2 MATERIALS

2.1 INTRODUCTION

The structures and component members treated in this text are composed of concrete reinforced with steel bars, and in some cases prestressed with steel wire, strand, or alloy bars. An understanding of the materials characteristics and behavior under load is fundamental to understanding the performance of structural concrete, and to safe, economical, and serviceable design of concrete structures. Although prior exposure to the fundamentals of material behavior is assumed, a brief review is presented in this chapter, as well as a description of the types of bar reinforcement and prestressing steels in common use. Numerous references are given as a guide for those seeking more information on any of the topics discussed.

2.2 CEMENT

A cementitious material is one that has the adhesive and cohesive properties necessary to bond inert aggregates into a solid mass of adequate strength and durability. This technologically important category of materials includes not only cements proper but also limes, asphalts, and tars as they are used in road building, and others. For making structural concrete, so-called *hydraulic cements* are used exclusively. Water is needed for the chemical process (hydration) in which the cement powder sets and hardens into one solid mass. Of the various hydraulic cements that have been developed, *portland cement*, which was first patented in England in 1824, is by far the most common.

Portland cement is a finely powdered, grayish material that consists chiefly of calcium and aluminum silicates.[†] The common raw materials from which it is made are limestones, which provide CaO, and clays or shales, which furnish SiO_2 and Al_2O_3. These are ground, blended, fused to clinkers in a kiln, and cooled. Gypsum is added and the mixture is ground to the required fineness. The material is shipped in bulk or in bags containing 94 lb of cement.

Over the years, five standard types of portland cement have been developed. Type I, *normal* portland cement, is used for over 90 percent of construction in the United States. Concretes made with Type I portland cement generally need about two weeks

[†] See ASTM C 150, "Standard Specification for Portland Cement." This and other ASTM references are published and periodically updated by ASTM International (formerly the American Society for Testing and Materials), West Conshohoken, PA.

to reach sufficient strength so that forms of beams and slabs can be removed and reasonable loads applied; they reach their design strength after 28 days and continue to gain strength thereafter at a decreasing rate. To speed construction when needed, *high early strength cements* such as Type III have been developed. They are costlier than ordinary portland cement, but within about 7 to 14 days they reach the strength achieved using Type I at 28 days. Type III portland cement contains the same basic compounds as Type I, but the relative proportions differ and it is ground more finely.

When cement is mixed with water to form a soft paste, it gradually stiffens until it becomes a solid. This process is known as *setting* and *hardening*. The cement is said to have set when it has gained sufficient rigidity to support an arbitrarily defined pressure, after which it continues for a long time to harden, i.e., to gain further strength. The water in the paste dissolves material at the surfaces of the cement grains and forms a gel that gradually increases in volume and stiffness. This leads to a rapid stiffening of the paste 2 to 4 hours after water has been added to the cement. *Hydration* continues to proceed deeper into the cement grains, at decreasing speed, with continued stiffening and hardening of the mass. The principal products of hydration are calcium silicate hydrate, which is insoluble, and calcium hydroxide, which is soluble.

In ordinary concrete, the cement is probably never completely hydrated. The gel structure of the hardened paste seems to be the chief reason for the volume changes that are caused in concrete by variations in moisture, such as the shrinkage of concrete as it dries.

For complete hydration of a given amount of cement, an amount of water equal to about 25 percent of that of cement, by weight—i.e., a *water-cement ratio* of 0.25—is needed chemically. An additional amount must be present, however, to provide mobility for the water in the cement paste during the hydration process so that it can reach the cement particles and to provide the necessary workability of the concrete mix. For normal concretes, the water-cement ratio is generally in the range of about 0.40 to 0.60, although for high-strength concretes, ratios as low as 0.21 have been used. In this case, the needed workability is obtained through the use of admixtures.

Any amount of water above that consumed in the chemical reaction produces pores in the cement paste. The strength of the hardened paste decreases in inverse proportion to the fraction of the total volume occupied by pores. Put differently, since only the solids, and not the voids, resist stress, strength increases directly as the fraction of the total volume occupied by the solids. That is why the strength of the cement paste depends primarily on, and decreases directly with, an increasing water-cement ratio.

