durability and frost resistance. Other factors that are important include size and distribution of air voids, specific surface, and spacing of bubbles. These factors are influenced not only by the amount of air-entraining agent but also by the nature and proportions of the other ingredients of the concrete, including other admixtures, water–cement ratio and consistency of the mix, type and duration of mixing, temperature, and the type and degree of compaction employed. Based on the constituent materials and the type of mixing, placing, and compaction, the amount of air-entraining agent should be adjusted to obtain the desired air content in the final product.

Set-Controlling Admixtures

Set-controlling admixtures are typically used in conjunction with water-reducing admixtures. As mentioned in Sec. 3. 1.3, retarders have to be used in conjunction with some water-reducing admixtures in order to extend the life, or effectiveness, of the water-reducing admixtures. Materials, such as lignosulfonic acids and their salts and hydroxylated carboxylic acids and their salts, can act as water-reducing, set-retarding admixtures.

Set-retarding admixtures are used by themselves to offset the accelerating effects of high temperatures and to extend the workable period for proper placing. Set retarders are also used to keep the concrete workable for a longer duration. The longer duration may be needed to avoid cold joints in large constructions or to prevent cracking in beams, bridge decks, and composite construction because of the deflections caused when the adjacent spans are loaded.

The amount of retardation obtained depends on the chemical composition of the admixture, its dosage, type and amount of cement, temperature, mixing sequence, and other job conditions. The quantity of admixture required should be determined carefully. Excessive retardation can damage the setting and hardening process, resulting in long-term detrimental effects.

Other Chemical Admixtures

Chemical admixtures other than the ones discussed so far include admixtures used for (1) air detraining, (2) gas forming, (3) producing expansion, (4) damp proofing, (5) bonding, (6) reducing alkali-aggregate expansion, (7) inhibiting corrosion, (8) flocculating, (9) coloring, and (10) fungicidal, germicidal, and insecticidal purposes. In addition, admixtures are available for water thickening and for reducing friction in pumping concrete.

Polymer and latex-modified concretes are also being used, especially for repair and restoration. Polymers are used as bonding agents and surface coatings to reduce permeability of the surface layer. Impervious surface layers typically improve the long-term durability.

3.1.5 Mineral Admixtures

Mineral admixtures in concrete provide improved resistance to thermal cracking because of reduced heat of hydration, reduce the permeability by reducing pore sizes, increase strength, and improve resistance against chemicals such as sulfate water and alkali-aggregate expansion. Most of the mineral admixtures have some pozzolanic property. A pozzolan is defined as a siliceous or siliceous and aluminous material that will chemically react with calcium hydroxide at normal temperatures to form compounds possessing cementitious properties. The material has to be in a finely divided form, and the reaction will take place only in the presence of moisture. Pozzolans possess little or no cementitious value by themselves. Typically, mineral admixtures are used in large volume fractions, generally in the range of 20 to 100% by weight of cement.

Even though a number of naturally occurring pozzolans exist and can be used as admixtures, most of the admixtures used in concrete, especially by industrialized nations, are industrial by-products. The most commonly used industrial by-product is coal fly ash produced by power plants. Blast-furnace slag and silica fume are the other major industrial by-products used in concrete. Silica fume, which contains much finer particles, is typically used for high-strength and impermeable concrete. These three admixtures are discussed next, followed by other mineral admixtures.

Fly Ash

In modern power plants, coal is fed into furnaces in powder form to improve thermal efficiency. As the coal powder passes through the high-temperature zone in the furnace, the volatile matter and carbon burn off, providing heat generation. The impurities, such as clay, feldspar, and quartz, melt and fuse. When the fused matter moves through zones of lower temperature, it solidifies as glassy spheres. The glass contents in these spheres range from 60 to 85%. Some of these spheres are hollow and very light. The spheres get blown out with the flue gas stream. These particles, collected with special equipment such as electrostatic precipitators, are called fly ash. Special equipment is needed for collecting most of the tiny particles so that the amount of ash discharged into the atmosphere is at an absolute minimum.

Based on the amount of calcium content, fly ash is classified as low-calcium or high-calcium. ASTM classifies high-calcium fly ash (CaO content 15 to 35%) as type C and low-calcium fly ash (CaO less than 10%) as type F fly ash. High-calcium fly ash is more reactive because the calcium occurs in the form of crystalline compounds. The chemical that is considered to be harmful to concrete is carbon. In most commercial fly ashes the carbon content is limited to 2%, and it rarely exceeds 5%. If the carbon content is high, the fly ash should not be used in concrete. A higher carbon content typically increases water demand and interferes with air-entraining admixtures.

The sizes of fly ash particles vary from <1 to 100 μm. The particle size distribution is shown in Fig. 3.3. This figure shows the particle distributions of type C and F fly ash, cement, and silica

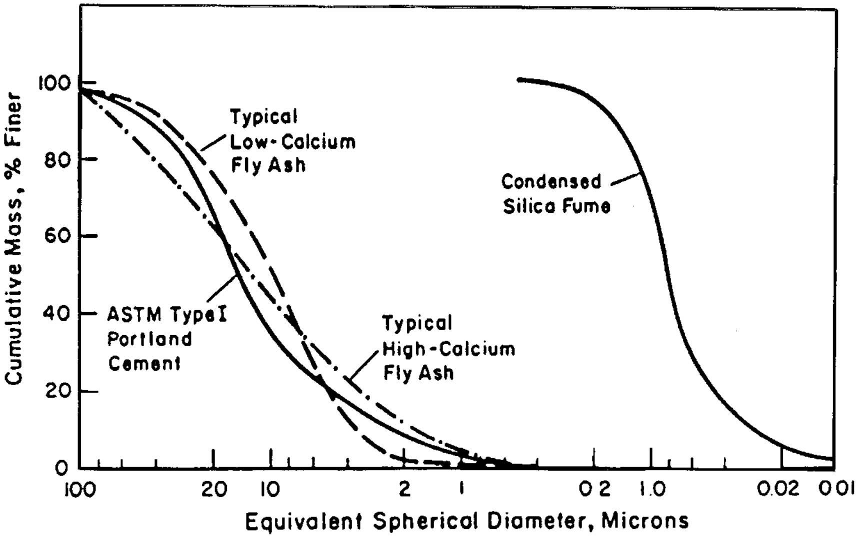


FIGURE 3.3 Particle-size distributions—comparison of cement, fly ash, and silica fume.

fume. It can be seen that the fly ash particle sizes are about the same as the cement particle sizes. About 50% of the particles are smaller than 20 μm. The particle size distribution influences the workability of fresh concrete and the rate of strength development.

It is well established that the addition of fly ash in the range of 10 to 20% by weight of cement improves the workability. Fly ash containing finer particles is more effective in improving workability. As much as 7% water reduction was reported with an addition of 30% fly ash.

One of the primary reasons for using fly ash in concrete is to reduce the heat of hydration during placement and the early curing period. Fly ash could reduce the rise in temperature almost in direct proportion to the amount of cement it replaces. Fly ash has been used in concrete as early as the 1930s. In most of the earlier applications the amount of cement replaced was limited to about 30%. Recently equal amounts of cement and fly ash have been used successfully for mass concrete construction. As mentioned earlier, the advent of superplasticizers was responsible for the use of high volume fractions of fly ash. At lower dosages, fly ash improves the workability. When large volume fractions are used, more water is needed to wet the fly ash particles. The use of superplasticizers allows for the least increase in water content, and hence high strengths can be achieved even with large volume fractions of fly ash.

The pozzolanic activity of fly ash refines the pores, and thus concrete containing fly ash is less permeable. The 90-day permeability of the cement can be reduced by almost an order of magnitude by replacing 1010 30% of cement with fly ash. This reduced permeability enhances the durability against chemical attacks significantly. Alkali-aggregate expansion can also be reduced by adding fly ash.

Replacement of cement with fly ash normally results in a reduction of the 28-day strength. But the long-term (56- or 90-day) strengths are always higher for fly ash concrete. High-strength concretes with compressive strengths greater than 8000 psi (55 MPa) always contain some form of mineral admixture.

Blast-Furnace Slag

Blast-furnace slag is a by-product of cast iron production. The chemical components present in the slag do not react with water at normal temperatures. The slag is ground to fine particles and used as mineral admixtures. In some cases the liquid slag is cooled rapidly to produce sandlike and pellet-like particles, called granulated and pelletized slag, respectively.

The properties of concrete containing slag are almost the same as the properties of concrete containing fly ash, in both the plastic and the hardened forms. The major difference is that slag particles smaller than 10 μm were found to contribute to an increase in the 28-day strength.

Silica Fume

Silica fume, also called condensed silica fume, volatilized silica, or microsilica, is a by-product of silicon metal and ferrosilicon alloy industries. The reduction of quartz to silicon at temperatures of 3632°F (2000°C) produces SiO vapors. These vapors oxidize and condense to form small spheres with diameters in the range of 0.01 to 0.2 μm (Fig. 3.3). Silica fume particles are about two orders of magnitude smaller than cement particles. Because of its extremely small particle size, silica fume is highly pozzolanic. But its higher surface area increases the water demand. Hence the use of silica fume was almost impossible until the advent of superplasticizers. The use of silica fume for high-strength concrete became a common occurrence after the super-plasticizers were introduced.

The silica fume content in concrete ranges from 5 to 30% by weight of the cement. The addition of silica fume typically results in denser concrete with low permeability. Silica fume is more effective in reducing permeability than fly ash. The strength increase provided by silica fume is also more substantial than that obtained by the addition of fly ash. Because of the collecting and processing expense, silica fume is much more expensive than fly ash. It is available in powder and slurry form. Silica fume concrete typically has a higher Young's modulus and is more brittle than concrete of comparable strength that contains no silica fume.

Rice-Husk Ash

Rice-husk ash can be considered a natural product, even though it is produced by controlled combustion of rice husks. Rice husks are the product of a dehusking operation in which outer shells are removed from rice. Husk constitutes about 20% of paddy by weight. The combustion process reduces the weight about fivefold. Ash produced by controlled combustion was found to produce nonreactive silica minerals. These ashes have to be finely ground before using in concrete.

Naturally Occurring Mineral Admixtures

Naturally occurring pozzolans are typically mined and processed. Processing normally involves the steps of crushing, grinding, and size separation. In some cases thermal activation may be needed.

Natural pozzolans are derived from volcanic rocks and minerals and from diatomaceous earth. Diatomaceous earth consists of hydrated silica derived from skeletons of diatoms. Diatoms are tiny water plants whose walls are composed of silica shells. Pozzolans were formed during volcanic eruption because of the quick cooling of magma-containing aluminosilicates.

Based on their major chemical components, natural pozzolans can be classified as (1) volcanic glasses, (2) volcanic tuffs, (3) calcined clays or shales, and (4) diatomaceous earth.

3.2 COMPUTATION OF REQUIRED AVERAGE COMPRESSIVE STRENGTH f'_{cr} AND MIX PROPORTIONING

The structural engineer who designs the components specifies the minimum compressive strength required for the concrete to be used. This strength is called specified compressive strength f'_c In most cases the specified strength is measured at the age of 28 days. In some cases 56- or 90-day strengths are also specified. Compressive strength tests are normally conducted using cylinders, cubes, or prisms. It is the job of the construction professionals to ensure that the concrete placed in position satisfies the specified strength requirement. Since concrete is a composite, cast in the field using a number of constituent materials and various casting and curing procedures, there is always a variation in strength. The major parameters that influence the strength include the following:

1. Amount of water used in the mix, or water–cement ratio
2. Aggregate-cement ratio
3. Quality of cement
4. Strength, shape, texture, cleanliness, and moisture content of aggregates
5. Type and amount of mineral and chemical admixtures
6. Mixing procedure and adequacy of mixing
7. Placing, compacting, and finishing techniques used during construction
8. Curing conditions and type of curing method
9. Test procedures

Because of variations in any of these parameters, or other factors such as temperature and humidity in the field during the construction, there is always a variation in compressive strength. Typically, quality control tests are run in both the field and the laboratory to monitor the variations and take corrective measures if needed. American Concrete Institute (ACI) code 318-92 specifies the following acceptance criteria for concrete[4]:

1. The average of all sets of three consecutive strength tests must equal or exceed f'_c, and
2. No individual strength test (average of two cylinders) must fall below f'_c by more than 500 psi (3.4 MPa)

The code also provides guidelines for achieving the acceptable concrete.

Stated in simple terms, the code requires that the concrete be proportioned to obtain an average compressive strength f'_{cr}, that is higher than the specified strength f'_c. The magnitude of overdesign, that is, the difference between f'_{cr} and f'_c, depends on the rigorousness and success of the quality control measures used on the job site. The concrete should be proportioned to obtain the average compressive strength f'_{cr}, and not the specified strength f'_c.

This section deals with the computation of f'_{cr}, which is also known as required average strength, and the mix proportioning procedures. The computation of f'_{cr} is based on ACI code guidelines. It should be noted that other codes may require different procedures for the computation of the required average strength.

3.2.1 Computation of Required Average Strength, f'_{cr}

If the concrete production facility has test records from previous projects, these records can be used to establish the variability and the mix proportions. In this case the computation of f'_{cr} is based on previous records. If the records are not available, a certain variability is assumed. As the project progresses, the data from the project can be used to establish the variability.

If field data are available, the computation of f'_{cr} requires determination of the standard deviation and the number of acceptable low tests. It is assumed that the strength variation of concrete (proportioned to obtain the same compressive strength) follows a normal distribution. If a large number of samples is collected, they were found to follow the normal distribution curve. The ACI code specifies a minimum of 30 samples. If 30 or more samples are available, their average and their standard deviation are assumed to be the same as the average and the standard deviation of a large population.

Once the strength variation is assumed to follow a normal distribution, the properties of the normal distribution curve can be used to predict the probability that a given sample will have a strength less than f'_{cr}. For example, if the average is x, and the standard deviation is s, it can be said that:

- Half the samples will have strengths less than x, and hence the probability that a certain sample will have a strength less than x is 50%.
- About 68.27% of all samples will have strengths greater than $(x - s)$ and less than $(x + s)$, or 15.87% of samples will have strengths less than $(x - s)$, or the probability that a given sample will have strength less than $(x - s)$ is 15.87%.
- About 95 .45% of all samples will have strengths between $(x - 2s)$ and $(x + 2s)$, or the probability that an individual sample will have strength less than $(x - 2s)$ is 2.25%.
- About 0.13% of all samples, or 1 in 741 samples, will have a strength less than $(x - 3s)$.

The aforementioned postulations were derived using the property of the normal or bell curve. It can be seen that the two factors that control the prediction are the sample average x and the standard deviation s.

If the average of a given set of strength results x, is assumed to be the required average strength f'_{cr}, then the mix proportions used to obtain the strengths are good enough for a specified strength f'_c subjected to the condition

$$f'_{cr} = f'_c + tS \tag{3.1}$$

where t is a statistic which depends on the number of tests permitted to fall below f'_c. If 50% of the tests are permitted to fall below, then $t = 0$, or f'_{cr} and f'_c are the same. Naturally, in the actual construction such a large number of low tests cannot be permitted. For structural concrete, the ACI code specifies the following two equations:

$$f'_{cr} \geq f'_c + 1.34S \tag{3.2}$$

$$f'_{cr} \geq f'_c + 2.33S - 500 \text{ psi} \qquad (1000 \text{ psi} = 6.89 \text{ MPa}) \tag{3.3}$$

Equation (3.2) results in a probability that not more than 1 in 100 averages of three consecutive strength tests (each being the average of two cylinders) will be less than f'_c. The t value for a probability of 1 in 100 is 2.33, and the number 1.34 is obtained by dividing 2.33 by 3.The division by 3 for three consecutive tests is based on theorems in statistics. Equation (3.3) results in the probability that not more than 1 individual strength in 100 (average of two cylinders) falls below f'_c – 500 psi (f'_c – 3.4 MPa). Note that these two equations are consistent with both acceptance criteria presented previously. Overall ACI code recommendations for the computation of f'_{cr} are based on the acceptance of 1 low test in 100. It should be noted that this probability is used only for the computation of f'_{cr}, and if the low strengths do occur, correction measures should be taken to increase the average strengths.

For the construction of facilities such as footpaths, a higher number of low tests could be acceptable. In this case the t value is reduced. For example, if 1 low test in 10 is acceptable, then the t value is 1.28. More information on the t values for various probabilities and the recommended low tests for various facilities can be found in the report of ACI Committee 214.[5]

Once Eqs. (3.2) and (3.3) are accepted as the basis for the computation of f'_{cr}, the process becomes a set of mathematical steps. The ACI code also establishes procedures for accepting data sets with less than 30 tests. In addition there are certain restrictions to be satisfied for using the data from previous projects. The following is the gist of the various provisions of the ACI code. The restrictions and their interdependence are presented as a flowchart in Fig. 3.4.

If a concrete production facility has test records, establish the standard deviation using its records, provided that the material quality control procedures and conditions are similar to the proposed project and the f'_c for the proposed work is within 1000 psi (6.89 MPa) of the f'_c for which the records exist.

If the records contain 30 or more consecutive tests, compute the standard deviations using the equation

$$S = \left[\frac{(x_i - \bar{x})^2}{n - 1} \right]^{1/2} \tag{3.4}$$

where n = number of consecutive strength tests

x_i = individual strength tests (average of two cylinders at 28 days or at designated test age for the determination of f'_c).

$$\bar{x} = \frac{x_i}{n} \tag{3.5}$$

If the records contain two groups of consecutive tests totaling at least 30 tests, compute the standard deviations S_1 and S_2 for the two sets using Eqs. (3.4) and (3.5). Compute the statistical average of S_1 and S_2 using

$$S = \left[\frac{(n_1 - 1)S_1^2 + (n_2 - 1)S_2^2}{n_1 + n_2 - 2} \right]^{1/2} \tag{3.6}$$

where S = statistical average standard deviation where two test records are used to estimate standard deviation

S_1, S_2 = standard deviations of sets 1 and 2

n_1, n_2 = number of tests in respective test record

If the available number of tests is less than 30 but greater than 25, multiply the estimated standard deviation by a factor of 1.03 and use this value in Eqs. (3.2) and (3.3). The corresponding factors for data sets with a minimum of 20 and 15 tests are 1.08 and 1.16, respectively.

Using the estimated value of X, compute the required average strength f'_{cr} using Eqs. (3.2) and (3.3).

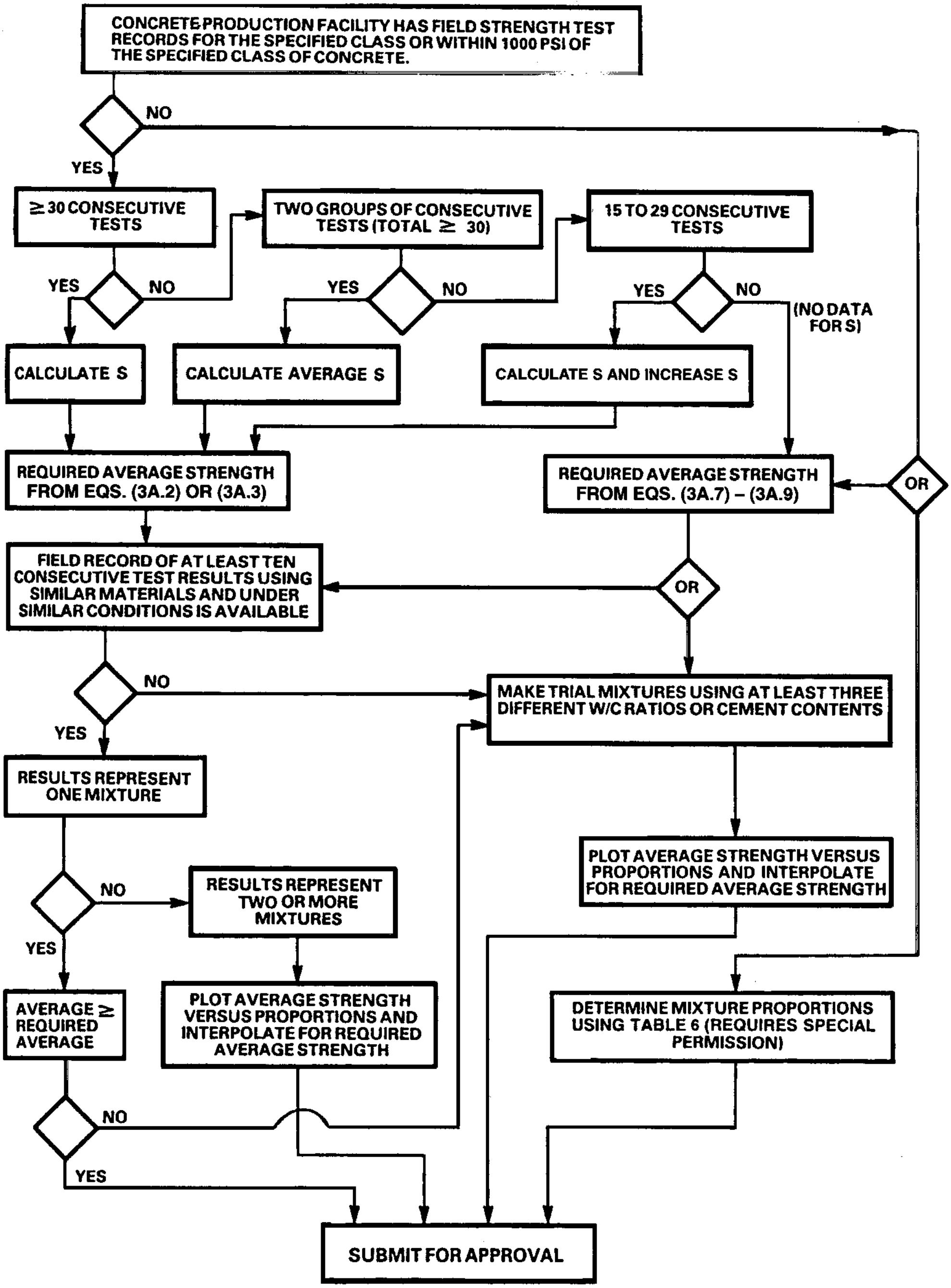


FIGURE 3.4 Flowchart for selection and documentation of concrete proportions. (*From ACI 318-92.*[4])

If data are not available for estimating the standard deviation,

$$f'_{cr} = f'_c + 1000 \text{ psi (6.89 MPa)} \qquad f'_c < 3000 \text{ psi (20.7 MPa)} \tag{3.7}$$

$$= f'_c + 1200 \text{ psi (8.27 MPa)} \qquad 3000 \le f'_c \le 5000 \text{ psi } (20.7 \le f'_c \le 34.5 \text{ MPa}) \tag{3.8}$$

$$= f'_c + 1400 \text{ psi (9.64 MPa)} \qquad f'_c > 5000 \text{ psi (34.5 MPa)} \tag{3.9}$$

Equations (3.7) to (3.9) are based on conservative (or overestimated) estimates of S values.

The following numerical examples further illustrate the procedure used for the computation of f'_{cr} under various conditions.

Example 3.l Compute the required average compressive strength f'_{cr} for the following cases if the specified compressive strength is 4000 psi (27.6 MPa). Assume similar materials and conditions for all cases. The available test results (average of two cylinders) are as follows:

(*a*)	(*b*)	(*c*)	(*d*)	(*e*)
4260	5200	*First set:*	4700	None
4760	5220			
4120	4750			
3650	4440	3100	4050	
4310	4750	4450	3500	
4960	4720	3900	3250	
4350	4650	4900	4100	
3980	5020	3750	4350	
4450	4920	4100	3750	
4430	5780	4400	3600	
4240	5350	4500	3550	
4400	5420	5100	3900	
3420	5070	3500	3850	
4760	5220	3700	4150	
4620	4200	3250	4200	
4260	5070	3750	4600	
3860	5600	4250	4300	
4290	5780	*Second set:*		
5120	4900	4700		
4870	5980	4750		
4150	5200	4850		
4170	4320	3750		
3740	5170	3050		
4180	5030	3000		
4860	5700	3900		
3890	5050	3850		
4720	5200	4100		
4880	4130	4150		
4980	4450	4350		
4240	5300	4400		
	4600	4950		
	4850	4050		
	6140	3450		
	5100	3200		
	5070	3250		
	4850			

1000 psi = 6.g95 MPa.

Solution In case (*a*) the number of tests ≧ 30. Hence the single set can be used:

$$\bar{x} = \frac{x_i}{n}$$

$$= \frac{4260 + 4760 + \cdots + 4980}{30}$$

$$= 4364 \text{ psi } (30 \text{ MPa})$$

The average is within 1000 psi of f'_c. Thus

$$S = \left[\frac{(x_i - x)^2}{n-1}\right]^{1/2}$$

$$= \left[\frac{(4364 - 4260)^2 + \cdots + (4364 - 4240)^2}{29}\right]^{1/2}$$

$$= 426 \text{ psi } (2.94 \text{ MPa})$$

$$f'_{cr} > 4000 + 1.34 \times 426 = 4571 \text{ psi } (31.5 \text{ MPa})$$

$$> 4000 + 2.33 \times 426 - 500 = 4493 \text{ psi } (31.0 \text{ MPa})$$

Hence f'_{cr} = 4571 psi, or 4600 psi (32 MPa).

In case (*b*) the number of tests is greater than 30. Hence the single set can be used:

$$\bar{x} = \frac{5200 + \cdots + 4850}{36}$$

$$= 5061 \text{ psi } (34.9 \text{ MPa})$$

The difference between the average strength and the specified strength of 4000 psi is greater than 1000 psi. Hence the data set cannot be used. The problem has to be treated as though data were not available. Equation (3.8) controls,

$$f'_{cr} = f'_c + 1200 \text{ psi}$$

$$= 4000 + 1200 \text{ psi} = 5200 \text{ psi } (36 \text{ MPa})$$

In case *c*) the average for the first data set $\bar{x}_1$ is 4046 psi, $n_1 = 14$. The average for the second data set $\bar{x}_2$ = 3985 psi, $n_2 = 17$. Both averages are within 1000 psi of f'_c and can be used for the computation of f'_c:

$$S_1 = 590 \text{ psi } (4.07 \text{ MPa})$$

$$S_2 = 637 \text{ psi } (4.39 \text{ MPa})$$

$$S = \left[\frac{13 \times 590^2 + 16 \times 637^2}{14 + 17 - 2}\right]^{1/2}$$

$$= 616 \text{ psi } (4.25 \text{ MPa})$$

$$f'_{cr} > 4000 + 1.34 \times 616 = 4825 \text{ psi } (33.3 \text{ MPa})$$
$$f'_{cr} > 4000 + 2.33 \times 616 - 500 = 4935 \text{ psi } (34.0 \text{ MPa})$$

Therefore,

$$f'_{cr} = 4935 \text{ psi,} \qquad \text{or } 4950 \text{ psi } (34 \text{ MPa})$$

In case (*d*) 15 tests are available:

$$\overline{X} = 3990 \text{ psi}$$

X is within 1000 psi of f'_c,

$$S = 415 \text{ psi } (2.86 \text{ MPa})$$

The multiplying factor is 1.16. Therefore S is used for the computation:

$$f'_{cr} = 1.16 \times 415 = 481 \text{ psi } (3.32 \text{ MPa})$$
$$f'_{cr} > 4000 + 1.34 \times 481\ 4645 \text{ psi } (32.0 \text{ MPa})$$
$$f'_{cr} > 4000 + 2.33 \times 481 - 500 = 4621 \text{ psi } (31.9 \text{ MPa})$$

Therefore,

$$f'_{cr} = 4645 \text{ psi,} \qquad \text{or } 4650 \text{ psi } (32 \text{ MPa})$$

For case (*e*) data are not available. Therefore,

$$f'_{cr} = 4000 + 1200 \text{ psi} = 5200 \text{ psi } (36 \text{ MPa})$$

which is the same as case (*b*).

3.2.2 Selection of Concrete Proportions

Once the required average compressive strength f'_{cr} is established, the mix proportions that can produce average strengths equal to or greater than f'_{cr} can be chosen based on (1) field records or (2) trial mixes. It should be noted that the required average strength f'_{cr} is only one of the parameters to be considered in mix proportioning. Other primary requirements are workability, consistency, and resistance to special exposure conditions. The final mix proportion should also be the most economical solution for the given set of conditions.

Workability of fresh concrete determines the ease with which the concrete can be transported, placed in position, consolidated, and finished. A mix that is too difficult to handle or consolidate may result in a final product that has honeycombs. Poorly consolidated concrete will not only have poor strength but will deteriorate quickly. Similarly, a mix with excess water may lead to segregation and bleeding, again resulting in a poor final product. The consistency of the mix should be just sufficient for proper placing, consolidation, and finishing operations. Slump is the most widely used indicator of workability. Section 3.3. 1 deals with the slump and its measurement. For general applications, concrete with sufficient strength is automatically assumed to be durable. However, special precautions should be taken when proportioning concrete that is to be exposed to severe environments. The following are the requirements for certain exposure conditions. The list is not comprehensive. Each structure should be treated carefully based on the exposure conditions.

- Normal-weight and lightweight concrete exposed to freezing and thawing or deicer chemicals should be air-entrained. The ACI code specifies 3.5 to 6% and 4.5 to 7.5% air content for moderate and severe exposure conditions, respectively. Higher air content is needed for mixes with smaller-size aggregates. For concretes with strengths higher than 5000 psi (35 MPa) the air-content requirement is slightly relaxed because these concretes made with lower water–cement ratios are generally less permeable. In addition to air entrainment, the water–cement ratio should be limited to 0.45 for concrete subjected to freezing and thawing.
- If the concrete is to be used for reinforced structural components exposed to salts, seawater, or brackish water, the water–cement ratio should be limited to 0.40.
- Water-cement ratio restrictions (preferably not more than 0.45) should also be used for concrete exposed to sulfates.
- For concrete exposed to freezing and thawing in the presence of deicing salts, the ACI code also stipulates a minimum cement content of 520 lb/yd^3 (310 kg/m^3).

The importance of the economics, or the cost-optimum solution, cannot be overemphasized. Typically, cement is the most expensive ingredient in concrete. However, because of the large quantities of materials involved, even a few cents per ton of aggregate may translate into millions of dollars. When selecting ingredients, availability and transportation costs should be considered carefully. In certain instances, locally available materials could be used in conjunction with admixtures rather than transporting materials that do not need admixtures for long distances. Generally the least amount of cement that is needed for obtaining the required strength and durability provides the best economical solution. The least amount of cement also provides some technical advantages, such as lower heat of hydration and less shrinkage. The use of mineral admixtures such as fly ash also provides cost savings. The use of industrial by-products produces savings in disposal costs and better utilization of resources. These aspects are important for both industrialized countries that have limited disposal space and developing countries that have limited resources.

3.2.3 Proportioning on the Basis of Field Strength Records

If field strength records are available for concrete with an average compressive strength in the range of f'_{cr}, the mix proportions used for this concrete can be used for the new project provided they satisfy the workability and durability requirements outlined in the previous section. The field test records should satisfy the following conditions:

- The available test records shall represent materials and conditions similar to those expected. Quality control, in terms of uniformity of materials, conditions, and proportions, shall be the same or better for the proposed work as compared to the project from which the records are taken. If field records were used for the computation of f'_{cr}, the same records can be used for the selection of mix proportions.
- It is preferable to have 30 or more consecutive test records. However, 10 consecutive test results may be used if the records encompass a period of time not less than 45 days.
- Mix proportions can also be chosen by interpolation using records that resulted in compressive strengths higher and lower than f'_{cr}.

3.2.4 Proportioning on the Basis of Trial Mixtures

When acceptable field records are not available, mix proportions can be established using trial mixtures. Trial mixes should be made using at least three different water–cement ratios or cement contents to establish a relation between compressive strength and water–cement ratio or between strength and cement content. The relationship of strength versus water–cement ratio can be used to establish the maximum water–cement ratio that can produce the required f'_{cr}. On the other hand, if the strength versus cement content relationship is obtained, it can be used to establish the minimum cement content. In either case the chosen mix proportion should satisfy the requirements for workability and durability. The trial mixes should also meet the following restrictions:

- The combination of the materials used should be the same as that of the materials to be used for the proposed work.
- Extrapolations should not be used. The trial mixes should have strengths both smaller and higher than f'_{cr}.
- The slump of the trial mix should be within 0.75 in (19 mm) of the permitted slump of the proposed work. Similarly, the air content should be within 0.5% for air-entrained concrete.
- For each test variable, at least three cylinders should be tested at 28 days or the age designated for the determination of f'_c.

The actual proportioning of the constituent materials is explained in Sec. 3.2.6.

3.2.5 Proportioning on the Basis of Maximum Water-Cement Ratio

The ACI code also allows proportioning using a maximum permissible water–cement ratio for the chosen f'_{cr}. The code recommends the maximum water–cement ratio that can be used for either air-entrained or non-air-entrained concrete (Table 3.6). These water–cement ratios cannot be used for lightweight concrete or concrete containing admixtures. The chosen water–cement ratio should also satisfy the durability requirements.

3.2.6 Computation of Mix Proportions

This section deals with the actual computation of the amounts of the various constituent materials that will result in a concrete with the required strength and durability. A number of procedures are available for proportioning normal-weight, lightweight, and heavyweight concretes. The method proposed by ACI Committee 211 for normal-weight concrete[7] is explained here. The other popular method used in the United States is the PCA method. This method is explained in a manual pub-

TABLE 3.6 Maximum Permissible Water-Cement Ratio for Concrete When Strength Data from Field Experience or Trial Mixtures Are Not Available

	Absolute water–cement ratio by weight	
Specified compressive strength f'_c, psi* (MPa)	Non-air-entrained concrete	Air-entrained concrete
2500 (17)	0.67	0.54
3000 (21)	0.58	0.46
3500 (24)	0.51	0.40
4000 (28)	0.44	0.35
4500 (31)	0.38	†
5000 (34)	†	†

*28day strength.
†For strengths above 4500 psi (31 MPa) (non-air-entrained concrete) and 4000 psi (28 MPa) (air-entrained concrete), concrete proportions shall be established by using trial mixes.
Source: From ACI Building Code.[6]

lished by Portland Cement Association.[8] The details of the procedure used in Britain, which is similar to the methods used in western Europe, Australia, and Asia, can be found in Neville.[9] For the sake of brevity, lightweight and heavyweight concretes are not discussed here. The details can be found in reports published by ACI Committee 211.[7,10]

The water–cement ratio is the most influential factor that affects strength. Hence the design charts and tables developed for mix proportioning are geared toward obtaining the minimum water–cement ratio that would produce a workable concrete. The following is the step-by-step procedure to estimate the quantities of various ingredients.

1. *Collection of background information:* The following information should be collected before starting the computations:
   - Sieve analysis data for fine and coarse aggregate including fineness modulus
   - Bulk specific gravity of aggregates, cement, and admixtures in solid (powder) form
   - Dry-rodded unit weight of coarse aggregate
   - Moisture content of fine and coarse aggregates
   - Ratio of solid-to-liquid contents of liquid (or slurry) admixtures
   - Special conditions such as permissible maximum water–cement ratio, minimum cement content, minimum air content, minimum slump, maximum size of aggregate, and strength requirements at early age
2. *Selection of slump:* If the slump is not specified, choose an appropriate value from Table 3.7. A minimum possible value should be chosen within the specified range.
3. *Selection of maximum size for aggregate:* For the same volume fraction a large maximum size of well-graded aggregate provides the least void space, requiring the least amount of mortar content. Hence the maximum possible aggregate size, consistent with the type of application, should be chosen. The maximum size should satisfy the following restrictions:

   $\ngtr \frac{1}{5}$ narrowest dimension between sides of form

   $\ngtr \frac{3}{4}$ of minimum clear spacing between reinforcing bars

   $\ngtr \frac{1}{3}$ depth of slab
4. *Estimation of amount of water and air:* The amount of water required to produce a given slump depends on the aggregate properties, the concrete temperature, and the amount of entrained air. The aggregate properties that influence the slump are maximum size, shape, and grading. The admixture can also influence the slump, the most notable being a high-range water-reducing admixture. Cement content, within the normal range, does not influence the slump. Table 3.8 can be used to estimate the approximate amount of water needed. It provides guidelines for both air-entrained and non-air-entrained concrete. Note that the table gives only approximate values, and the influence of any admixtures other than air-entraining admixtures is not considered. Only trial batches can establish the actual water and the corresponding air content.

TABLE 3.7 Recommended Slump for Various Types of Construction

Type of construction	Maximum slump*	Minimum slump
Reinforced foundation walls and footings	3 in (75 mm)	1 in (25 mm)
Plain footings, caissons, and substructure walls	3 in (75 mm)	1 in (25 mm)
Beams and reinforced walls	4 in (100 mm)	1 in (25 mm)
Building columns	4 in (100 mm)	1 in (25 mm)
Pavements and slabs	3 in (75 mm)	1 in (25 mm)
Mass concrete	2 in (50 mm)	1 in (25 mm)

*May be increased I in (25 mm) for methods of consolidation other than vibration.
Source: From Ad Committee 211.[7]

TABLE 3.8 Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Maximum Sizes of Aggregates

Slump, in	Water, lb/yd³ of concrete for nominal maximum aggregate sizes							
	⅜ in[a]	½ in[a]	¾ in[a]	1 in[a]	1½ in[a]	2 in[a,b]	3 in[b,c]	6 in[b,c]
Non-air-entrained concrete								
1–2	350	335	315	300	275	260	220	190
3–4	385	365	340	325	300	285	245	210
6–7	410	385	360	340	315	300	270	—
>7[a]	—	—	—	—	—	—	—	—
Approximate amount of entrapped air in non-air-entrained concrete, %	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
1–2	305	295	280	270	250	240	205	180
3–4	340	325	305	295	275	265	225	200
6–7	365	345	325	310	290	280	260	—
>7[a]	—	—	—	—	—	—	—	—
Recommended average[d] total air content for level of exposure, %								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5[e,f]	1.0[e,f]
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5[e,f]	3.0[e,f]
Severe exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5[e,f]	4.0[e,f]

[a]The quantities of mixing water given for air-entrained concrete are based on typical total air content requirements as shown for moderate exposure. These quantities of mixing water are for use in computing cement contents for trial batches at 68–77 °F. They are maximum for reasonably well-shaped angular aggregates graded within limits of accepted specifications. Rounded aggregate will generally require 30 lb less water for non-air-entrained and 25 lb less for air-entrained concretes. The use of water producing chemical admixtures (ASTM C494) may also reduce mixing water by 5% or more. The volume of the liquid admixtures is included as part of the total volume of the mixing water. The slump values of more than 7 in are only obtained through the use of water-reducing chemical admixture; they are for concrete containing nominal maximum-size aggregate not larger than tin.

[b]The slump values for concrete containing aggregate larger than 1/2 in are based on slump tests made after removal of particles larger than 1½ in by wet screening.

[c]These quantities of mixing water are for use in computing cement factors for trial batches when 3- or 6-in nominal maximum-size aggregate ta used. They are average for reasonably well-shaped coarse aggregate, well-graded from coarse to fine.

[d]Additional recommendations for air content and necessary tolerances on air content for control in the field are given in a number of ACt documents, including ACI 201, 345, 3lg, 301, and 302. ASTM C94 for ready-mixed concrete also gives air-content limits. The requirements in other documents may not always agree exactly, so in proportioning concrete consideration must be given to selecting an air content that will meet the needs of the job and also the applicable specifications.

[e]For concrete containing large aggregates that will be wet-screened over the 1½-in sieve prior to testing for air content, the percentage of air expected in the 1½-in material should be as tabulated in the 1½-in column. However, initial proportioning calculations should include the air content as a percent of the whole.

[f]When using large aggregate in low cement factor concrete, air entrainment need not be detrimental to strength. In most cases the mixing water requirement is reduced sufficiently to improve the water–cement ratio and to thus compensate for the strength-reducing effect of air-entrained concrete. Generally, therefore, for these large nominal maximum sizes of aggregate, air contents recommended for extreme exposure should be considered even though there may he little or no exposure to moisture and freezing.

[g]These values are based on the criteria that 9% air is needed in the mortar phase of the concrete. if the mortar volume will be substantially different from that determined in this recommended practice, it maybe desirable to calculate the needed air content by taking 9% of the actual mortar volume.

Note: ho 25.4 mm; 1 lb/yd³ = 0.59 kg/m³.

Source: From ACI Committee 211.

TABLE 3.9 Relationships between Water-Cement Ratio and Compressive Strength of Concrete

Compressive strength at 28 days, psi* (MPa)	Water-cement ratio, by weight	
	Non-air-entrained concrete	Air-entrained concrete
6000 (41)	0.41	—
5000 (35)	0.48	0.40
4000 (28)	0.57	0.48
3000 (21)	0.68	0.59
2000 (14)	0.82	0.74

*Values are estimated average strengths for concrete containing not more than the percentage of air shown in Table 3.g. For a constant water–cement ratio, the strength of concrete is reduced as the air content is increased. Strength is based on 6- by 12-in cylinders moist cured 28 days as 73.4 ± 3°F (23 ± 1 .7°C) in accordance with Sec. 9(b) of ASTM C3 I, *Making and Curing Concrete Compression and Flexure Test Specimens in the Field*.

Source: From Act Committee 211.[7]

5. *Selection of water–cement, or water–cementitious-materials, ratio*. Water-cement ratio is a classical term. When mineral admixtures such as fly ash or silica fume are used, they could be considered as part of the cementitious materials. In this case the water is computed based on the weight of the cementitious (cement + fly ash or silica fume) materials, rather than just the weight of cement. As mentioned earlier, the amount of water used should satisfy both strength and durability requirements. Information given in Table 3.9 can be used as an approximate first step to establish the water–cement ratio. Since the strength is also affected by factors such as aggregate and cement types and by the properties of other cementitious materials, the values shown in Table 3.9 should be used only as a guideline. It is highly desirable to develop a water–cement (cementitious materials) ratio for the particular type of materials to be used in the proposed work.

   If the structure to be built is going to be exposed to severe environmental conditions, water–cement (cementitious materials) ratios should be limited to the values shown in Table 3.10. As mentioned earlier, a lower water content typically reduces permeability and improves overall durability.

6. *Computation of cement content:* Once the amount of water and the water–cement ratio are established, the computation of the amount of cement becomes a simple division of water content by water–cement ratio. The cement content should also satisfy any special minimum cement content stipulated in the specification.

TABLE 3.10 Maximum Permissible Water-Cement Ratios for Concrete in Severe Exposures

Type of structure	Structure wet continuously or frequently and exposed to freezing and thawing*	Structure exposed to seawater or sulfates†
Thin sections (railings, curbs, sills, ledges, ornamental work) and sections with less than 1-in (25 mm) cover over steel	0.45	0.40
All other structures	0.50	0.45

*Concrete should also be air-entrained.

†If sulfate-resisting cement (type It or type v of ASTM C150) is used, the permissible water–cement ratio may be increased by 0.05.

Source: From ACI Committee 211.[7]

7. *Estimation of coarse aggregate content:* Typically the use of more coarse aggregate per unit volume of concrete leads to better economy. The larger the size of the particles in coarse aggregate and the finer the sand, the more volume fraction of coarse aggregate can be incorporated without sacrificing workability. The volume fractions of coarse aggregate that will produce a workable mix for various maximum aggregate sizes and fineness moduli of sand are shown in Table 3.11.