The chemical process involved in the setting and hardening liberates heat, known as *heat of hydration*. In large concrete masses, such as dams, this heat is dissipated very slowly and results in a temperature rise and volume expansion of the concrete during hydration, with subsequent cooling and contraction. To avoid the serious cracking and weakening that may result from this process, special measures must be taken for its control.

2.3 Aggregates

In ordinary structural concretes the aggregates occupy about 70 to 75 percent of the volume of the hardened mass. The remainder consists of hardened cement paste, uncombined water (i.e., water not involved in the hydration of the cement), and air voids. The latter two evidently do not contribute to the strength of the concrete. In general, the more densely the aggregate can be packed, the better the durability and economy of the concrete. For this reason the gradation of the particle sizes in the aggregate, to produce close packing, is of considerable importance. It is also important that the aggregate has good strength, durability, and weather resistance; that its surface is free from impurities such as loam, clay, silt, and organic matter that may weaken the bond with cement paste; and that no unfavorable chemical reaction takes place between it and the cement.

Natural aggregates are generally classified as fine and coarse. *Fine aggregate* (typically natural sand) is any material that will pass a No. 4 sieve, i.e., a sieve with four openings per linear inch. Material coarser than this is classified as *coarse aggregate*. When favorable gradation is desired, aggregates are separated by sieving into two or three size groups of sand and several size groups of coarse aggregate. These can then be combined according to grading charts to result in a densely packed aggregate. The *maximum size of coarse aggregate* in reinforced concrete is governed by the requirement that it shall easily fit into the forms and between the reinforcing bars. For this purpose it should not be larger than one-fifth of the narrowest dimension of the forms or one-third of the depth of slabs, nor three-quarters of the minimum distance between reinforcing bars. Requirements for satisfactory aggregates are found in ASTM C 33, "Standard Specification for Concrete Aggregates," and authoritative information on aggregate properties and their influence on concrete properties, as well as guidance in selection, preparation, and handling of aggregate, is found in Ref. 2.1.

The unit weight of *stone concrete*, i.e., concrete with natural stone aggregate, varies from about 140 to 152 pounds per cubic foot (pcf) and can generally be assumed to be 145 pcf. For special purposes, lightweight concretes, on the one hand, and heavy concretes, on the other, are used.

A variety of *lightweight* aggregates is available. Some unprocessed aggregates, such as pumice or cinders, are suitable for insulating concretes, but for structural lightweight concrete, *processed aggregates* are used because of better control. These consist of expanded shales, clays, slates, slags, or pelletized fly ash. They are light in weight because of the porous, cellular structure of the individual aggregate particle, which is achieved by gas or steam formation in processing the aggregates in rotary kilns at high temperatures (generally in excess of 2000°F). Requirements for satisfactory lightweight aggregates are found in ASTM C 330, "Standard Specification for Lightweight Aggregates for Structural Concrete."

Three classes of lightweight concrete are distinguished in Ref. 2.2: low-density concretes, which are chiefly employed for insulation and whose unit weight rarely exceeds 50 pcf; moderate strength concretes, with unit weights from about 60 to 85 pcf and compressive strengths of 1000 to 2500 psi, which are chiefly used as fill, e.g., over light-gage steel floor panels; and structural concretes, with unit weights from 90 to 120 pcf and compressive strengths comparable to those of stone concretes. Similarities and differences in structural characteristics of lightweight and stone concretes are discussed in Sections 2.8 and 2.9.

Heavyweight concrete is sometimes required for shielding against gamma and x-radiation in nuclear reactors and similar installations, for protective structures, and for special purposes, such as counterweights of lift bridges. Heavy aggregates are used for such concretes. These consist of heavy iron ores or barite (barium sulfate) rock crushed to suitable sizes. Steel in the form of scrap, punchings, or shot (as fines) is also used. Unit weights of heavyweight concretes with natural heavy rock aggregates range from about 200 to 230 pcf; if iron punchings are added to high density ores, weights as high as 270 pcf are achieved. The weight may be as high as 330 pcf if ores are used for the fines only and steel for the coarse aggregate.