   For the chosen maximum aggregate size, say 1 in (25 mm), and the fineness of the sand to be used, say 2.6, the table provides the volume fraction of coarse aggregate in the dry-rodded form, 0.69. For 1 yd^3 (0.76 m^3) of concrete, the volume of coarse aggregate is 0.69 × 27, or 18.63 ft^3 (0.52 m^3). The corresponding weight is obtained by multiplying 18.63 by the dry-rodded unit weight. The values shown in Table 3.11 can be reduced by up to 10% to improve the workability for special circumstances, such as pumping or concreting members with congested reinforcement.

8. *Estimation of fine aggregate content:* At the completion of step 7, the amounts of all ingredients, except fine aggregate, have been estimated. Hence if the unit weight of fresh concrete is known, the weight of fine aggregate can be estimated by subtracting the total weight of all other ingredients from the weight of fresh concrete. This type of computation is called the weight method. In the absence of previous experience, a first estimate of the unit weight of concrete can be obtained using Table 3.12. The table covers both air-entrained and non-air-entrained concrete. Medium-rich concrete and a coarse aggregate specific gravity of about 2.7 have been assumed for developing Table 3.12. Even rough estimates of unit weight were found to provide satisfactory results for trial mixes.

   There is another procedure, called the absolute volume method, in which the volume of fine aggregate is computed by subtracting the volumes of all other ingredients from the unit volume of fresh concrete. This method is considered more accurate, but the specific gravity of all ingredients is needed prior to the computation.

9. *Adjustments for aggregate moisture:* In most cases the stock aggregates retain some moisture. The computations of aggregate weights in steps 7 and 8 are based on saturated surface-dry conditions. Hence the weight of moisture present in the aggregate should be accounted for. The easiest way to make the correction is to adjust the weight of aggregates for moisture. For example,

TABLE 3.11 Volume of Dry-Rodded Coarse Aggregate* per Unit Volume of Concrete for Different Fineness Moduli of Sand

	Fineness modulus of sand			
Maximum size of aggregate, in (mm)	2.40	2.60	2.80	3.00
⅜ (9)	0.50	0.48	0.46	0.44
½ (13)	0.59	0.57	0.55	0.53
¾ (19)	0.66	0.64	0.62	0.60
1 (25)	0.71	0.69	0.67	0.65
1½ (38)	0.75	0.73	0.71	0.69
2 (50)	0.78	0.76	0.74	0.72
3 (76)	0.82	0.80	0.78	0.76
6 (152)	0.87	0.85	0.83	0.81

*Volumes are based on aggregates in dry-rodded condition as described in ASTM C29, Unit Weight of Aggregate." These volumes are selected from empirical relationships to produce concrete with a degree of workability suitable for usual reinforced construction. For less workable concrete such as required for concrete pavement construction the volume may be increased by about 100%. For more workable concrete, such as may sometimes he required when placement is to be by pumping, it maybe reduced by up so 10%.

Source: From Ad Committee 211.[7]

TABLE 3.12 First Estimate of Weight of Fresh Concrete

Maximum size of aggregate, in (mm)	Concrete weight,* lb/yd³	
	Non-air-entrained concrete	Air-entrained concrete
⅜ (9)	3840 (2266)	3710 (2189)
½ (13)	3890 (2295)	3760 (2218)
¾ (19)	3960 (2336)	3840 (2266)
1 (25)	4010 (2366)	3850 (2272)
1½ (38)	4070 (2401)	3910 (2307)
2 (50)	4120 (2431)	3950 (2331)
3 (76)	4200 (2478)	4040 (2384)
6 (152)	4260 (2513)	4110 (2425)

*Values calculated for concrete of medium richness (550 lb of cement per cubic yard or 325 kg per cubic meter) and medium slump with aggregate specific gravity of 2.7. Water requirements based on values for 3 to 4 in (75 to 100 mm) of slump are given in Table 3.8. If desired, the estimated weight may be refined as follows when necessary information is available. For each 10-lb (4.5-kg) difference in mixing water from Table 3.8 values for 3 to 4 in (75 to 100 mm) of slump, correct the weight per cubic yard by 15 lb (6.8 kg) in the opposite direction; for each 100-lb (45.4-kg) difference in cement content from 550 lb (250 kg), correct the weight per cubic yard by 15 lb (6.8 kg) in the same direction; for each 0.1 by which aggregate specific gravity deviates from 2.7, correct the concrete weight by 100 lb (45.4 kg) in the same direction.

Source: From ACI Committee 211.[7]

if the moisture content of coarse aggregate is 2%, the amount of coarse aggregate needed for the batch should be increased by 2%. The actual amount of water, which is 2% of the weight of the coarse aggregate, should be subtracted from the water to be used for the mix. The reduction in water is necessary to maintain the water–cement ratio chosen in step 5. Similar adjustments should also be made for fine aggregate.

10. *Trial batch adjustments:* Since the estimation of the various ingredients is only approximate, adjustments are needed to obtain the mix that satisfies the workability and strength requirements. Fresh concrete should be tested for slump, segregation of aggregates, air content, and unit weight. The hardened concrete, cured under standard conditions, should be tested for strength at the specified age. The test methods are described in Sec. 3.3. In some instances it may take several trials to obtain a satisfactory mix. The following guidelines may be used for the adjustment of ingredients. The recommended numerical values are for 1 ft^3 (0.028 m^3) of concrete.

- If the slump of the trial mix is not correct, increase or decrease the estimated water by 10 lb (4.5 kg) for each 1-in (25-mm) required increase or decrease of slump.
- If the desired air content is not achieved, adjust the admixture content. Since the amount of air content influences the slump, the water content should also be changed with the change in air-entraining admixture. Change the water content by 5 lb (2.3 kg) for each 1% of air.
- Adjust the yield by using the unit weight of fresh concrete.

The following example further illustrates the mix proportioning process.

Example 3.2 To compute the mix proportions of normal-weight concrete, we use these specifications:

Required (28-day) average compressive strength f'_c	4100 psi (28 MPa)
Type of construction	Reinforced concrete footing
Slump	Minimum 3 in (75 mm)
Exposure condition	Below ground; no freezing, no exposure to chemicals
Maximum size of aggregate	1.5 in (38 mm)

Solution

1. *Background information on properties of constituent materials:*

Cement	ASTM type I; bulk density 196 lb/ft^3 (3136 kg/m^3)
Coarse aggregate	Maximum size 1.5 in (38 mm)
	Bulk density 168 lb/ft^3 (2690 kg/m^3)
	Dry-rodded unit weight 100 lb/ft^3 (1600 kg/m^3)
	Moisture content 2.0% over SSD condition
Fine aggregate	Bulk density 160 lb/ft^3 (2560 kg/m^3)
	Fineness modulus 2.8
	Moisture content 3.0% over SSD condition

2. *Selection of slump:*

$$\text{Specified minimum} = 3 \text{ in (75 mm)}$$

The specified minimum value is consistent with the 3 in (75 mm) recommended for reinforced concrete foundation walls and footings in Table 3.7.

3. *Maximum aggregate size:*

$$\text{Specified maximum} = 1.5 \text{ in (38 mm)}$$

4. *Estimation of amount of water and air:* Since there is no freezing or exposure to chemicals, non-air-entrained concrete is assumed to be adequate. Using Table 3.8, the amount of water for 1 yd^3 (0.76 m^3) = 300 lb for 3-to 4-in slump (136 kg^3 for 75- to 100-mm slump).
5. *Selection of water–cement ratio:* Using Table 3.9, the water–cement ratio for the required average strength is 0.56. The value of 0.56 is obtained by interpolating linearly between 4000 and 5000 psi (27 and 34 MPa). Note that this value is only a first estimate.
6. *Cement content:*

$$\text{Cement content} = \frac{\text{amount of water}}{\text{water–cement ratio}}$$

$$= \frac{300}{0.56} = 536 \text{ lb/yd}^3 \text{ (316 kg/m}^3\text{)}$$

7. *Coarse aggregate content:* Using Table 3.11:

Volume fraction of dry-rodded aggregate for 1.5-in maximum-size aggregate and a sand fineness modulus of 2.8	0.71
Volume of coarse aggregate per cubic yard of concrete	0.71×27 ft^3 = 19.2 ft^3 (0.54 m^3)
Weight of coarse aggregate (since dry-rodded unit weight is 100 lb/ft^3 or 1600 kg/m^3)	19.2×100 1920 lb/yd^3 (1133 kg/m^3)

8. *Estimation of fine aggregate:*

Estimated unit weight of concrete, from Table 3.12	4070 lb/yd^3 (2401 kg/m^3)
Weight of cement (step 6)	536 lb/yd^3 (316 kg/m^3)

Weight of coarse aggregate (step 7)	1920 lb/yd^3 (1133 kg/m^3)
Weight of water (step 4)	300 lb/yd^3 (177 kg/m^3)
Weight of fine aggregate	4070 – (536 + 1920 + 300) 1314 lb/yd^3 (775 kg/m^3)

9. *Adjustments for aggregate moisture:* The moisture content of coarse aggregate above SSD is 2%. In order to obtain 1920 lb of dry-rodded weight, we have to use 1.02 × 1920 lb of stock sample.

Weight of stock sample of coarse aggregate	1.02 × 1920 lb/yd^3 = 1958 lb/yd^3 (1155 kg/m^3)
Weight of stock sample of fine aggregate	1.03 × 1314 lb/yd^3 = 1353 lb/yd^3 (798 kg/m^3)
Free water from coarse aggregate	1958 – 1920 = 38 lb/yd^3 (22 kg/m^3)
Free water from fine aggregate	1353 – 1314 = 39 lb/yd^3 (23 kg/m^3)
Water to be added	300 – 39 – 38 223 lb/yd^3 (132 kg/m^3)

The final weights of the constituent materials per cubic yard (0.76 m^3) are:

Cement	536 lb (243 kg)
Fine aggregate	1353 lb (614 kg)
Coarse aggregate	1958 lb (888 kg)
Water	223 lb (101 kg)
Total	4070 lb (1846 kg)

Note that the unit weight of concrete is the same even after the corrections are made for the moisture contents of aggregates.

10. *Trial batches:* Trial batches should be made using about 0.1 yd^3 (0.076 m^3) of concrete to determine the properties of fresh and hardened concrete. If the slump is too high or too low, corrections can be made immediately. Since it takes 28 days to obtain strength results, it may be more efficient to make two or three trial mixes with various water contents rather than waiting for the results from a single mix.

3.3 QUALITY ASSURANCE

Quality assurance procedures are needed in order to ascertain that the concrete placed in the actual structures satisfies the required specifications. As mentioned earlier, the quality of the concrete is influenced by a large number of variables. Hence continuous monitoring of properties is needed. The quality and physical properties of the constituent materials, namely, cement, aggregates, water, and admixtures, should also be checked periodically. ASTM standards are available for the test procedures needed to evaluate the properties of constituent materials and concrete. The properties of constituent materials were discussed in Sec. 3.1. This section deals with the test methods for fresh and hardened concrete and the quality assurance procedures. The frequency of testing depends primarily on the type of structure. For buildings, samples should be taken at least once a day or once for every 150 yd^3 (115 m^3) of concrete. If large surface areas are being constructed, the ACI code recommends at least one test for each 5000 ft^2 (464 m^2). For structures such as nuclear containment buildings where failure could result in disastrous consequences, more stringent quality control is needed.

3.3.1 Tests for Fresh Concrete

The most universally used test for fresh concrete is the slump test. It measures only the consistency of concrete. However, this test is used as an indicator of workability. The slump test is also used to assure the uniformity from batch to batch. The other two tests used for fresh concrete are the V-B test and the compaction factor test. These two tests are used primarily in the laboratory environment, whereas the slump test is used both in the laboratory and in the field.

Slump Test

The slump test is covered in ASTM C143. In this test a sample of freshly mixed concrete is placed inside a mold, which has the shape of the frustrum of a cone. The concrete is compacted using a standard procedure and the mold is raised to allow the concrete to slump. The amount by which the concrete slumps is measured in inches or millimeters and is called the slump value. The following are the pertinent details of this test.

The test equipment consists of (1) mold (Fig. 3.5), (2) a tamping rod, (3) a ruler with a reading accuracy of at least 0.125 in (3 mm), and (4) a nonabsorbent rigid pan. The mold shown in Fig. 3.5 should be clean and free of any projections such as rivets or dents on the inside surface. The top and bottom faces should be parallel to each other and perpendicular to the vertical axis. The tamping rod consists of a ⅝-in (16-mm)-diameter and 24-in (610-mm)-long steel bar with hemispherically rounded ends.

The test procedure consists of the following steps:

1. Dampen the mold and place it on a rigid, flat surface. The surface should be moist and nonabsorbent. Stand on the two foot pieces in order to hold the mold in place during the filling operation.
2. Pour concrete until the mold is filled to one-third of the volume. One third of the volume is reached when the mold is filled to a height of 2⅝ in (66 mm). Rod this bottom layer of concrete with 25 strokes of the tamping rod. Apply approximately half of the strokes near the perimeter and progress spirally toward the center. The bottom layer should be rodded throughout its depth.
3. Pour concrete to fill the mold to two-thirds of its volume. To reach this volume, the mold should be filled to a height of 6⅛ in (155 mm) from the bottom. Rod this layer using 25 strokes. The strokes should just penetrate into the underlying bottom layer.
4. Fill the remaining part of the mold, which forms the top layer. Again rod this top layer with 25 uniformly distributed strokes. The top level of concrete should stay slightly above the top surface of the mold. When concrete subsides below the top surface because of compaction, add more concrete. The strokes of the tamping rod should just penetrate into the second layer.

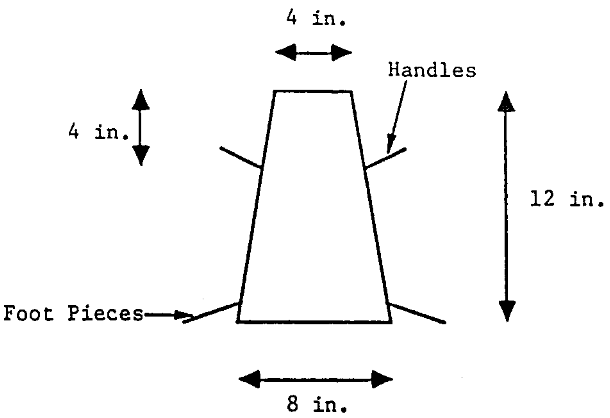


FIGURE 3.5 Salient features and dimensions of slump cone.

5. Strike off the excess concrete on the top using the tamping rod. A screeding and rolling motion should be used to obtain a relatively smooth top surface.
6. Remove the mold by lifting it in a vertical direction. Lateral or torsional motion should be avoided. The lifting operation should be complete in 5 ±2 s. The entire operation of filling and lifting the mold should be completed without interruption in 2.5 mm.
7. When the mold is removed, the concrete will slump down. If the concrete is stiff, it might move only a fraction of an inch. If the concrete is of flowing consistency, the whole mass will collapse. Measure the difference between the original 12 in (300 mm) and the height of the slumped concrete and record it to the nearest 0.25 in (6 mm). This value is called the slump of the concrete. If the concrete slides to one side, the slump value should be disregarded and the test repeated.

V-B Test

The equipment for the V-B test consists of a vibration table, a cylindrical pan, and a glass or plastic disk, Fig. 3.6. The glass or plastic disk is attached to a free-moving rod which serves as a reference end point. The cone is placed in the cylindrical pan and filled with concrete. The cone is removed, the disk is brought into position on the top of the concrete, and the vibrating table is set into motion. When the table starts vibrating, the concrete in the conical shape remolds itself into a cylinder. The time required for remolding the concrete into cylindrical shape, until the disk is completely covered with concrete, is recorded in seconds and reported as V-B time. Since the V-B test is conducted using vibration, the V-B time is a better indicator of workability when in the actual construction the concrete is compacted using vibration. However, this test is not easily adoptable for testing on the construction site.

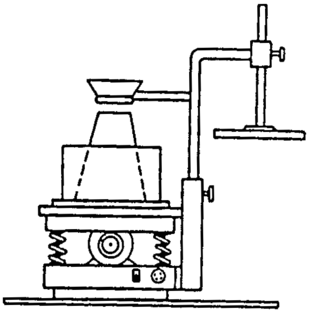


FIGURE 3.6 V-B apparatus.

Compaction Factor Test

This test measures the amount of compaction achieved when the concrete mixture is subjected to a standard amount of compacting work. The result is expressed as a compacting factor, which is a ratio of the density actually achieved in the test to the density of the same concrete under fully compacted conditions. This test, which was developed in Britain, is not as popular as the other two tests.

The three tests mentioned primarily provide an indication of the workability of concrete. In the case of air-entrained concrete the air content of fresh concrete has to be measured to assure that the concrete has the specified amount. Other properties measured using fresh concrete include unit weight, setting time, and concrete temperature. The air content can be determined using either the pressure or the volumetric method. These tests are described in ASTM C23 1 and C 173, respectively. The gravimetric method is covered in ASTM C 138. Only the volumetric method should be used for lightweight concrete. The unit weight, setting time, and temperature measurements are covered in ASTM C 138, C403, and C 1064, respectively.

3.3.2 Tests for Hardened Concrete

Hardened concrete should meet the minimum-strength requirements and should be durable. The compressive strength test is the most widely used quality control test. Various types of theoretical and empirical relationships have been developed to relate the compressive strength to other properties of concrete so that other tests need not be conducted for every situation. However, test methods exist for determining strength at various modes of loading, such as tension, flexure, shear, and torsion, and to ascertain durability under freeze-thaw conditions. The most commonly used tests and the corresponding ASTM standards are as follows:

Compressive strength of cylindrical concrete specimens (ASTM C39)

Flexural strength of concrete (ASTM C78, C293)

Splitting tensile strength (ASTM C496)

Modulus of elasticity and Poisson's ratio (ASTM C469)

Length change of hardened paste or mortar (ASTM C 157)

Creep under compression (ASTM C512)

Air void parameter (ASTM C457)

Resistance of concrete to rapid freezing and thawing (ASTM C666)

Resistance to scaling (ASTM C672)

Only the compressive strength test is described here. The details for the other tests can be found in the appropriate ASTM standards.

3.3.3 Compressive Strength Test

The compressive strength test can be conducted using cylinders, cubes, or prisms. In the United States, 6- by 12-in (150- by 300-mm) cylinders are the most popular test specimens for obtaining the compressive strength. In recent years 4- by 8-in (100- by 200-mm) cylinders have also been used, especially for high-strength concrete. In special circumstances smaller [3- by 6-in (75- by 150-mm)] and larger [(up to 24- by 48-in (600- by 1200-mm)] cylinders are also being be used. The cylinders are typically made in the laboratory for trial mixes. The cylinders made for quality control are usually made on the construction site. In either case, standard procedures should be followed for the casting, curing, capping, and testing of cylinders. This section presents the salient features of the procedure to be used for the preparation and testing of cylinders in the laboratory. For field cast and cured specimens appropriate ASTM standards should be followed for the sampling, making, and curing of cylinders.

Mixing, Molding, and Curing of Concrete Test Specimens

The concrete can be mixed either by hand or by machine. Hand mixing should be avoided as much as possible because it is very difficult to achieve uniform mixing. At least 10% more concrete than needed for molding should be mixed. Hand mixing, which is not to be used for air-entrained and no-slump concrete, and quantities exceeding 0.25 ft^3 (0.007 m^3) can be achieved by using the following steps:

1. Mix cement and fine aggregate in a watertight, clean, and damp metallic pan until the contents are thoroughly blended.
2. Add the coarse aggregate to the cement-sand mix and mix until the coarse aggregates are uniformly dispersed.
3. Add water and the admixtures, if any, and mix the contents until they become a homogeneous mass.

The mixing sequence for machine mixing is as follows. It is assumed that a drum-type mixer is used for mixing.

1. Place coarse aggregate and half the mixing water in the mixer and mix the contents for 30 s.
2. Add fine aggregate, cement, and the remaining water and mix for 3 min.
3. Stop the mixer and rest the mixture for 3 min.
4. Mix for additional min.

If admixtures are used, they should be mixed with water before starting the mixing process. The open end of the mixer should be covered with a pan to avoid evaporation. Typically, some concrete sticks to the sides of the drum. This concrete contains more mortar than the discharged concrete. Hence the concrete used for making samples could contain less mortar. This is particularly true when small quantities are mixed. This could be avoided by mixing a similar batch of concrete and disposing of the contents. The test batch is then mixed without cleaning the mixer. This process is called buttering the mixer. If the exact amount of mortar that sticks to the sides can be established, the mix can be adjusted to contain this excess mortar. This process is more difficult because it is not easy to establish the accurate amount of mortar that will stick to the sides.

Molding of specimens should be done in a place that is very near to the storing place where the specimens will be kept for the first 24 h. Molds should be made of a material which is nonabsorbent and nonreactive with concrete. They should be dimensionally stable and watertight. Reusable molds should be coated lightly with mineral oil for easy removal of the specimens.

The mixed concrete is placed in the mold in layers and compacted to form the test specimens. For a cylinder height of 12 in (300 mm) or smaller, casting should be done in three equal layers if compaction is done by rodding. If compaction is done by vibration, two layers should be used. For larger cylinders, more layers may be needed. If the slump is greater than 3 in (75 mm), compaction by rodding is preferable. If the slump is less than 1 in (25 mm), vibration should be used for compaction. If the slump is in the range of I to 3 in (25 to 75 mm), either rodding or vibration can be used. If the cylinder diameter is 4 in (100 mm) or less, only external vibration should be used. For larger cylinders, either internal or external vibration can be used.

The following points should be observed when compaction is done by rodding.

For 3-by 6-or 4-by 8-in (75- by 150- or 100- by 200-mm) cylinders, use a ⅛-in (9-mm)-diameter 12-in (300-mm)-long metal rod with hemispherical ends. For 6- by 12-in (150- by 300-mm) cylinders use ⅝-in (15-mm)-diameter 12-in (300-mm)-long metal rod with hemispherical ends.

Each layer should be rodded 25 strokes, uniformly distributed over the cross section. The bottom layer should be rodded throughout the depth. While rodding the upper layers, allow the rod to penetrate about ½ in (12 mm) into the underlying layer for 3-by 6- and 4-by 8-in (75- by 150- and 100- by 200-mm) cylinders and about 1 in (25 mm) for 6- by 12-in (150- by 300-mm) cylinders.

After rodding each layer, tap the outside of the mold about 15 times with a rubber mallet to close any holes left by the tamping rod and to release large entrained air bubbles. Spade the top of the concrete lightly before placing the subsequent layer.

If the compaction is done by vibration, fill the mold in a number of layers of equal height and vibrate them. Place all the concrete for each layer before starting the vibrating equipment. When adding the final layer, do not overfill more than 0.25 in (6 mm). The duration of vibration required depends on the workability of the concrete and the effectiveness of the vibration. Compaction can be assumed to be complete if the top surface is smooth. Overvibration should be avoided. Each layer should be vibrated to the same extent. When using an internal vibrator, use three insertions for each layer. Allow the vibrator to penetrate through the layer being vibrated and approximately 1 in (25 mm) into the underlying layer. The vibrator should be pulled out slowly so as to avoid air pockets. After vibrating each layer, tap the side 10 to 15 times with a rubber mallet to release entrapped air bubbles. When external vibration is used, ensure that the mold is held securely against the vibrating surface.

At the end of consolidation, strike off the top surface with a trowel. Flatten the top surface such that it is level with the rim of the mold and has no depressions or projections larger than 0.125 in (3 mm). The top surface of the freshly made cylinder may be capped with a stiff cement paste.

After finishing, cover the specimens with a nonabsorptive and nonreactive plate or an impervious plastic sheet. Remove the specimens from the mold 24 ± 8 h after casting. Cure the specimens at 73 ± 3°F (23 ± 2°C) from the time of removal until testing. Curing can be done by immersing the specimens in lime-saturated water or in a room maintained at 100% relative humidity by using moist sprays.

Capping of Cylindrical Specimens

The cylinders should be capped before testing to assure two flat surfaces that are perpendicular to the axis of the cylinders (ASTM C617). The capping material should be at least as strong as the concrete being tested. Freshly made specimens can be capped with cement paste. This is not usually done because it is very difficult to achieve the required accuracy. Normally capping is done after the cylinders are cured. Common capping materials are high-strength gypsum cement or molten sulfur. Standard equipment is available for melting the sulfur and for the alignment of caps so that they are perpendicular to the axis of the cylinder.

For high-strength concrete, with compressive strength greater than 12,000 psi (80 MPa), grinding of the ends is recommended. Grinding prevents the interference of the capping materials.

The tolerance for level surfaces is 0.002 in (0.05 mm). The caps should be about 0.125 in (3 mm) thick. They should not be more than 0.31 in.(8 mm) thick. Gypsum plaster should cure at least for 4 h prior to testing. The minimum curing period for sulfur caps is 2 h.

Test Apparatus

The test apparatus consists of a testing machine, scale, and calipers. If the stress-strain relationship is measured, then a compressometer setup is also needed. Most of the testing machines are driven by hydraulic fluid pressure. These machines can be operated so as to apply the load at a constant rate at a certain number of psi per minute or at a constant displacement rate. Some of the machines with servo control mechanisms can be run under different controls such as displacement or strain control.

A compressometer setup can be used to measure the deformation and, hence, the strain at a given load. This device is needed to obtain the stress-strain relationship and the Young's modulus of elasticity. The setup consists of two yokes (rings) (Fig. 3.7). The bottom ring (yoke) is attached to the cylinder using three screws. The top ring is attached to the cylinder using two screws placed at diametrically opposite points. This ring can rotate about the two screws. One end of the rotatable ring is connected to the bottom ring using a pivot rod. The pivot rod does not allow any translation but allows the ring to rotate. A dial gauge or other deformation-measuring instrument, such as an LVDT, is attached on the other side (Fig. 3.7). When the cylinder deforms, the two points where the screws of the top ring are attached move downward. Since one end cannot move due to the pivot rod, the ring rotates. Due to symmetry, if the deformation along the gauge length is S, the dial gauge will record 2S, hence improving the accuracy of the measurements, specially for small deformations. The rings are placed in proper position using spacing bars. Normally the gauge length for 6- by 12-in (150- by 300-mm) cylinders is 6 in (150 mm).

Specimen Preparation

As mentioned earlier, the cylinders should be capped so that the ends are flat and perpendicular to the axis of the cylinder. The diameter of one cylinder should not vary by more than 2% of that of the companion cylinder. If the difference is more than 2%, the cylinders should be discarded. The diameter should be measured to the nearest of 0.01 in (0.25 mm) by averaging two diameters measured at

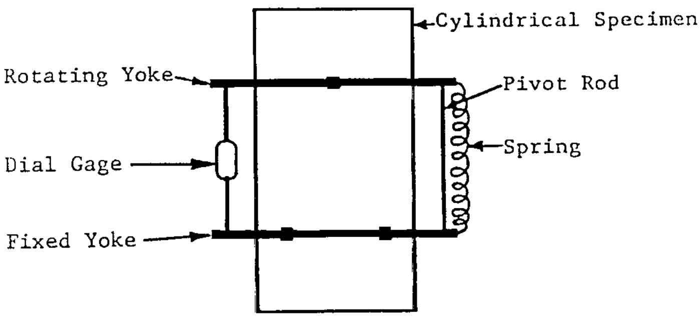


FIGURE 3.7 Compressometer arrangement.

right angles to each other at about midheight of the specimen. If the length-to-diameter ratio is less than 1.8 or more than 2.2, or if the height is used to compute the volume of the cylinder, the height should be measured to the nearest of "0.05 diameter." The test specimen should be in saturated surface-dry condition. The specimens should be tested at the specified age. The following are the ASTM permissible tolerances:

Test age, days	Permissible tolerance, hours
1	±0.5
3	±2.0
7	±6.0
28	±20.0
90	±48.0

Test Procedure

1. Clean the upper and lower bearing blocks and place the specimen on the lower block. Carefully align the axis of the cylinder with the center of thrust of the spherically seated upper block. Most testing machines have concentric circles marked on the bearing plates, and hence centering is not a difficult task.
2. Start the machine and raise the lower block so that the top block comes in contact with the cylinder. Once the top and bottom plates are touching the cylinder, lock the top plate to prevent its rotation.
3. Start applying load without shock. The loading should be a continuous process. For hydraulically operated machines the loading rate should be in the range of 20 to 50 psi/s (138 to 344 kPa/s). The loading rate should be constant. Adjustments should not be made, even when the specimen begins to fail.
4. Record the maximum load, type of failure, and appearance of the concrete. Five types of failures shown in Fig. 3.8, cover most of the failure modes.
5. Compute the compressive strength by dividing the maximum load by the area of cross section. If the length-to-diameter ratio is less than 1.8, apply the following correction factors. Cylinders with lower length-to-diameter ratios resist more loads due to a different strain distribution along the length.

Length/diameter	1.75	1.50	1.25	1.00
Correction factor	0.98	0.96	0.93	0.87

The values for other length-to-diameter ratios can be interpolated. These correction factors are applicable for normal-strength concrete with strengths in the range of 2000 to 6000 psi (14 to 42 MPa) and lightweight concrete with densities in the range of 100 to 120 lb/ft^3 (1600 to 1920kg/m^3). Report the compressive strength to the nearest 10 psi (0.1 MPa).

To obtain the modulus of elasticity the following additional steps are needed.

6. Prior to testing for the modulus of elasticity determine the compressive strength of concrete.
7. Attach the compressometer to the cylinder and place the specimen in the machine.
8. Load the specimen at a rate of 35 ± 5 psi/s (240 ± 34 kPa/s) to approximately 40% of the ultimate load. If the companion cylinders are not available, load the cylinders until the longitudinal strain reaches a value shown in Table 3.13. Immediately upon reaching the designated load, reduce the load to zero at the same rate at which it was applied.
9. Reload the specimen and record loads and deformations at predefined intervals. A sufficient

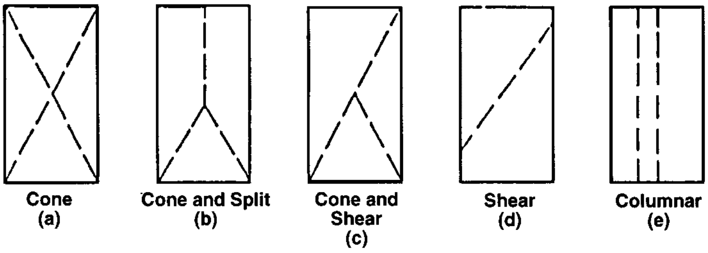


FIGURE 3.8 Types of fractures.

number of readings should be taken to establish the stress-strain relationship. Load the specimen up to about 50% of its capacity. Reduce the load to zero and repeat the measurements. If the two sets of measurements are the same, then proceed to compute the modulus E_c. If there is a significant difference between the two sets, an additional set of readings should be taken.

10. After obtaining a consistent set of load-deformation information, remove the compressometer and test the cylinder to failure.
11. Compute the stress by dividing the load by the area and the strain by dividing the deformation by the gauge length. Plot the stress-strain curve. The curve should be approximately linear, at least up to 40% of the failure load.
12. Compute E_c using the equation

$$E_c = \frac{S_2 - S_1}{\epsilon_2 - 0.00005} \tag{3.10}$$

where E_c = modulus of elasticity
S_1 = stress corresponding to a longitudinal strain of 0.00005
S_2 = stress corresponding to 40% of ultimate load
ϵ_2 = longitudinal strain corresponding to S_2.

Report the modulus of elasticity to the nearest 50,000 psi (0.5 GPa).

TABLE 3.13 Maximum Strain Values

Strain at age indicated × 10^{-6}			
Unit weight at time of test, lb/ft^3 (kg/m^3)		7 days or more	Less than 7 days
205	(3282) and over	300	200
165–204	(2642–3266)	375	250
135–164	(2161–2626)	450	300
115–134	(1841–2145)	525	350
105–114	(1681–1825)	600	400
95–104	(1521–1665)	675	450
85–94	(1361–1505)	750	500
75–84	(1201–1345)	825	550

3.3.4 Statistical Variation of Compressive Strengths and Quality Control

Due to the influence of various factors, there is always a variation in the strengths of concrete. The variations occur in all modes of loading, including compression, splitting tension, flexure, shear, and torsion. In most construction practices the variations of compressive strength are used for quality control. Hence only compressive strength variations and their tracking procedures are discussed here. In some instances involving pavements, flexural strengths are measured and used for quality assurance. The statistical procedure used for compressive strength can also be used for flexural strength.

As mentioned in Sec. 3.2.1, the variation of compressive strength is usually assumed to follow a normal distribution. The properties of the normal curve, sample average, and sample standard deviation are used to compute the required average compressive strength to satisfy the specified strength requirements. Hence at the beginning of a project, the strength level of the concrete being produced is based on the calculation of the required average strength. This hypothetical production strength is based on the assumption that the variables affecting the strength of concrete will be the same in the future as they have been in the past. As the data become available, the average of the actual production strength will replace the hypothetical average strength and the standard deviation. If the average and the standard deviation obtained during the project are about the same as the values used in the computation, the project average strength should be carefully maintained.

If the project average strength is smaller than the required average strength while the standard deviation is the same, the percentage of tests below the specified strength will be greater than the acceptable value, and steps must be taken to increase the average strength of the concrete. The average strength should also be increased if the standard deviation of the project is greater than the assumed standard deviation. On the other hand, if the project average is higher, or if the standard deviation is lower; the average strength could be reduced.

Continuous evaluation of the data is necessary in order to assure that the concrete being placed satisfies the specified strength requirements. An updated determination of the average strength and the standard deviation will provide an indication of how well the quality control procedures are working. At any given time the approximate percentage of tests falling below the specified strength can be calculated using the equation

$$p = \frac{\bar{x} - f'_c}{S} \tag{3.11}$$

where p = probability factor
$\bar{x}$ = average strength
f'_c = specified strength
S = standard deviation

Note that Eq. (3. 11) is a transformed version of Eq. (3. 1). The variables have been changed to permit using the project average strength x and the project standard deviation S. In Eq. (3. 11), if p = 2.33, the probability of the cylinder strength (average of two cylinders) falling below f'_c is 1%. Probabilities for other values can be estimated using statistical tables in statistics books or tables provided in the report of ACI Committee 214.[5]

In order to satisfy the equations of ACT code 3 18-92 [Eqs. (3.2) and (3.3)], the project average and the standard deviation should satisfy the following two inequalities:

$$1.34 < \frac{\bar{x} - f'_c}{S} \quad \text{or} \quad \frac{\bar{x} - f'_c}{S} \geq 1.34 \tag{3.12}$$

$$2.33 < \frac{\bar{x} - (f'_c - 500)}{S} \quad \text{or} \quad \frac{\bar{x} - (f'_c - 500)}{S} \geq 2.33 \tag{3.13}$$

Quality control charts are quite often used for a visual picture of concrete performance. Three typical quality control charts used in the industry are shown in Fig. 3.9. Various forms of other charts are also being used. With the advent of tabletop computers it is very easy to develop, maintain, and update these charts. The charts can also be transferred from location to location using phone lines.

Figure 3.9(*a*) shows the variations of (1) the individual cylinder strength, (2) the average of two cylinders, and (3) specified and required average strengths. The number of low tests can be easily picked out from this chart. Note that the number of low tests is computed using the average of two cylinders (solid line). If the volume of concrete produced requires more than one test per day, the average of all the tests (instead of two) can be plotted for that day. The charts can also be plotted using calendar dates.

Figure 3.9(*b*) and (*c*) is plotted using the values of Fig. 3.9(*a*). Each point in Fig. 3.9(b) represents the average of the previous five tests. The number of tests used to calculate this moving average depends on the type of job and the number of tests per day. In Fig. 3.9(*b*) some of the high variabilities of individual tests are suppressed. This chart can be used to identify the influence of major factors such as seasonal changes and changes in materials. Figure 3.9(*c*) shows the moving average range of the previous 10 groups of cylinders. Considerable change in this chart is an indication of high variability.

The control charts are valuable tools not only for the current project, but also for future projects. As discussed earlier, good records can be used for the computation of f'_{cr} and mix proportions instead of trial mixes, thus saving a considerable amount of time and effort.

The variability caused by the test procedure is always a concern in quality control. It is always advisable to separate the variability caused by the testing procedure from variabilities caused by other factors such as change in material properties because the variability in testing does not represent a variability in the strength of the concrete used in the actual construction. The following procedure can be used to estimate the magnitude of variation due to testing.

A test consists of all the cylinders made under identical conditions. The cylinders should be made using the same sample of concrete, cured at the same conditions, and tested at the same age. If

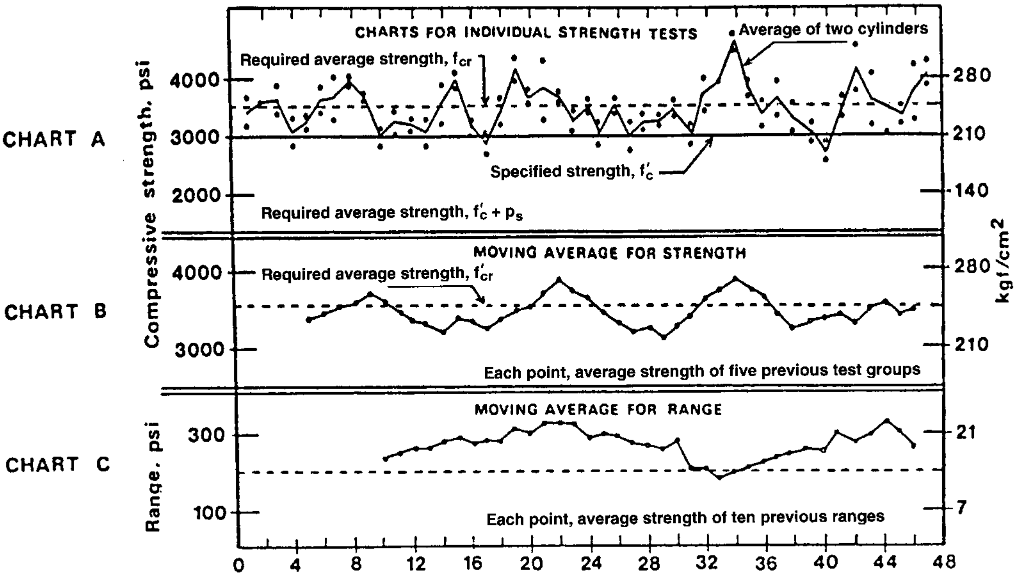


FIGURE 3.9 Quality control charts for concrete production and evaluation. (A) Individual strength tests. (B) Moving average for strength. (C) Moving average for range. (*From ACI Committee 214.*[11])

it is assumed that two or more test cylinders made from the same sample of concrete and tested at the same age should have the same strength, variations in the strengths of these cylinders can be attributed to the testing procedures. However, since differences in casting and curing could also make a difference, only a major part (and not 100%) of the variation can be attributed to testing. Differences between cylinders cast from the same sample are called within-test variations. The within-test standard deviation S_{wt}, can be calculated using the equation

$$S_{wt} = \frac{R}{d_2} \tag{3.14}$$

where R = average range for all tests of a class of concrete
d_2 = factor based on number of cylinders within test

The values of d_2 are 1.128, 1.693, and 2.059 for two, three, and four cylinders, respectively. The range is the difference between the highest and lowest values of strength.

The within-test coefficient of variation V_{wt} can be computed using the equation

$$V_{wt} = \frac{S_{wt}}{\bar{x}} \times 100 \tag{3.15}$$

where $\bar{x}$ is the average strength for the class of concrete.

If V_{wt} is less than 1.5%, the field control testing can be considered excellent. If the value is greater than 4%, the within-test variation should be considered as being poor, and errors in testing may be a major contributing factor to strength variation. If S_{wt} is between 1.5 and 2.0, 2 and 3, or 3 and 4, the testing performance is considered very good, good, or fair, respectively.

3.3.5 Accelerated Strength Tests

In modern-day construction, large volumes of concrete are placed in a single day. In some cases, such as slip-formed construction, it is possible to complete a substantial portion of a structure in a single day. For example, in the case of the CN Communication Tower in Toronto, Canada, the slip-formed construction procedure was used to complete almost 20 ft/day (6 in/day). Therefore one cannot wait 28 days to ascertain the strength. If the strength were found to be unsatisfactory after 28 days, hundreds of feet of structure would have to be taken down. Accelerated strength test procedures were developed for use in such situations. Using these procedures potential 28-day strengths can be estimated in 1 or 2 days. The four procedures, procedure A (warm-water method), procedure B (boiling-water method), procedure C (autogenous curing method), and procedure D (high-temperature and -pressure method), are recognized in ASTM C 684. This section deals with brief descriptions of these methods and their use in quality assurance.

Warm-Water Method (Procedure A)

In this method the cylinders are placed in warm water right after casting. The specimens are cured in their molds in the water maintained at 95 ± 5°F (35 ± 3°C) for a period of 23.5 ± 0.5 h. After this curing period of about 24 h the cylinders are capped and tested to determine their compressive strengths. An extensive study conducted by the U.S. Corps of Engineers established that this is a reliable method for routine quality control of concrete. The primary limitation of this method is that the strength gain is not substantial as compared to 24-h moist cured samples.

Boiling-Water Method (Procedure B)

In this method the cylinders are stored at 70 ± l0°F (21 ± 5°C) for the first 23 ± 0.25 h and then placed in boiling water for a period of 3.5 h ±5 mm. The specimens are allowed to cool for 1 h and tested. The strength increase provided by this type of accelerated curing is much higher than for the warm-water method, and hence the specimen can be transported to the laboratory site without being damaged. This method is the most commonly used one among the four methods.

Autogenous Method (Procedure C)

In this procedure the accelerated curing effect is obtained by using the heat of hydration. The cylinders are placed in an insulated container right after casting to retain the heat generated by hydration. The cylinders are tested after curing for 48 ± 0.25 h and a rest period of 30 mm at room temperature. The strength gain obtained in this method is lower than that obtained by the boiling-water method. This procedure was found to be less accurate than procedures A and B. However, it was used successfully in the CN tower in Toronto, Canada. The project, which was completed in 1974, involved the placement of about 51,000 yd^3 (39,000 m^3) of concrete.

High-Temperature and -Pressure Method (Procedure D)

This procedure is limited to concrete containing aggregates smaller than 1 in (25 mm). Wet sieving can be used for concrete containing larger aggregates. Sealed 3- by 6-in (75- by 150-mm) cylinders are cured at a temperature of 300 ± 5°F (149 ± 3°C) and a pressure of 1500 ± 25 psi (10.3 ± 17 MPa) for a period of 5 h ± 5 min. The curing process starts right after casting. In most cases capping is not required because of the presence of end plates and external pressure. Hence the specimens can be tested within 15 min after curing. For specimens that need capping, testing is done after 30 min. This procedure needs sophisticated equipment and hence is more expensive compared to the three other procedures.

A summary of all four procedures is presented in Table 3.14. This table shows the curing medium, the temperature and duration of curing, and the age at testing for all four procedures.

3.3.6 Quality Control Using Accelerated Strength Tests

The most important use of accelerated test data is quality control. These tests permit rapid adjustment of batching and mixing. In the current practice, the accelerated strength results are used to estimate 28-day strengths because of the traditional use of 28-day strength for design purposes. A correlation between the chosen accelerated strength and the 28-day strength should be established before starting the project. The mix proportions and materials used for developing the correlation equation should be the same as the materials and mix proportions to be used for the project.

ACI Committee 214[12] recommends a minimum of 30 data sets for establishing the correlation between the 28-day strength and the accelerated strength. The 28-day strength range should include

TABLE 3.14 Accelerated Curing Procedures

Procedure	Molds	Accelerated curing medium	Curing begins	Duration of curing	Age at testing
A Warm water	Reusable or single use	Warm water, 950F (35°C)	Immediately after casting	23½ h ± 30 in	24 h ± 15 min
B Boiling water	Reusable or single use	Boiling water	23 h ± 15 mm after casting	3½ h ± 5 min	28½ h ± 15 min
C Autogenous	Single use	Heat of hydration	Immediately after casting	48 h ± 15 min	49 h ± 15 min
D High temperature and pressure	Reusable	External heat and pressure, 300°F (149°C)	Immediately after casting	5 h ± 5 mm	s5¼ h ± S mm

Source: From ASTM C654, 1993.[2]

the specified strength and should not fall below 75% of the specified strength. The 28-day strength is normally expressed as a linear function of accelerated strength using the equation

$$y = ax + b \tag{3.16}$$

where y = 28-day strength
x = accelerated strength
a, b = constants

The constants a and b are obtained using statistical correlation. The correlation coefficient should be at least 0.8. A typical correlation curve is shown in Fig. 3.10. The 95% confidence limits in Fig. 3.10 show the variations of 28-day strength that can be expected 95% of the time. For example, if the accelerated strength is 3000 psi (21 MPa), the expected 28-day strength is about 5700 psi (40 MPa). Of the 28-day strengths 95% can be expected to be in the range of 4800 to 6500 psi (33 to 45 MPa). An interpretation of the accelerated strength test results and their use for quality assurance follows.

Interpretation of Test Results

Accelerated strength test results can be interpreted using the same procedures as those used for 28-day strength results (Sec. 3.3.4). The required average strength f'_{cr} can still be computed using the equation

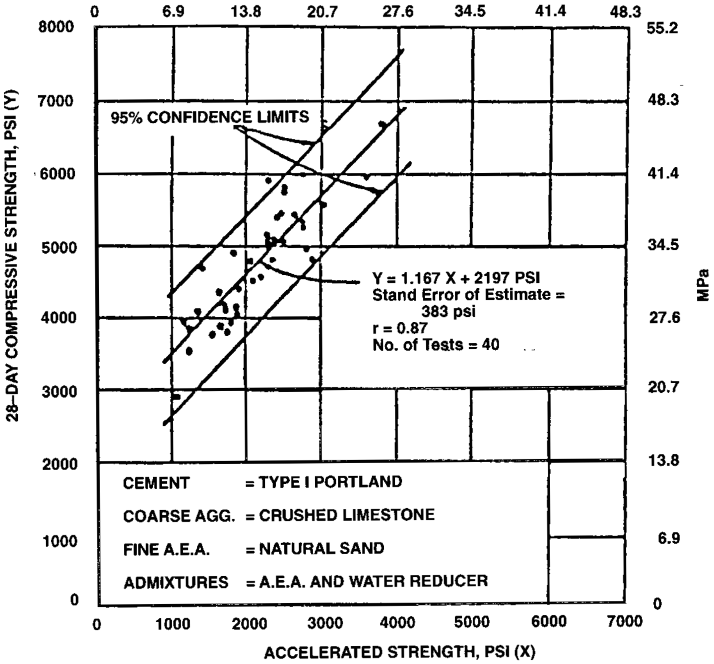


FIGURE 3.10 Relationship between accelerated and 28-day compressive strengths of concrete—Data obtained using boiling-water method (ASTM procedure B). (*From ACI Committee 214.*[12])

$$f'_{cr} = f'_c + t\sigma \tag{3.17}$$

where f'_c = specified design strength
t = constant depending on proportion of tests that may fall below f'_{cc} (Table 3.15).
σ = a standard deviation of data set used for prediction of f'_c

Table 3.15 presents the t values for a number of low test values ranging from 1 in 2 to 1 in 1000. Permissible low tests should be chosen primarily based on the type of structure, as explained earlier.

The required average strength can be established based on either specified accelerated strength or specified 28-day strength and the correlation equation between accelerated and 28-day strengths. The following examples illustrate the computation procedure.

Example 3.3 The specifications require an accelerated strength of 2000 psi (13.8 MPa). Compute the required average (accelerated) strength f'_{cr} if the acceptable number of low tests is 1 in 100. The standard deviation from past records for accelerated strength tests is 500 psi (3.4 MPa).

Solution From Table 3.15, the value oft for 1 in 100 low tests is 2.33

$$\begin{aligned} f'_{cr} &= f'_c + t\sigma \\ &= 2000 + 2.33(500) \\ &= 3165 \text{ psi } (21.8 \text{ MPa}) \end{aligned}$$

Note that f'_{cr}, f'_c, and σ correspond to accelerated strengths.

Example 3.4 The specifications require a 28-day compressive strength of 4500 psi (31 MPa). Compute the required accelerated average strength using the following information. The relationship between 28-day strength y and accelerated strength x is

$$y = 1.167x + 2197$$

The acceptable number of low tests is 1 in 10, and the standard deviation for accelerated strengths is 410 psi (2.83 MPa)

Solution The specified 28-day strength is 4500 psi (31 MPa). Using the correlation equation, the corresponding accelerated strength is

TABLE 3.15 Values oft for the Equation $f'_{cr} = f'_c + t$

Number	Likelihood of low test results, %	t
1 in 1000	0.1	3.09
1 in 500	0.2	2.88
1 in 100	1.0	2.33
1 in 50	2.0	2.06
1 in 25	4.0	1.75
1 in 20	5.0	1.65
1 in 10	10.0	1.28
1 in 5	20.0	0.84
1 in 2	50.0	0.00

Source: From ACI Committee 214.[12]

$$x = \frac{y - 2197}{1.167}$$

$$= \frac{4500 - 2197}{1.167} = 1973 \text{ psi (13.6 Mpa)}$$

For 1 low test in 10, $t = 1.28$,

$$(f'_{cr})_{\text{accelerated}} = 1973 + 1.28(410) = 2498 \text{ psi}$$

Hence the required average accelerated strength is 2500 psi (17 MPa).

If the number of pairs of data used for the regression line relating accelerated and 28-day strengths is less than 30, special statistical procedures can be used for the prediction of f'_{cr}. The procedure can be found in the report of ACI Committee 214.[12] The number of pairs should be at least 10.

3.3.7 Nondestructive Tests

Nondestructive tests are valuable tools for evaluating the properties of in situ concrete. These methods can be used to estimate the strength, durability, or elastic properties of concrete. In addition they can also be used to estimate the location and condition of reinforcement, and for locating cracks, large voids, and moisture content. The test methods can be classified as:

Surface hardness methods

Penetration resistance techniques

Pull-out tests

Ultrasonic pulse velocity method

Maturity concepts

Electromagnetic methods

Acoustical methods

These methods are described briefly in this section.

Surface Hardness Methods

In surface hardness methods the hardness of the surface measured, using the size of the indentation or the amount of rebound, is taken as an indicator of the strength of concrete. The most popular method in this category is the rebound hammer test. This test is also known as Schmidt rebound hammer, impact hammer, or sclerometer test. The rebound hammer, shown in Fig. 3.11, consists of a spring-loaded mass and a plunger. When the plunger is pressed against the concrete, it retracts against the force imparted by a spring. When the spring is retracted to a certain position, it releases automatically. Upon release the mass rebounds, taking a rider with it along a guide scale. The distance traveled by the mass, expressed as a percentage of the initial extension of the spring, is called the rebound number. ASTM C805 covers the procedure for conducting the rebound hammer test.

The rebound number, which is a measure of the hardness of the concrete surface, can be empirically related to the compressive strength of concrete. However, in certain circumstances the surface hardness may not represent the strength of the concrete inside the structure. For example, the presence of a large aggregate immediately underneath the plunger would result in a large rebound number. A large void underneath the plunger, on the other hand, would provide an unusually low rebound number. Other factors that influence the rebound number include the type of aggregate, smoothness of the surface, moisture condition, size and age of the specimen, degree of carbonation,

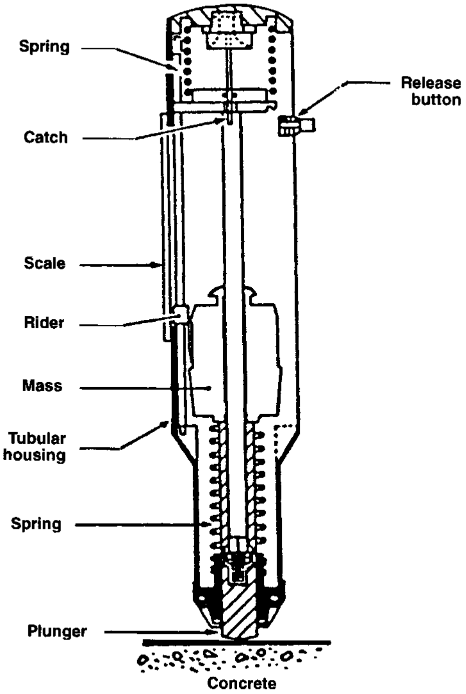


FIGURE 3.11 Major components of rebound hammer.

and position of the hammer (vertical versus inclined or horizontal). The hammer should be used only against a smooth surface. Troweled surfaces should be rubbed smooth using a carborundum stone. If the concrete being tested is not part of a large mass, it has to be supported so that the specimen does not move during the impact. Concrete in the dry state tends to record a higher rebound number. Because of gravity, the rebound number of floor concrete would be smaller than that of the soffit concrete, even though both concretes are similar. The rebound number of inclined and vertical surfaces would fall somewhere in between.

The rebound hammer is best utilized for checking the uniformity of concrete in a large structure or for comparing the quality of concrete in similar structural components such as precast beams. It can also be used for estimating the strength of concrete that is being cured for the purpose of removing formwork. If the rebound numbers have to be used for estimating strength, correlation should be established between the rebound number and the compressive strength for each type of concrete used on a site. If mix proportions or constituent materials are changed, then a new set of tests should be conducted and a new correlation equation obtained.

Typically there is a large variation in rebound numbers. At least 10 to 12 readings should be taken in each location. ASTM C805 provides guidelines for averaging the rebound numbers. In certain cases some individual readings might have to be omitted from the average. If proper calibration is used, the accuracy of prediction of concrete strength is about ±20% for laboratory specimens and ±25% for in situ concrete.

Penetration Resistance Techniques

In this method the penetration resistance of concrete is used as the indicator of its strength. The most commonly used test is the Windsor probe test. In this test a hardened alloy probe is fired (or driven) by a driver using a standard charge of powder. The exposed length of the probe is taken as a measure of penetration resistance. It is assumed that the compressive strength of concrete is proportional to its penetration resistance. Here again the hardness of the aggregates plays an important role. A correlation has to be developed for a particular concrete if this method is to be used for predicting strengths.

The probes are driven in sets of three in close vicinity, and the average value is used for estimating the strength. The test procedure is covered in ASTM C803. This test is more expensive than the rebound hammer test, but much less expensive than core tests. This test, which is considered to be more accurate than the rebound hammer test because the measurement is not made just on the surface, is also an excellent tool for determining the uniformity of concrete and the relative rate of strength gain at early age for the purpose of removing formwork.

Pull-Out Tests

In a standard pull-out test the concrete strength is estimated using the force required to pull out a specially shaped steel insert whose enlarged end has been cast into the fresh concrete. The specifications of the test are covered in ASTM C900. Because of the shape of the insert, a lump of concrete, in the shape of a frustrum of a cone, is pulled out along with the insert. In most cases the fracture occurs at about a 45° angle. Even though the failure occurs due to tension and shear, the strength computed using an idealized area of the frustrum was found to be approximately equal to the shear strength of concrete. An approximate linear correlation seems to exist between pull-out strength and compressive strength. The ratio of pull-out strength to compressive strength decreases slightly for concrete with higher compressive strengths.

The pull-out test is more accurate than the penetration or the rebound hammer test because it is based on the actual failure load. However, this test, which is more involved, has to be planned in advance, and the damaged area has to be repaired. In some cases, such as estimation of strength for form removal, pulling out the assembly may not be necessary. The test can be stopped when a predetermined force is reached, assuring sufficient strength for removing the forms.

Other forms of tests similar to the standard pull-out test are still being developed. Those test methods include the pull-out of inserts placed in drilled holes, pulling out a wedge anchor using torque, and break-off tests. In the break-off test the flexural strength of concrete is determined by applying a transverse force on the cylinder created by inserting a tube in the fresh concrete. Inserts placed in drilled holes can be used for existing structures. But their success is still to be established.

Ultrasonic Pulse Velocity Method

In this method the longitudinal wave velocity in concrete is used to estimate the compressive strength and check the uniformity of concrete. An exciter is used to initiate a pulse which is picked up at another designated location. The distance between the two locations divided by the time required for the travel is the pulse velocity. Correlation relationships have been developed between pulse velocity, compressive strength, and modulus of elasticity. The relationships are affected by a number of variables, such as the moisture condition of the specimen, type and volume fraction of the aggregate, water–cement ratio, and age of specimen. In general, pulse velocity alone cannot be used to estimate the strength, unless a correlation exists for the particular type of concrete being tested. But the method is an excellent tool for quality control measures. The test method is covered in ASTM C597. The pulse velocity method can be used for both laboratory and field tests. In field tests, the presence of reinforcement and the vibration of elements being tested (such as piers under a roadway in use) might pose problems. A combination of rebound hammer and pulse velocity methods is sometimes used to evaluate both the surface characteristics and the quality of concrete located well inside the surface.

Maturity Concepts

It is well established that the strength development of concrete depends on duration, curing temperature, and pressure. If moist curing is done at atmospheric pressure, then the combination of duration and curing temperature could be used to estimate the maturity and hence the compressive strength. A number of relationships relating strength and maturity exist in the published literature. Maturity meters are also available for use in the field and the laboratory. This concept can be used to estimate the early strength of structural members for form removal. The correlation between maturity and strength should be established before starting the actual project.

Electromagnetic and Acoustical Methods

Various forms of magnetic, electrical, and acoustical techniques have been tried to determine the properties of concrete such as dynamic modulus, presence of cracks, honeycombing, and measurement of cover to reinforcement. These methods have not attained common acceptance so far.

3.A.3.8 Core Tests

If there is a reason to believe that the concrete in place may not have the specified strength, nondestructive tests discussed in the previous section can be used to determine the uniformity. If the location in question behaves very similar to other locations, the quality of the concrete could be satisfactory. However, if the tests indicate variability, cores might have to be taken to determine the strength. The procedure for evaluation using cores is covered in ASTM C42.

Typically, cores are drilled using diamond drills. A number of factors should be considered in evaluating core strengths. The following are some of the important points:

- Typically core strengths are lower than standard cylinder strengths. The differences could be more significant for high-strength concrete.
- The strength of the core could depend on its position in the structure. Cores taken near the top of a structural element are typically weaker than cores taken from the bottom.
- Cores taken from very thick sections could contain microcracks due to excessive heat of hydration and hence register lower strengths.
- The presence of large aggregates in small cores could result in erroneous strengths.

3.4 MECHANICAL PROPERTIES

Strength, stiffness, and dimensional stability constitute the core of the mechanical properties. Strength can be measured under various modes of loading such as compression, tension, flexure, shear, and torsion. This section deals with these basic mechanical properties of concrete. Properties such as the durability of concrete exposed to various chemicals or to freezing and thawing, and permeability are also very important for some structures. These properties are not covered in this book for lack of space. The reader is referred to the literature.

3.4.1 Compressive Strength

Compressive strength is the most commonly used design parameter for concrete. In most cases the concrete is specified using its compressive strength measured at 28 days. In some instances 56-day strength or minimum early strength is specified. Up to around 1960, the compressive strength of concrete was limited to about 6000 psi (42 MPa). The advent of new admixtures as well as mixing, placing, and compacting techniques led to the development of higher strengths. Concrete with a specified compressive strength of 12,000 psi (84 MPa) was used in Water Tower Place in Chicago, which was topped off in 1972. The development of high-range water-reducing admixtures (superplasticizers) resulted in routine use of high-strength concrete. Concrete used in a Seattle building in the late 1980s had an average strength in excess of 20,000 psi (135 MPa). This section deals with the various factors that affect the compressive strength of concrete. An understanding of the influence of the various factors is needed for effectively proportioning, making, and casting concrete in the field.

The major factors that influence compressive strength are water–cement ratio; aggregate-cement ratio; maximum size of aggregate; grading, surface texture, shape, strength, and stiffness of aggregate particles; degree of compaction; curing conditions; and testing parameters. The admixtures used can also influence the strength by improving workability and through better compaction.

Water-Cement Ratio

The relation between compressive strength and water–cement ratio was established in 1918 by Duff Abrams. For a fully compacted concrete, he found that the strength f'_c can be expressed as

$$f'_c = \frac{k_1}{k_2 w/c} \tag{3.18}$$

where w/c = water–cement ratio

k_1. k_2 = empirical constants

If the amount of air voids does not exceed 1% by volume, the concrete can be considered as fully compacted concrete. Typical variations of compressive strength and the influence of compaction are shown in Fig. 3.12. From this figure it can be seen that compaction plays an important role. The use of admixtures that improved workability led to better compaction, resulting in higher and higher strengths in the 1980s. Typical variations of compressive strength for water–cement ratios ranging

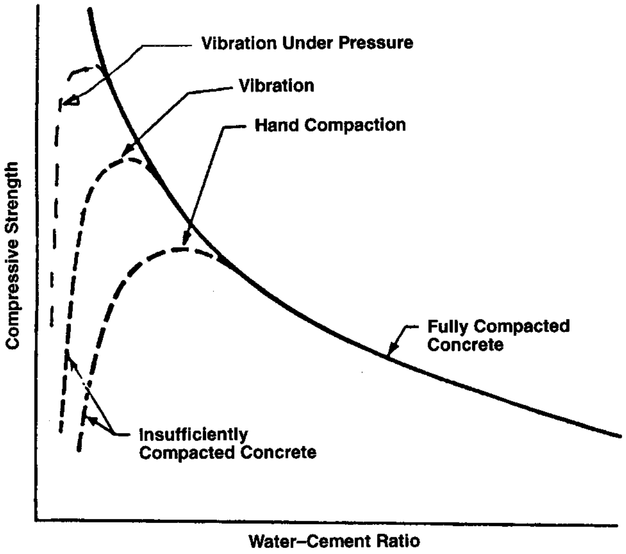


FIGURE 3.12 Relationship between strength and water–cement ratio of concrete.

from 0.35 to 1.2 are shown in Fig. 3.13. In field construction the commonly used water–cement ratios vary from 0.25 to 0.65. As mentioned in Sec. 3.2, the water–cement ratio should be restricted to 0.4 for obtaining durable concrete. If the water–cement ratio is less than 0.4, in most cases some form of admixture is needed for obtaining a workable concrete.

The amount of voids present in concrete controls its strength. In concrete with a high water- cement ratio, the excess water results in more voids and hence lower strength. Improperly compacted concrete also has higher voids, resulting in low strength. Hence a balanced approach should be used in mix proportioning. A water–cement ratio of about 0.4 is needed for complete hydration of the cement. However, complete hydration of cement does not produce the highest strength. The presence of unhydrated cement as inert particles was found to provide better strength. In most cases the lower limit for the water–cement ratio is 0.28. However, water–cement ratios lower than 0.28 have been used with superplasticizers and other admixtures for producing very high-strength concrete.

Aggregate-Cement Ratio

The aggregate-cement ratio affects the strength of concrete if the strength 15 about 5000 psi (35 MPa) or more. The influence of the aggregate-cement ratio is not as significant as that of the water–cement ratio, but it has been found that for a constant water–cement ratio, leaner mixes provide higher strengths, as shown in Fig. 3.14. The increase in strength could be due to absorption of water by the aggregate and hence a lower effective water–cement ratio. In addition leaner mixes have lower amounts of total water and paste content and hence lower amounts of voids.

A more recent study indicates that the strength increases with an increase in the cement content if the volume of aggregates is less than 40%. But the trend reverses in the aggregate volume ratios of 40 to 80% (Fig. 3.15).

Cement Type and Age

The degree of cement hydration determines the porosity of the hydrated cement paste and hence the compressive strength. Under standard curing conditions type III cement hydrates faster than type I

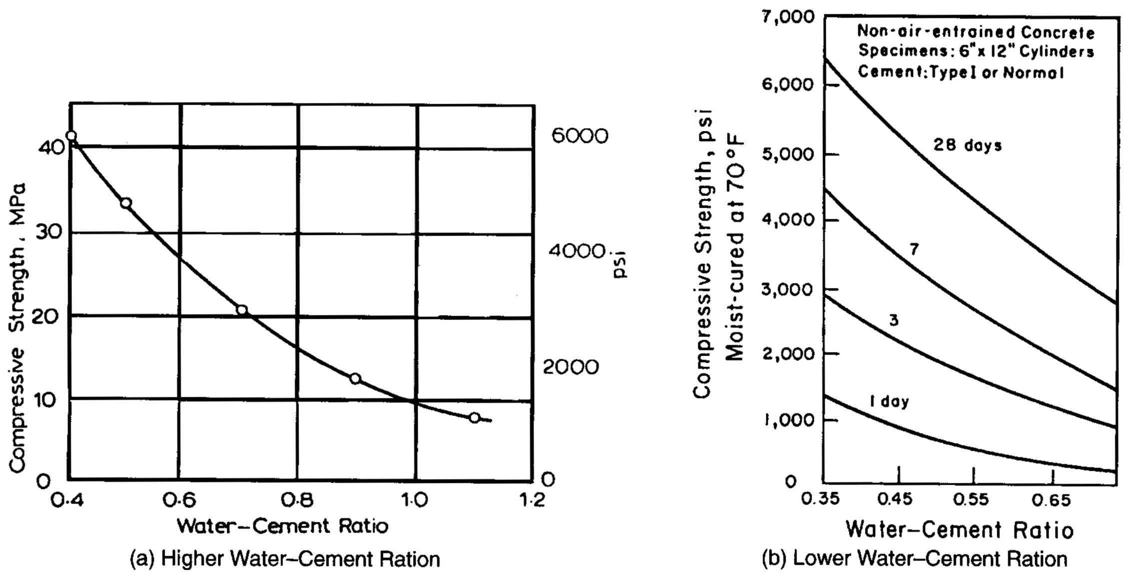


FIGURE 3.13 Influence of water–cement ratio and moist curing on concrete strengths. (a) Higher water–cement ratio. (b) Lower water–cement ratio. (*From PCA.*[8])

cement. Hence at early ages type III cement provides higher strengths. The variations in strength, for type I and III cements at 1, 3, 7, and 28 days are shown in Fig. 3.16. The bands shown in this figure cover the majority of the data obtained in the laboratories. Relative strength gains for three water–cement ratios are shown in Fig. 3.17. From this figure it can be seen that early strength gain is higher for lower water–cement ratios. Empirical relationships are available in the literature for predicting 28-day strengths based on the results of 1-, 3-, or 7-day strengths and vice versa.

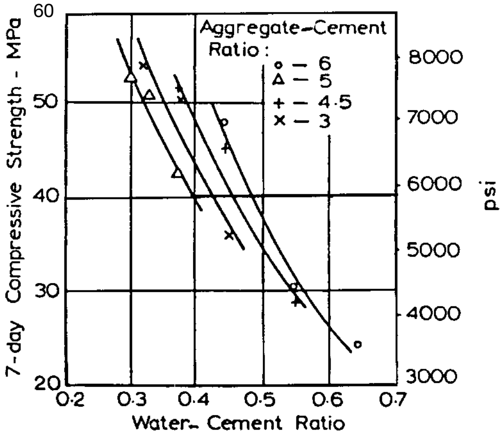


FIGURE 3.14 Influence of aggregate-cement ratio on strength of concrete. (*From Singh.*[13])

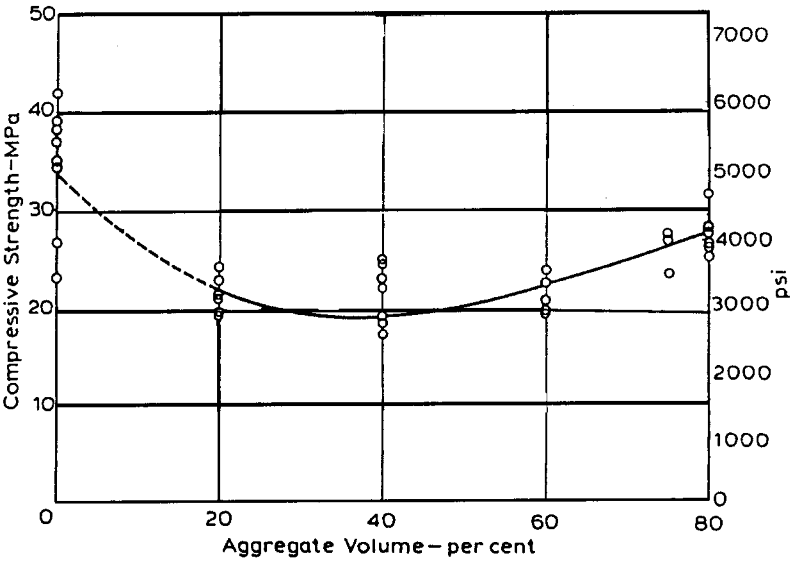


FIGURE 3.15 Influence of aggregate content on strength. (*From Stock et al.*[14])

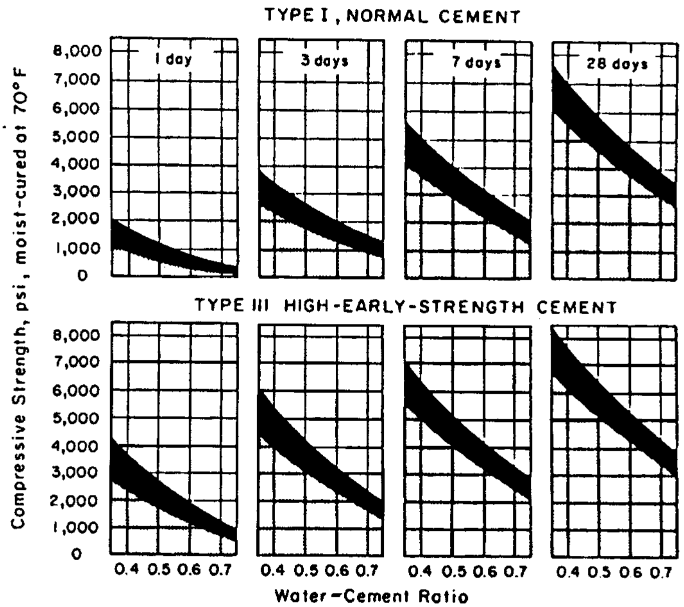


FIGURE 3.16 Influence of water–cement ratio, duration of moist curing. and cement type on strength. (*From PCA.*[8])

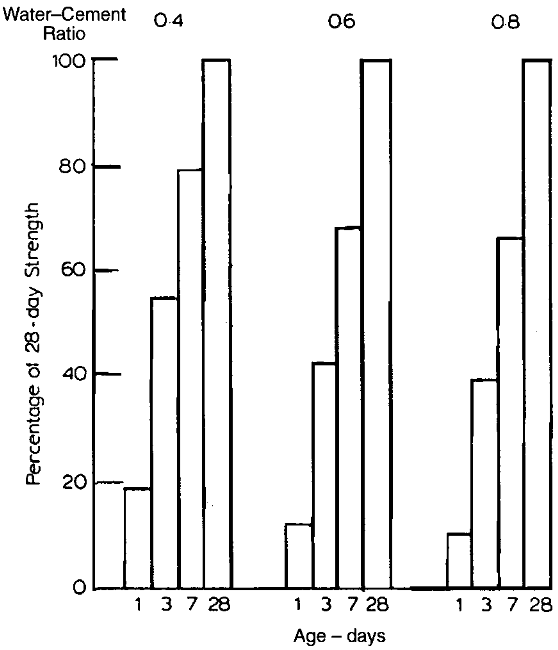


FIGURE 3.17 Relative strength gain of concrete with time for different water–cement ratios.

At normal temperatures ASTM type II, IV, and V portland cements hydrate at a slower rate, and hence concrete containing these cements will have lower early strengths. The results presented in Table 3.16 show typical relative strengths at 1, 7, and 28 days for various cements. After 90 days the variation in strength between cement types is negligible.

Coarse Aggregate

Typically, aggregates are stronger than the matrix and hence do not control the fracture strength. However, factors such as aggregate size, shape, surface texture, and mineralogy can influence the

TABLE 3.16 Approximate Relative Strength of Concrete as Affected by Type of Cement

Type of portland cement		Compressive strength, % of strength of type I*		
		1 day	7 days	28 days
I	Normal or general-purpose	100	100	100
II	Moderate heat of hydration and moderate sulfate resisting	75	85	90
III	High early strength	190	120	110
IV	Low heat of hydration	55	65	75
V	Sulfate resisting	65	75	85

*Compressive strength is 100% for all cements at 90 days.
Source: From PCA.[8]

workability, degree of compaction, and formation of gel around the aggregate. Consequently it can affect the compressive strength.

The maximum size of aggregate has considerable influence, especially at low water–cement ratios, as shown in Fig. 3.18. Concrete with large-size aggregates requires less water, and hence the same water–cement ratio provides better compaction, resulting in higher strength. On the other hand, larger sizes provide weaker transition zones around the aggregate, resulting in lower strength. These opposing influences are water–cement ratio dependent and provide more pronounced effects at lower water–cement ratios. In most cases the strength can be expected to go down with an increase in the maximum size of the aggregate.

If water–cement ratio and maximum size of aggregate are kept constant, aggregate grading influences the consistency of the concrete and hence the strength. An increase in fines typically increases the water demand, and if the amount of water is not increased, consistency decreases. The decrease in strength was found to be as high as 12%.

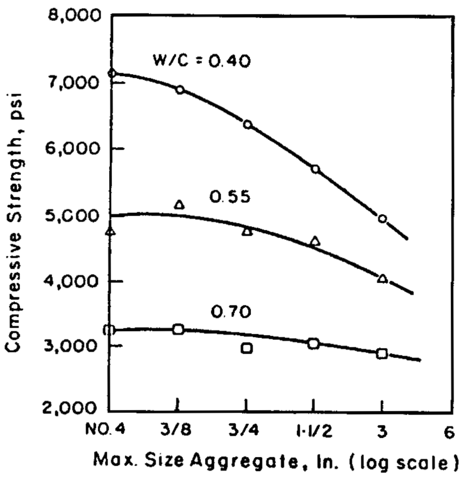


FIGURE 3.18 Influence of aggregate size and water–cement ratio on strength. (*From Cordon and Gillespie.*[15])

Aggregates with rough texture were found to result in early high strengths as compared to aggregates with smooth textures. The rough surface provides a better bond between matrix and aggregate, especially at early stages when the hydration is not complete. At later stages the influence diminishes. Aggregates with smooth surfaces are easier to work with and hence provide a better final product.

The mineralogical composition of aggregates was also found to influence the concrete strength. Calcareous aggregates tend to provide better strength than siliceous aggregates. Fig. 3.19 shows the compressive strengths obtained using various types of aggregates. It can be seen that the strength variation could be as high as 50%.

Air Content

Air is entrained in concrete to improve its durability. The air bubbles tend to improve the workability of fresh concrete and hence improve the compaction. But the presence of air bubbles in the hardened concrete increases its porosity and reduces the density of the composite. Hence air entrainment leads to a decrease in compressive strength.

The decrease in strength due to air entrainment was found to depend on both the water–cement ratio and the cement content. As the water–cement ratio decreases, the strength loss increases. Hence the strength loss is considerable for high-strength concrete. When the cement content is reduced, the influence of the air content also decreases. In fact the air content improves the strength slightly if the cement content is very low. In the normal strength range, about a 2% loss in compressive strength can be expected for each 1% increase in air content.

The air content was found to improve the workability of lightweight concrete. This is particularly true for mixes of low paste content.

Curing Conditions

Concrete should be moist cured for at least 28 days to obtain best results. Premature drying can reduce the strength by more than 50%. In terms of curing, the major factors are time, humidity, temperature, and pressure.

A longer curing time under moist conditions always provides better results. A 100% relative humidity is the best condition. This can be achieved by pooling water, placing wet burlaps, or making

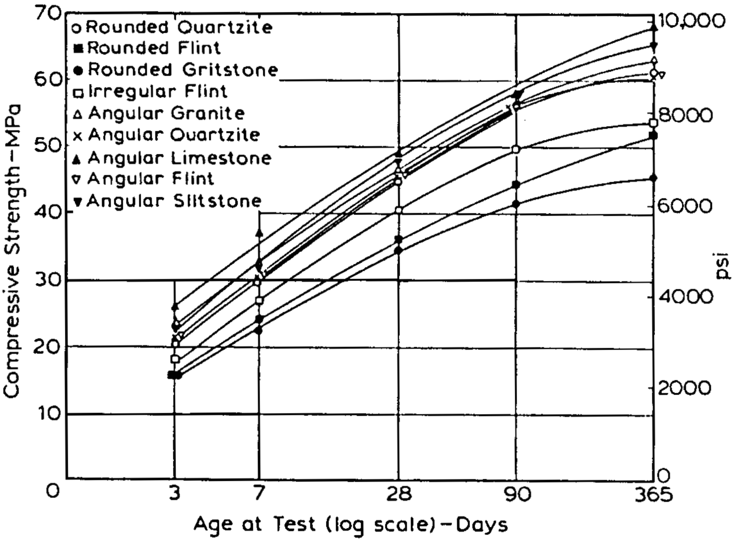


FIGURE 3.19 Influence of aggregate type on strength. (*From a report by the Building Research Station, London, 1969.*)

other arrangements. Figure 3.20 shows the influence of moist curing. It can be seen that 3 days of moist curing provides 50% improvement over air-dried samples. It is preferable to moist cure for 28 days. In any circumstance, the moist curing should be done for at least 7 days.

Typically higher temperatures provide faster curing. At about 12°F (–11°C) the cement stops hydrating. Hence at this temperature there may not be any increase in strength, even after long periods of time. Early age strength increase can be accelerated by using warm water. Temperatures higher than 100°F (380C) are not normally used because the increase in acceleration beyond this tempera-

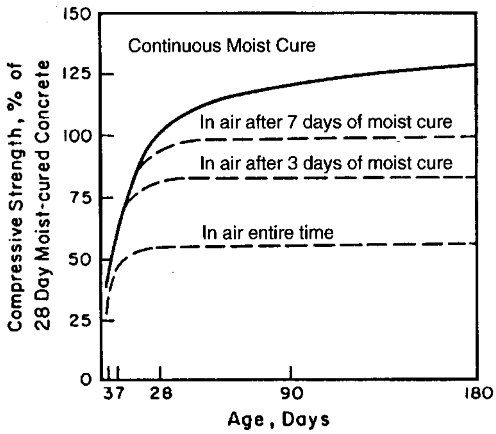


FIGURE 3.20 Influence of curing conditions on strength. (*From PCA.*[8])

ture is not high enough for economic reasons and the initial high temperature could result in lower long-term strength. However, precast elements are sometimes steam cured in order to reduce turnaround time and speed up reuse of forms.

Higher pressure typically accelerates the curing. But the increase is not significant for commercial applications. Hence pressure curing is used only for steam curing.

Test Conditions

In North America 6-by 12-in (150- by 300-mm) cylinders are the standard test specimens, even though 4- by 8-in (100- by 200-mm) cylinders are sometimes used for high-strength concrete. Other types of specimens include cubes and prisms. In the case of cylinders and prisms, the minimum length-to-diameter ratio (or longer dimension versus lateral dimension) should be at least 2 to avoid end effects. If specimens with lower length-to-diameter ratios are used, correction factors should be applied for determining the compressive strength. ASTM specifications provide guidelines, as explained in Sec. 3.2. Cylinders tend to record lower strengths than cubes. The ratio of cylinder to cube strength is about 0.85. Larger samples tend to record lower strengths as compared to smaller samples. Other test conditions that influence strength include end conditions, rate of loading, moisture condition of the specimen, and condition of the plattens of the machine.

Cylinders should be capped properly, making sure that the ends are plane, parallel to each other, and perpendicular to the longitudinal axis. The testing machine should have capability to allow slight rotation of the ends. The machine should also be calibrated periodically to avoid errors in the measurement of loads.

3.4.2 Tensile Strength

The tensile strength can be measured using uniaxial tension specimens or cylinders. Uniaxial tension specimens are very difficult to test because special gripping devices are needed. The most common practice is to test a 6- by 12-in (150- by 300-mm) cylinder along the longitudinal axis. This test is called the splitting tension test, and the strength value obtained is the splitting tensile strength.

There is a relationship between compressive and tensile strength. But the relationship is not well established. Tensile strength is about 10% of compressive strength for normal-strength concrete. As the compressive strength increases, the ratio decreases. Table 3.17 presents typical tensile strength values for compressive strengths varying from 1000 to 9000 psi (7 to 60 MPa). A number of factors, including type and shape of coarse aggregate, properties of fine aggregate, grading of aggregate, air

TABLE 3.17 Relation between Compressive, Flexural, and Tensile Strengths of Concrete

Strength of concrete, psi (MPa)			Ratio, %		
Compressive strength	Modulus of rupture	Splitting tensile strength	Modulus of rupture to compressive strength	Splitting tensile strength to compressive strength	Splitting tensile strength to modulus of rupture
1000 (6.9)	230 (1.6)	110 (0.8)	23.0	11.0	48
2000 (13.8)	375 (2.6)	200 (1.4)	18.8	10.0	53
3000 (20.7)	485 (3.3)	275 (1.9)	16.2	9.2	57
4000 (27.6)	580 (4.0)	340 (2.3)	14.5	8.5	59
5000 (34.5)	675 (4.7)	400 (2.8)	13.5	8.0	59
6000 (41.3)	765 (5.3)	460 (3.2)	12.8	7.7	60
7000 (48.2)	855 (5.9)	520 (3.6)	12.2	7.4	61
8000 (55.1)	930 (6.4)	580 (4.0)	11.6	7.2	62
9000 (62.0)	1010 (7.0)	630 (4.3)	11.2	7.0	63

Source: From Price.[16]

entrainment, and age at testing, affect the ratio between tensile and compressive strengths. A number of empirical relations are available for estimating the tensile strength. Most of these equations are of the form

$$f_t = k(f'_c)^n \tag{3.19}$$

where f'_c = compressive strength
f_t = tensile strength
k, n = empirical constants

Concretes containing lightweight aggregate typically have lower tensile strengths.

3.4.3 Modulus of Rupture

The flexural strength of concrete is called modulus of rupture. The strength values are determined using 4- by 4- by 14- or 6- by 6-by 20-in (100- by 100- by 350- or 150- by 150- by 550-mm) prisms subjected to four-point loading. When analyzing reinforced concrete beams and slabs for flexural loading, the modulus of rupture is used to compute cracking load and deflection. Since the beams are in the flexure mode, the modulus of rupture is more representative than splitting or direct tensile strengths. Typical values of flexural strength are presented in Table 3.17.

For design purposes, the modulus of rupture f_r can be estimated using the following equation:

$$f_r = 7.5\sqrt{f'_c} \tag{3.20}$$

In most cases this equation provides a conservative estimate. For high-strength concrete, other forms of equations have been proposed. Nevertheless, Eq. (3.20) provides a good estimate for design purposes. For lightweight concrete a constant smaller than 7.5 should be used.

Most factors that influence the ratio of tensile to compressive strength also influence the modulus of rupture. The magnitude of influence is slightly lower for flexural strength.

3.4.4 Shear Strength

Concrete is seldom subjected to pure shear. But when structural members are loaded under various modes, the concrete in those members could be subjected to a shear force. It is extremely difficult to measure direct shear strength. Torsion or deep beam specimens can be used to measure shear strength indirectly.

Equations for the prediction of shear strength are not well established. Most researchers agree that the shear strength is proportional to the square root of compressive strength. But the constant of proportionality is not established. For beams subjected to bending and shear, the ACI code allows a shear stress of $4\sqrt{f'_c}$ for uncracked sections. However, this value is useful only for beams subjected to shear. A number of researchers have tested specimens under torsion to establish the shear strength. But the results are still inconclusive in terms of having a single equation for predicting shear strength.

3.4.5 Modulus of Elasticity

The modulus of elasticity of a material is the slope of the initial linear portion of the stress-strain curve. Since the stress-strain curve of concrete is nonlinear, there are three types of moduli, namely, the tangent, the secant, and the chord modulus. The secant modulus is the most commonly used parameter for design purposes. It is defined as the slope of the line joining the origin and the point on the stress-strain curve corresponding to 40% of compressive strength. For high-strength concrete the initial portion of the curve is almost linear, and hence the tangent and the secant moduli are same. Since measurement of the modulus is a more involved process, its value is normally estimated using the compressive strength and the unit weight of concrete.

The ACI code recommends the following equations for normal-strength concrete[4]:

$$E_c = 33\, W_c^{1.5}\sqrt{f'_c} \tag{3.21}$$

where E_c = secant modulus
E_c = unit weight of concrete
f'_c = compressive strength of concrete

For normal-weight concrete which has a unit weight of about 145 lb/ft^3 (2321 kg/m^3) E_c can be computed using the following equation:

$$E_c = 57{,}000\sqrt{f'_c} \tag{3.22}$$

For high-strength concrete the aforementioned equations were found to overestimate the value of the modulus. The equation recommended for high-strength concrete is

$$E_c = (40{,}000\sqrt{f'_c} + 10^6)\left(\frac{W_c}{145}\right)^{1.5} \tag{3.23}$$

A number of other equations are also available in the literature. The primary factors that influence the modulus of elasticity are aggregate type, amount of paste, mineral admixtures such as fly ash and silica fume, and moisture conditions of the specimen. Tests should be conducted carefully using ASTM procedures. The modulus value is very sensitive to test conditions such as rate of loading.

3.4.6 Shrinkage

Shrinkage is a phenomenon in which concrete shrinks under no load due to the movement and evaporation of water. There are two types of shrinkage known, plastic and drying shrinkage. Plastic shrinkage occurs during the initial and final setting times, extending to several hours after the initial placement of concrete. Major factors that affect plastic shrinkage are area of exposed surface, surface and concrete temperature, and surface air velocity.

Drying shrinkage occurs in hardened concrete, and it can continue for up to 2 years. It occurs due to the evaporation of water and the movement of water within, resulting in a more compact matrix.

If concrete is immersed in water, a small amount of expansion known as swelling can be observed. However, the expansion cannot completely reverse the shrinkage. Shrinkage is a time-dependent process. In the first few months it is much larger than at later ages. A number of factors influence the shrinkage. These factors are briefly discussed in the following. More details can be found in the literature dealing with concrete.

Aggregate Type and Volume Fraction

The aggregates restrain the shrinkage of cement paste because they do not shrink. Hence the concrete with lower cement content has less shrinkage. In addition the elastic properties and other characteristics of the aggregate also influence the shrinkage because the amount of restraint proivided by the aggregate depends on its elastic modulus and how much force it can transmit along the interface. Typical shrinkage values for different aggregate-cement and water–cement ratios are shown in

Table 3.18. From this table it can be seen that shrinkage can be reduced by as much as four times by increasing the aggregate content.

In terms of type of aggregates, quartz provides the least amount of shrinkage. Aggregates that produce more shrinkage in progressive order are limestone, granite, basalt, gravel, and sandstone. Lightweight aggregate concretes typically shrink more than normal-weight concrete.

Water-Cement Ratio

As the water–cement ratio increases, shrinkage increases, as shown in Table 3.18. This should be expected because more water leads to more evaporation and movement. Water-cement ratios higher than 0.7 could lead to excessive shrinkage.

Exposure Conditions

The relative humidity, temperature, and air movement to which the element is exposed affect both the rate of shrinkage and the total (ultimate) shrinkage. Higher relative humidity, lower temperature, and low air velocity decrease shrinkage.

Size of Member

As the size of the member increases, the rate of evaporation decreases and hence the rate of shrinkage is lower. As the concrete matures, the amount of water lost to evaporation also decreases, because water cannot move freely in matured concrete. This results in a decrease in ultimate shrinkage.

Type of Cement

Cements that hydrate faster tend to produce more shrinkage. Special cements, called shrinkage-compensating cement, are available for reducing shrinkage. ASTM type K cement is called expansive cement, as it provides increases in volume rather than a decrease, or shrinkage.

Admixtures

As mentioned in Sec. 3. 1, a number of mineral and chemical admixtures are used in concrete to obtain certain properties in the fresh and hardened states. Accelerating admixtures tend to increase the rate of shrinkage whereas retarding admixtures will decrease the rate of shrinkage. Since concrete with water-reducing admixtures tends to have lower water–cement ratios, both the rate and the ultimate shrinkage for this concrete decreases. The effect of mineral admixtures depends on the type and volume fraction. Air-entraining agents were found to have little effect on shrinkage.

Carbonation

Carbonation, which occurs due to a reaction between the carbon dioxide present in the atmosphere and the cement paste, tends to produce shrinkage which is called carbonation shrinkage. This phe-

TABLE 3.18 Typical Values ($\times 10^{-6}$) of Shrinkage after 6 Months for Mortar and Concrete Specimens*

	Water-cement ratio			
Aggregate-cement ratio	0.4	0.5	0.6	0.7
3	800	1200	—	—
4	550	850	1050	—
5	400	600	750	850
6	300	400	550	650
7	200	300	400	500

*Based on 5-by 5-in (125- by 125-mm) prisms, stored at a relative humidity of 50% soda temperature of 70°F (21°C).

Source: From Lea.[17]

nomenon occurs only near the exposed surface. Carbonation tends to occur over a longer period of time than drying shrinkage. At humidities less than 50%, carbonation decreases drastically whereas drying shrinkage accelerates.

External Restraints
External restraints reduce shrinkage. The most common restraints are the reinforcing bars. Reinforcement tends to resist movement, and hence the total magnitude of shrinkage decreases. Because of the restriction in movement, a stress state occurs in reinforced concrete elements due to shrinkage. Typically reinforcement is subjected to compression and the surrounding concrete is subjected to tension. If the concrete is weak and its cross section is not sufficient to withstand the tensile forces, it might crack.

Mixing, Placing, Consolidation, and Curing
The way the concrete is mixed, placed in position, compacted, finished, and cured affects the quality of the concrete and hence the shrinkage. If these operations are not done properly, the resulting concrete could be porous, sustaining higher shrinkage strains. Curing is very important because the presence of moisture on the exposed surface reduces the evaporation loss considerably, resulting in reduced shrinkage.

Reduced shrinkage strain is better for the integrity of the concrete elements. Shrinkage increases the long-term deflections of beams and slabs, as well as the crack widths. In prestressed concrete shrinkage produces a loss in prestress. In composite construction and indeterminate structures, shrinkage produces a redistribution of stresses. These effects should be considered in the structural design.

3.4.7 Creep

The phenomenon of creep has a number of similarities with shrinkage. Most factors that affect shrinkage also affect creep. Shrinkage occurs under no load whereas creep occurs under a state of sustained stress. Typical variations of creep and shrinkage strains with time are shown in Fig. 3.2 1. Figure 3.2 1(*a*) shows the variation of shrinkage strain obtained using an unloaded specimen. If the specimen is sealed and subjected to sustained stress, the variation of strain with respect to time is shown in Fig. 3.21(*b*). Since the specimen is sealed, the shrinkage is essentially eliminated. As soon as the specimen is loaded, there is an elastic response producing an elastic or instantaneous strain. The strain continues to increase under the sustained stress. This additional strain is called creep strain. If the unsealed specimen is kept under sustained stress, both creep and shrinkage phenomenon occur. In addition, more shrinkage occurs because of the stress. The stress aids the movement of water, resulting in additional shrinkage strain.

As mentioned earlier, the factors that affect shrinkage also affect creep. The influences are similar in most cases. The factors that have less effect on creep than on shrinkage are type of cement and carbonation. The following additional factors influence the creep strain.

Level of Stress
There is a proportionality between the magnitudes of sustained stress and creep if the level of stress is less than 50% of compressive strength. Hence the creep strain is proportional to the elastic strain under normal working load conditions. However, if the level of stress increases beyond 70% of compressive strength, excessive creep strain occurs, leading to failure.

Time of Loading
The time of loading influences the creep strain because the strength of concrete increases with time. For example, if the concrete is subjected to sustained load at 7 days, it undergoes more creep strain as compared to concrete loaded at 28 days. It should be noted that the level of stress should be the same for both loading conditions. The influence of the time at loading decreases after 28 days and

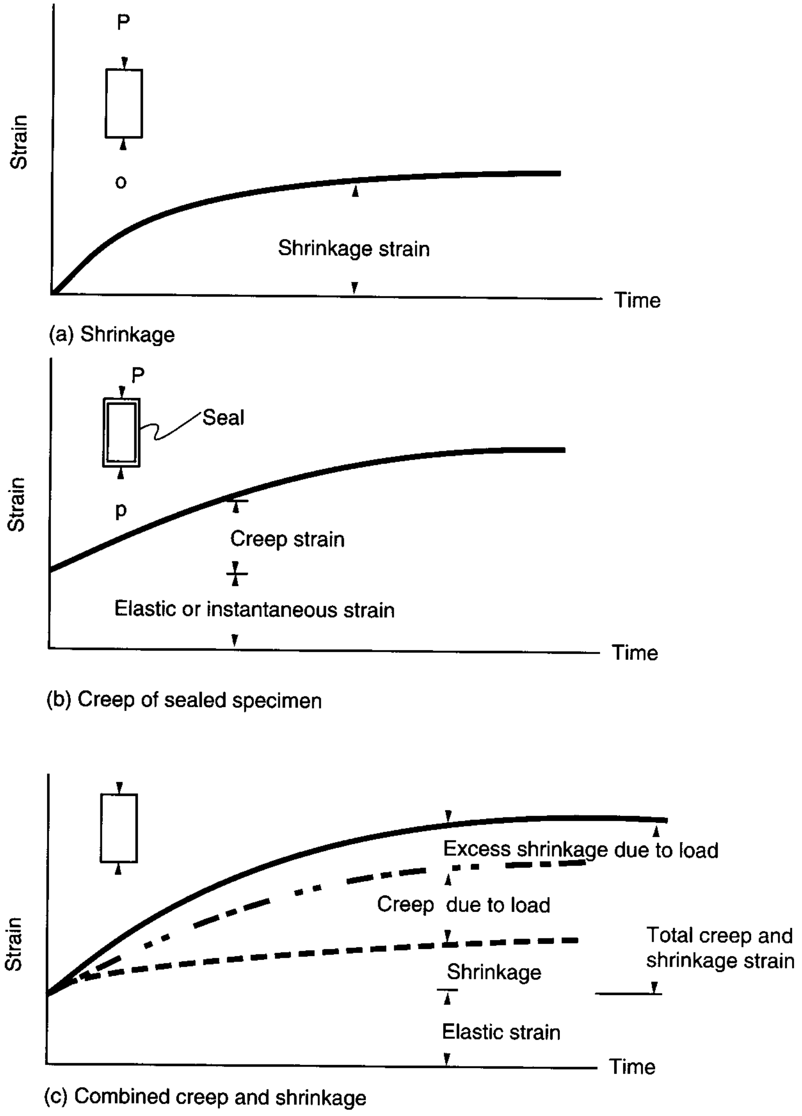


FIGURE 3.21 Typical variations of creep and shrinkage strains with time. (a) Shrinkage. (b) Creep of sealed specimens. (c) Combined creep and shrinkage.

becomes almost insignificant if the time at loading exceeds 56 days. This should be expected because the change in compressive strength after 56 days is negligible for normal concrete.

As is the case of shrinkage strain, creep strain increases deflections and crack widths, causes loss of prestress, and results in a redistribution of stresses in indeterminate structures and composite members. In the case of reinforced concrete columns, the redistribution of stresses could result in yielding of steel and buckling of eccentrically loaded columns.

Unloading typically leads to elastic and creep recovery. As in the case of shrinkage, creep recovery is not complete, leading to some amount of permanent deformation.

3.4.8 Estimation of Creep and Shrinkage Strains

A number of rheological models have been developed to simulate creep and shrinkage. These models consisting of Kelvin solid and Maxwell fluid can be used to predict the behavior of concrete subjected to various modes of loading such as constant stress, constant strain, and loading for a certain amount of time. These models can predict both creep and creep recovery. However, most designers prefer to use simple empirical equations for the prediction of creep and shrinkage strains.

A number of empirical equations exist for the prediction shrinkage and creep at a given time under load. The equations recommended by ACI Committee 209[18] are

$$(\epsilon_{sh})_t = \frac{t}{35+t}(\epsilon_{sh})_u \tag{3.24}$$

$$\gamma = \frac{t}{10+t^{0.6}}(C_u) \tag{3.25}$$

where $(\epsilon_{sh})_t$ = shrinkage strain at time
$(\epsilon_{sh})_t$ = ultimate shrinkage strain
γ_t = creep coefficient
$(C)_u$ = ultimate creep coefficient

For a given concrete the ultimate creep coefficient and ultimate shrinkage strain have to be assumed. These values can also be obtained using short-term readings taken at, say, 14, 28, or 56 days.

The equations recommended by the Comité Euro-International du Béton (CEB), Paris, France,[19] involve the use of coefficients for various exposure conditions and hence are more accurate. The shrinkage creep strains are computed using the following equation:

$$\epsilon_{sh} = \epsilon_c K_b K_t K_e \tag{3.26}$$

where

$$\epsilon_{cr} = \phi \text{ (elastic strain)} \tag{3.27}$$

$$\phi = K_c K_b K_d K_t' K_e' \tag{3.28}$$

where ϵ_{sh}, K_b, K_e, K_t, K_c, K_d, K_t', and K_e' are coefficients. The values for these coefficients can be obtained using Figs. 3.22 and 3.23.

3.4.9 Behavior under Multiaxial Stresses

In some structures such as two-way slabs and nuclear reactor containment vessels the concrete is subjected to biaxial and triaxial stresses. Another example for triaxial state of stress is off-shore oil platforms. Concrete located near the bottom of the sea is subjected to water pressure and external load, resulting in a triaxial state of stress.

In the case of biaxial loading, the strength of concrete increases by about 20% under biaxial compression. Figure 3.24 shows the failure envelope for concrete subjected to biaxial loading tn tension and compression. The increase in strength under biaxial loading is normally neglected in de-

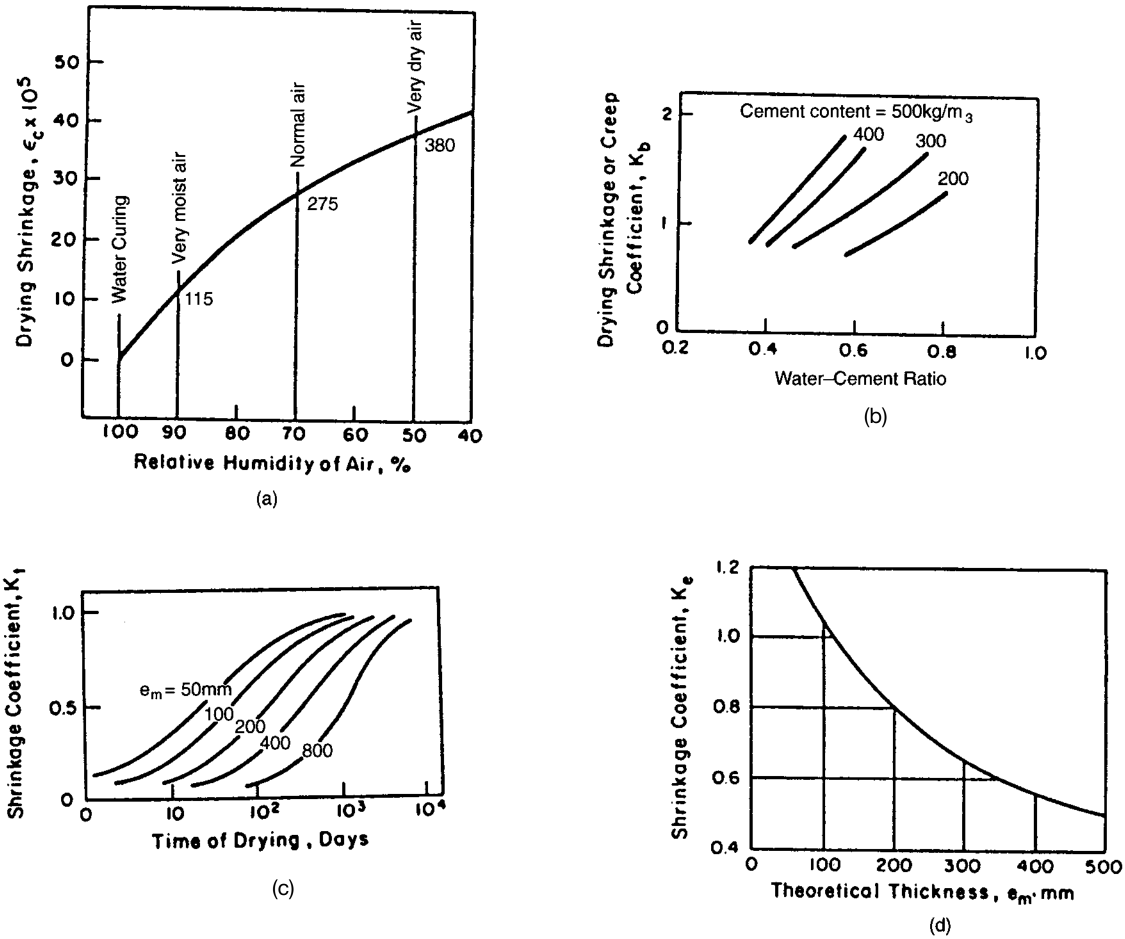


FIGURE 3.22 Coefficients ϵ_c, K_b, K_t, and K_e for CEB method. (*From CEB/FIP.*[19])

sign computations. Hence structural components subjected to biaxial loading can be designed using uniaxial strengths.

Concrete subjected to triaxial loading can withstand much higher stresses as compared to uniaxial strength, as shown in Fig. 3.25. In general, triaxial compression improves the compressive strength of leaner or low-strength concrete more than that of a stronger or high-strength concrete. Failure theories have been developed for the triaxial state of stress. The most popular ones are known as octahedral shear stress theory and Mohr-Coulomb failure theory. Some researchers have also developed empirical relationships between minor axial stress σ_3 and major axial stress σ_1, which creates failure. Under the uniaxial state of stress σ_3 is zero, and hence σ_1 is the compressive strength f'_c. One such empirical equation is

$$\sigma_1 = f'_c + 4.8\,\sigma_3 \tag{3.29}$$

where f'_c is the compressive strength.

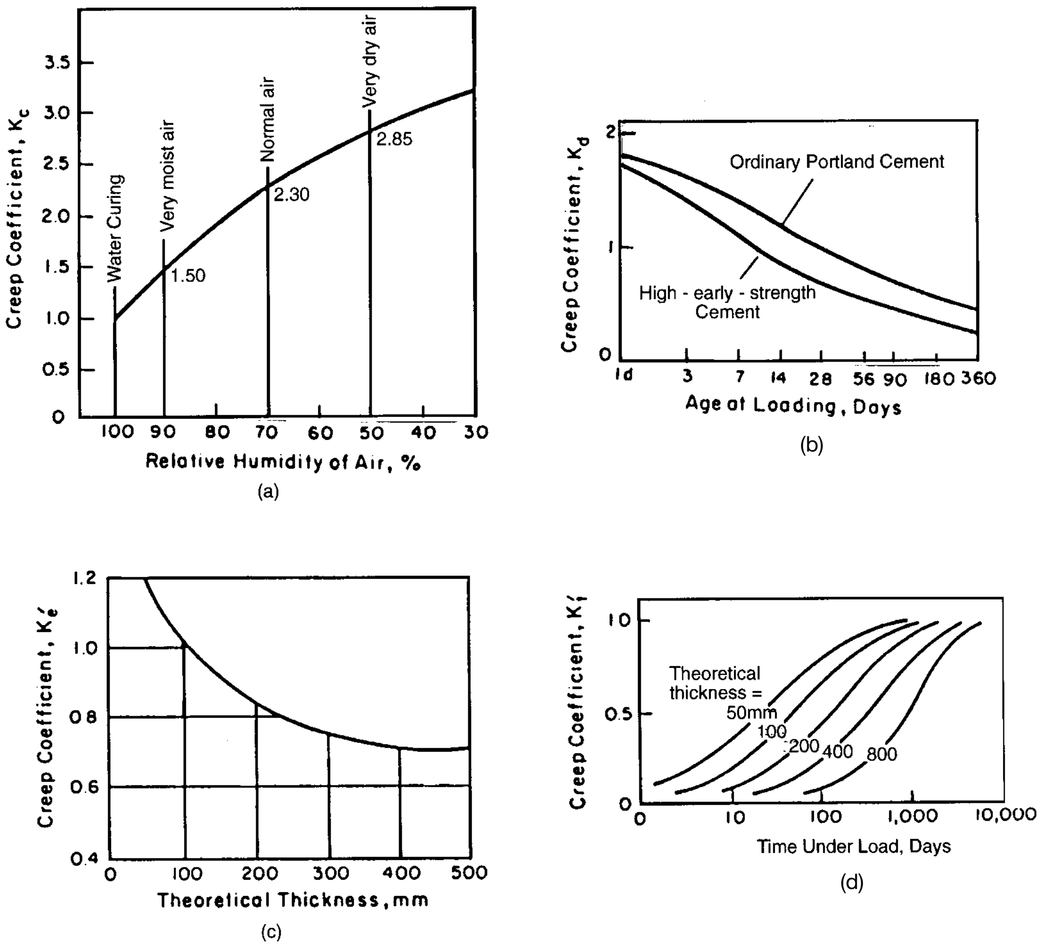


FIGURE 3.23 Coefficients K_c, K_d, K'_e, and K'_t for CEB method. (*From CEB/FIP.*[19])

For example, concrete with a compressive strength of 4000 psi (28 MPa) can be expected to withstand 8800 psi (61 MPa) under a lateral pressure of 1000 psi (6.89 MPa) ($\sigma_1 = 4000 + 4.8 \times 1000$). More detailed information regarding various theories and stress-strain behavior can be found in the literature.

3.4.10 Fatigue Loading

In fatigue loading, the structural components are subjected to varying stresses. Typical examples are bridge beams, offshore structures subjected to wave loads, and machine foundations. Extensive research have been conducted on the behavior of concrete subjected to fatigue loading. The following major conclusions, arrived at by the various investigators, may be useful for the designer.

- Concrete can withstand 10 million cycles if the stress range (difference between maximum and minimum stresses) is less than 55% of compressive strength and the minimum stress is about zero.

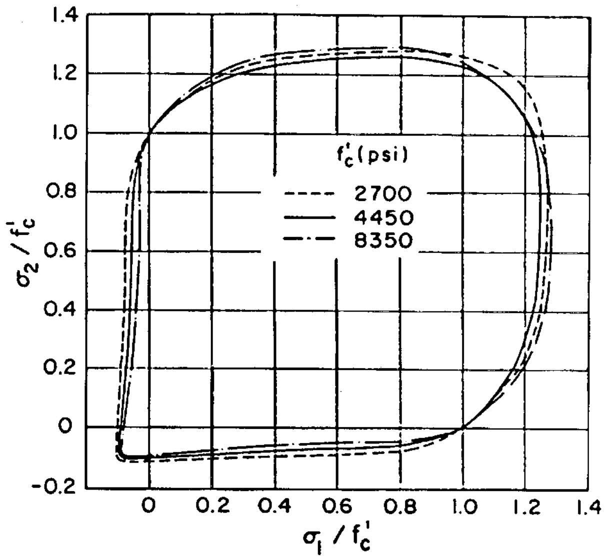


FIGURE 3.24 Biaxial stress interaction curves. (*From Kupfel et al.*[19])

- The fatigue strength decreases with an increase in minimum strength.
- Components subjected to different intensities of fatigue loading can be designed using Minor's hypothesis.
- Frequency of loading of between 70 and 900 cycles has little effect on fatigue strength, provided the stress range is less than 70% of compressive strength.
- The variables of mix proportion, such as water–cement ratio and cement content, affect fatigue strength in the same way as static compressive strength. Behavior in tension and flexure is about the same as behavior in compression. For example, the flexural fatigue strength at a stress range of 55% of static flexural strength (modulus of rupture) is about 10 million cycles.
- The stress gradient increases the fatigue life.
- Rest periods do not affect the fatigue life significantly.
- The creep strain under fatigue loading is higher than the creep strain under the static load that corresponds to the maximum fatigue load. In other words, even though the average stress is lower for fatigue load conditions the creep strain is higher.

3.5 COLD-WEATHER CONCRETING

When the temperature falls below 20°F (–6°C), the hydration of cement becomes extremely slow. Therefore concrete placed at low temperatures should be protected until it gains sufficient strength to resist freezing action. Normally the minimum recommended compressive strength is 500 psi (4 MPa). Hydration of cement also reduces the degree of saturation because the chemical action consumes the water.

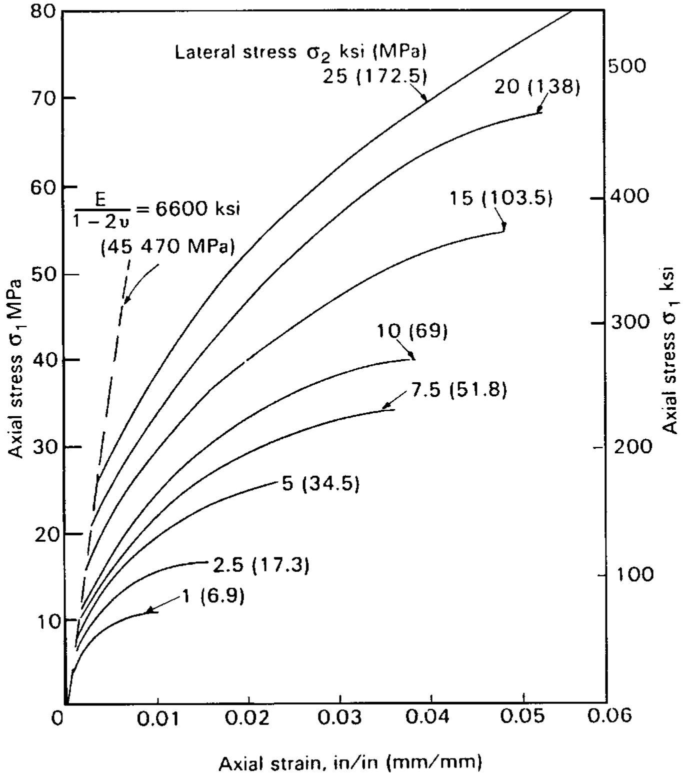


FIGURE 3.25 Behavior in triaxial compression.

The practice recommended by ACI Committee 306 for cold-weather concreting is shown in Table 3.19. The committee recommends that the concrete be maintained at a certain minimum temperature for lb 3 days, depending on whether it is the conventional or the high-early-strength type. For moderately and fully stressed members longer durations are recommended. The recommended minimum temperature depends on the exposure temperature and the thickness of the specimen. Since thicker specimens dissipate the heat of hydration more slowly, they could be maintained at a lower temperature than thin sections. Non-air-entrained concrete should be protected for at least twice the number of days because it is much more susceptible to freeze-thaw damage than air-entrained concrete. In most cases the concrete could be maintained without using external heat sources if proper care is taken to maintain the temperature of the ingredient materials and the insulation is properly placed. The effect of frozen ground, reinforcing bars, and formwork should be considered in computing the temperature requirements of the ingredients. For very thin sections external heat may be needed to maintain the recommended concrete temperature. Temperatures higher than 70°F (21 °C) are not normally recommended.

Most specifications are written based on minimum compressive strength. It is assumed that if the concrete has the specified compressive strength, it is durable. This may not be true in all cases, particularly for concrete exposed to cold weather in its fresh state. The frost action could impart considerable internal damage, making the concrete less durable. If durability is a main consideration the concrete should be protected for longer periods than recommended in Table 3.19.

The temperature of the fresh concrete can be controlled by controlling the temperature of the

TABLE 3.19 Recommended Concrete Temperatures for Cold-Weather Construction, Air-Entrained Concrete

	Sections < 12 in (0.3 in) thick		Sections 12–36 in (0.3–0.9 in) thick		Sections 36–72 in (0.9–1.8 in) thick		Sections > 72 in (1.8 in) thick	
	°F	°C	°F	°C	°F	°C	°F	°C
Minimum temperature for fresh concrete as mixed in weather indicated								
Above30°F(–l°C)	60	16	55	13	50	10	45	7
0 to 30°F(–18to1°C)	65	18	60	16	55	13	50	10
Below 0°F (–18°C)	70	21	65	18	60	16	55	13
Minimum temperature for fresh concrete *as placed and maintained*	55	13	50	10	45	7	40	5
Maximum allowable *gradual* drop in temperature in first 24 h after end of protection	50	28	40	22	30	17	20	11

Source: From ACt 306 R-SS.[21]

constituent materials, namely, cement, aggregates, and water. Since the specific heat of water is 1.0 as compared to 0.22 for cement and aggregates, it is more efficient to heat the water. In addition it is easier to heat the water than the other ingredients used in concrete.

If the outside temperature is above freezing, aggregates are not usually heated. In temperatures below freezing, heating of fine aggregates could be sufficient. Coarse aggregate is heated only as a last resort because it is more difficult to heat loosely packed materials. Fine aggregates are generally heated by circulating hot air or steam through pipes that are embedded in them. In any case, the final temperature of the freshly mixed concrete should be maintained at the specified level. The temperature of fresh concrete T can be estimated using the following equation:

$$T = \frac{0.22(T_a W_a + T_c W_c) + T_w W_w + T_{wa} W_{wa}}{0.22(W_a + W_c) + W_w + W_{wa}} \tag{3.30}$$

where T_a, T_c, T_w, and T_{wa} are the temperatures of aggregate, cement, water, and free moisture in aggregates, respectively, and W_a, W_c, W_w, and W_{wa} are the weights of aggregate, cement, water, and free moisture in aggregates, respectively. All the temperatures are in °F and the weights are in pounds. For SI units the temperatures are expressed in °C and the weights in kilograms. The temperatures of concrete should be checked using thermometers. Both mercury and bimetallic thermometers are available for that purpose.

3.6 HOT-WEATHER CONCRETING

Concreting in hot weather leads to a rapid hydration rate and evaporation loss, resulting in microcracks and inferior final product. Since lower humidity and higher wind velocity also lead to rapid evaporation, hot weather for concreting purposes is taken as a combination of temperature, relative humidity, and wind velocity. If the relative humidity is high, such as near sea shores, special precautions may not be needed up to about 85°F (29°C). If the humidity is very low, such as in desert conditions, precautions might be needed even at temperatures lower than 80°F (27°C). The combined effect of temperature, relative humidity, and wind velocity can be judged using the amount of water that evaporates from fresh concrete. Nomograms are available for estimating the rate of evaporation for a given site condition.

Solar radiation can also affect the properties of fresh concrete in a number of ways. If the ingredients are stored outside, they can absorb heat during transport or during the waiting period. Heating of reinforcement and formwork can further aggravate the situation.

Fresh Concrete

Hot weather adversely affects workability. For the same workability, or to improve the workability, more water is needed at higher temperatures. Unfortunately the excess water added results in reduced strength and less durable concrete. Hence it is advisable to use water-reducing admixtures such as superplasticizers rather than excess water. Researchers have shown that superplasticizers can be used effectively to increase workability without adversely affecting strength and durability.

Concrete stiffens much faster at higher temperatures. The rate of hydration approximately doubles for every 18°F (10°C) increase in temperature. Hence the workability decreases much faster at high temperatures. The change in workability can be best represented by the loss in slump values. Figure 3.26 shows the variation of slump at two different temperatures. It can be seen that slump loss is much more rapid at 95°F (35°C) than at 73°F (23°C). Set-retarding admixtures may be used to delay the stiffening of concrete.

When the concrete is placed in position, vibrated, and the top surface finished to the desired texture, a certain amount of water rises up to the surface. This water, called bleed water, should be kept to a minimum. The bleed water does help to prevent shrinkage cracking. However, at high temperatures the bleed water may evaporate rapidly, causing excessive shrinkage cracks.

Hardened Concrete

Concrete placed and cured at higher temperatures develops higher strengths at the early ages of maturity. But the final strength, measured after 28 days, decreases with an increase in placing and curing temperature. The reduction in strength occurs both in the compression and the flexure mode. Typical variations of compressive strengths at various temperatures are shown in Fig. 3.27.

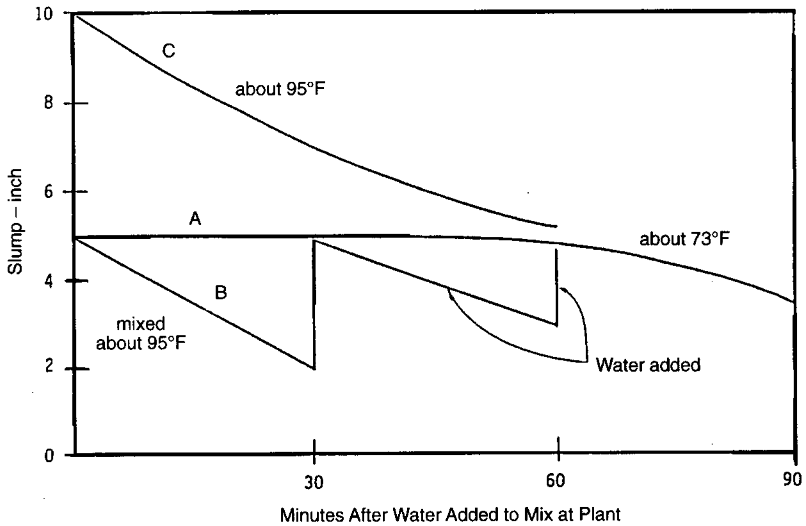


FIGURE 3.26 Influence of temperature on slump variation with time. (*From Shilstone.*[22])

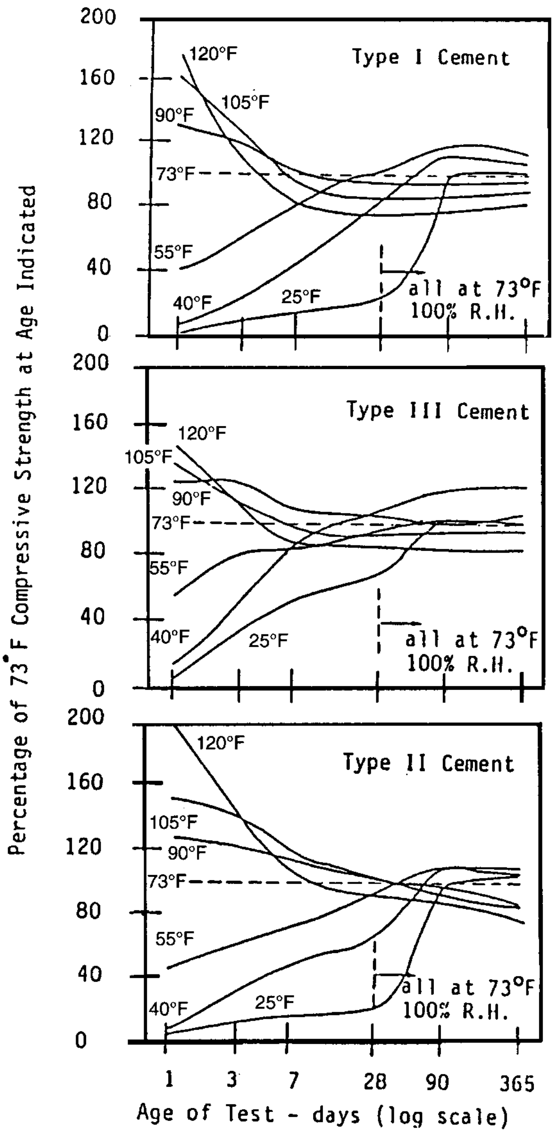


FIGURE 3.27 Effect of temperature on compressive strength for different types of cement. (*From Kliegar.*[23])

Concrete cast during hot weather develops more shrinkage cracks. This results in more permeable and less durable concrete. In general, concrete placed at high temperatures without taking special precautions can be considered weak under both thermal (freeze-thaw) and chemical attacks.

In general, creep of concrete increases with the increase in temperature. Concrete exposed to temperatures higher than 90°F (32°C) could undergo considerably more creep strains. In certain cases the higher creep strains should be taken into consideration in structural design.

Precautionary Measures for Hot-Weather Concreting

The best approach is to cast concrete at temperatures lower than 85°F (29°C). In some instances concrete could be cast during the evening or night hours rather than the daytime. If concrete has to be placed at higher temperatures, the best way to avoid problems is to keep the concrete temperature low. This can be achieved by cooling the aggregates or the water.

Using ice in the mixing water is the most efficient way to lower the concrete temperature. The approximate temperature of the concrete can be calculated using Eq. (3.30). If ice is used as part of the mixing water, the equation can be modified as follows:

$$T = \frac{0.22(T_a W_a + T_c W_c) + T_w W_w + T_{wa} W_{wa} - 112\, I}{0.22(W_a + W_c) + W_w + W_{wa} + I} \tag{3.31}$$

where I is the weight of ice in pounds.

The amount of ice needed to obtain a certain concrete temperature can be computed by the following modified form of Eq. (3.31):

$$I = \frac{0.22[W_a(T_a - T) + W_c(T_c - T)] + W_{wa}(T_a - T) + W_{wa}(T_w - T)}{112 + T} \tag{3.32}$$

Shaved ice can be added to the mixer as part of the mixing water. If block ice is used, it should be crushed before adding. In either case, ice must be completely melted at the end of the mixing cycle. Water from melted ice must be considered as part of the total mixing water, keeping the water–cement ratio the same.

Under certain circumstances, liquid nitrogen has been shown to be economical and practical, especially if low concrete temperatures are needed. However, this method can be used only when nitrogen manufacturing facilities are available locally. Liquid nitrogen can be used to cool the aggregates, the water, or the concrete mix. Experience shows that a combination provides a repeatable and quality mix.

Some of the proven techniques helpful for hot-weather concreting are listed here. The recommendations are not elaborated upon for lack of space.

- Keep the concrete temperature low, preferably below 85°F (29°C).
- Provide shade over stockpiles of coarse and fine aggregates.
- Use chilled water or partly replace mixing water with ice for mixing concrete, or use liquid nitrogen to cool the concrete.
- Keep the cement content to the minimum required for strength and durability.
- Use appropriate water-reducing, superplasticizing, and set-retarding admixtures after establishing, under site conditions, dosage and compatibility with the cement used.
- Paint all mixing and conveying equipment for concreting with reflective or light-colored paint.
- Precool the forms, reinforcement, and surroundings prior to concrete placement.
- Place concrete at night.
- Place concrete in layers of optimum thickness for efficient compaction and avoidance of cold joints.
- Wet or moist curing and membrane protection are necessary after the concrete has hardened.

Retempering of Concrete in Hot Weather

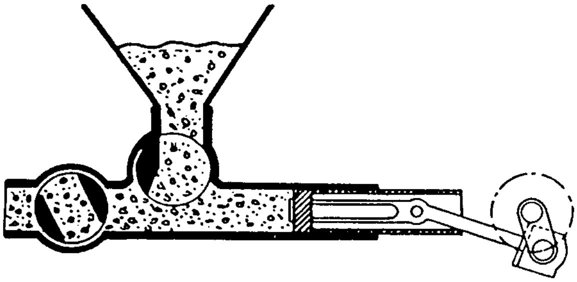
Normally, retempering of concrete to improve its workability is considered a bad option. But recent research shows that the retempering technique can be used successfully without adverse effects. A study conducted using water and superplasticizer for retempering concretes, mixed at temperatures up to 140°F (60°C), led to the following conclusions.

- Additional water and cement are needed (maintaining the same water–cement ratio) at higher ambient temperatures to achieve the same slump. The additional water demand is very high for concretes with low water-to-cement ratios (0.4) at temperatures higher than 104°F (40°C). However, for concretes with higher water–cement ratios (0.5 and 0.6) there is only a slight increase in the water required to maintain the same slump for a temperature range of 86 to 140°F (30 to 60°C).
- For all concretes the quantity of retempering water required to restore their initial slumps, after an elapse of a 30-min period, increases with an increase in the ambient temperature for both first and second retemperings. The quantity of water needed for second retempering is significantly higher than that for first retempering at all temperatures. Concretes with lower water–cement ratios (0.4) need considerably higher quantities of retempering water at higher temperatures.
- The cohesiveness and finishability of concrete seems to be better after retempering than after initial mixing.
- Slump loss is considerably higher for a concrete with a water–cement ratio of 0.4 than for concretes with water–cement ratios of 0.5 and 0.6. After retempering the rate of slump loss is higher for all concretes. The rate of slump loss is not significantly higher at higher temperatures.
- No appreciable change in the unit weight of fresh concrete occurs after first and second retemperings at all temperatures tested. For concretes with low water–cement ratios (0.4) an increase in the ambient temperature causes a decrease in the plastic unit weight.
- There is no apparent change in the entrapped air either due to a temperature increase or due to retempertngs.
- All hardened concrete properties (compressive strength, splitting tensile strength, flexural strength, static modulus of elasticity, dynamic modulus, pulse velocity, and dry unit weight) are affected similarly by an increase in the ambient temperature from 86 to I 40°F (30 to 60°C) and due to first and second retemperings. There is a successive, though not significant, reduction in the strength and modulus values after first and second retemperings. There is a slight reduction (less than 5%) as the temperature increases from 86 to 1400F (30 to 60°C). The concretes most affected by temperature increase are those with a water–cement ratio of 0.4.
- There is no positively recognizable change in the relationships between the various properties of hardened concrete either due to an increase in temperature from 86 to 1400F (30 to 60°C) or due to two retemperings.
- There is no significant difference in the properties, particularly in the case of compressive strength, of concretes mixed, cast, and cured under identical conditions and with two different agents, namely, superplasticizer and water. The observed difference between the two is less than 15%. Moreover the variation is not consistent. A statistical analysis conducted using regression equations relating the compressive strength and other properties, such as splitting tensile strength, flexural strength, pulse velocity, static modulus, dynamic modulus, and dry unit weight, confirms the observation that the two different retempering agents have the same influence on the properties of concrete.

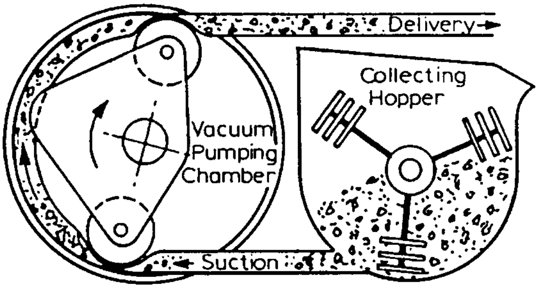
3.7 PUMPING OF CONCRETE

In modern construction, pumping of concrete has become quite common. Pumping has a number of advantages, such as providing a continuous supply of concrete, access to hard to reach places, and economy . The pumping system consists of a hopper, a concrete pump, and pipes that can be connected and dismantled easily.

Two typical pumps are shown in Fig. 3.28. Direct-acting horizontal piston pumps shown in Fig. 3.28(a) are more common. The semirotary valves allow the passage of coarse aggregate particles. Concrete, which is fed into the hopper from the mixer, gets sucked into the pipes by gravity and the



(a)



(b)

FIGURE 3.28 Concrete pumps. (a) Direct-acting horizontal piston pump. (b) Squeeze-type pump.

vacuum action created during the suction stroke. These pumps can pump up to 1500 ft (457 in) horizontally and 140 ft (43 in) vertically.

The squeeze-type pumps shown in Fig. 3.28(b) are normally smaller and truck-mounted. In this pump concrete placed in the collecting hopper is fed by rotating blades into a pliable pipe located in the pumping chamber. The pumping chamber, which can maintain a vacuum of about 26 in (660 mm) of mercury, supplies continuous feed to the delivery pipe. Delivery is normally done using folding boom consisting of 3- and 4-in (75- and 100-mm) pipes. Squeeze pumps can pump up to 300 ft (91 in) horizontally and 100 ft (30 in) vertically.

Different pump and pipe sizes are available. Squeeze pumps can deliver up to 25 yd^3 (19 m^3) per hour whereas piston pumps can deliver up to 80 yd^3 (61 m^3) per hour. Piston pumps work with pipes up to 9 in (230 mm) in diameter. The pipe diameter should be at least three times the maximum size of the aggregate. Hence even squeeze pumps with 3-in (75-mm)-diameter pipes can handle most concrete used for buildings.

Two types of blockages can occur in pumping. The first occurs due to segregation, the second due to excessive friction. When the concrete has too much water, the solid particles consisting of aggregates cannot be carried through by the liquid medium because the water escapes through the mix. Since water is the only medium pumpable in its natural state, the aggregates stay behind and get clogged. If the mix is cohesive, then the water will carry the solid particles with it.

When the mix is very cohesive, the friction between the walls and the mix becomes high and the pump cannot overcome this friction, which will result in a blockage. This type of failure is more common in high-strength concrete mixes and in mixes containing a high proportion of very fine material such as crushed dust or fly ash.

The optimum mix is the one that produces maximum internal frictional resistance within the ingredients and minimum frictional resistance against the pipe walls. Void sizes should also be minimum. For concrete containing 0.75-in (19-mm) maximum size aggregate, fine aggregate should be in the range of 35 to 40%. Of these 15 to 20% should pass through ASTM sieve 50, or finer than 300 μm.

Concrete should be mixed well before feeding it into the hopper. In some cases additional mixing is done in the hopper using stirrers. The mix cannot be too harsh, too dry, too wet, or too sticky. A slump of 1.5 104 in (38 to 100 mm) normally produces satisfactory results. Since pumping provides some compaction, the mix at the delivery point could have a slump lowered by as much as 1 in (25 mm). When the concrete is at the correct consistency, a thin lubricating film forms near the surface of the pipes, allowing smooth flow of concrete.

The following are some of the additional factors to be considered in pumping concrete.

- Pumping is economical only if it can be used over long uninterrupted periods because considerable effort is needed for lubricating and cleaning at the beginning and end.
- A short piece of flexible hose near the end makes the placement easier. But the flexible hose could increase the friction loss.
- Bends should be kept to a minimum.
- Aluminum pipes should not be used because they react with alkalis in cement and generate hydrogen bubbles, weakening the hardened concrete.
- The shape of the aggregate influences the pumpability of the mix. Natural sands are preferable to crushed sands because of their spherical shape and continuous uniform grading.
- The presence of entrained air increases the pumping effort. If large amounts of entrained air are present, the entire movement of the piston could be wasted on compressing the air bubbles, resulting in no flow. In general air-entrained concrete can be pumped only shorter distances as compared to non-air-entrained concrete.
- Typically pumping lightweight aggregate concrete needs more effort. Sealing of the surface of the aggregates may be necessary, or special admixtures may be needed to pump lightweight concrete. Otherwise aggregates can absorb more water under pressure, making the mix stiffer and hence more difficult to pump. Some of the aggregate may also crush during the pumping process.
- Concrete with unsatisfactory conditions in the fresh state cannot be pumped. Pumpable concrete, in almost all cases, has the right consistency for placing and finishing.

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