2.4 Proportioning and Mixing Concrete

The various components of a mix are proportioned so that the resulting concrete has adequate strength, proper workability for placing, and low cost. The third calls for use of the minimum amount of cement (the most costly of the components) that will achieve adequate properties. The better the gradation of aggregates, i.e., the smaller the volume of voids, the less cement paste is needed to fill these voids. In addition to the water required for hydration, water is needed for wetting the surface of the aggregate. As water is added, the plasticity and fluidity of the mix increase (i.e., its workability improves), but the strength decreases because of the larger volume of voids created by the free water. To reduce the free water while retaining the workability, cement must be added. Therefore, as for the cement paste, the *water-cement ratio* is the chief factor that controls the strength of the concrete. For a given water-cement ratio, one selects the minimum amount of cement that will secure the desired workability.

Figure 2.1 shows the decisive influence of the water-cement ratio on the compressive strength of concrete. Its influence on the tensile strength, as measured by the nominal flexural strength or modulus of rupture, is seen to be pronounced but much

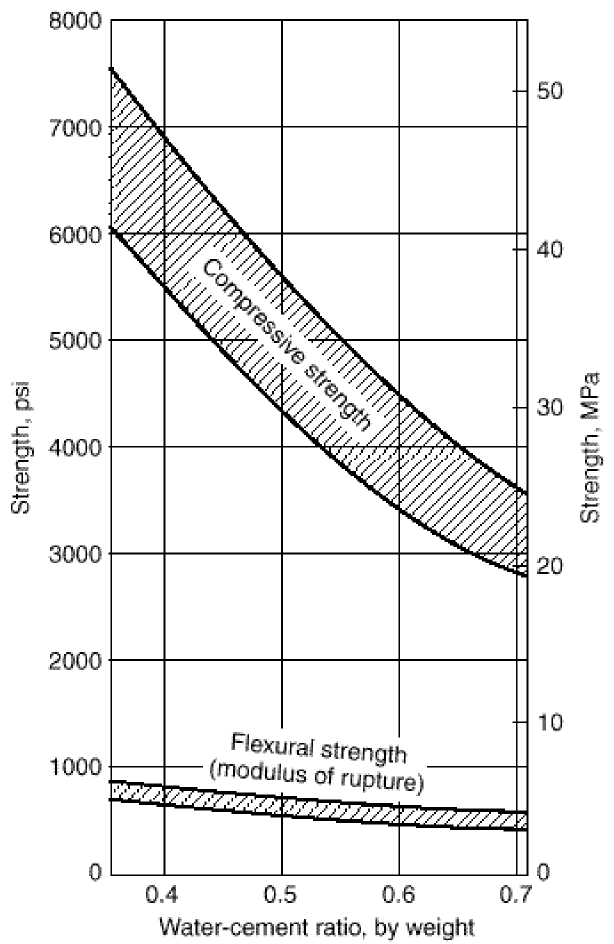


FIGURE 2.1
Effect of water-cement ratio on 28-day compressive and flexural tensile strength.
(Adapted from Ref. 2.4.)

smaller than its effect on the compressive strength. This seems to be so because, in addition to the void ratio, the tensile strength depends strongly on the strength of bond between coarse aggregate and cement mortar (i.e., cement paste plus fine aggregate). According to tests at Cornell University, this bond strength is only slightly affected by the water-cement ratio (Ref. 2.3).

It is customary to define the *proportions* of a concrete mix in terms of the total weight of each component needed to make up 1 yd^3 of wet concrete, such as 517 lb of cement, 300 lb of water, 1270 lb of sand, and 1940 lb of coarse aggregate, plus the total volume of air, in percent, when air is deliberately *entrained* in the mix (typically 4 to 7 percent). The weights of the fine and coarse aggregates are based on material in the *saturated surface dry condition*, in which, as the description implies, the aggregates are fully saturated but have no water on the exterior of the particles.

Various methods of proportioning are used to obtain mixes of the desired properties from the cements and aggregates at hand. One is the so-called *trial-batch method*. Selecting a water-cement ratio from information such as that in Fig. 2.1, one produces several small trial batches with varying amounts of aggregate to obtain the required strength, consistency, and other properties with a minimum amount of paste. Concrete *consistency* is most frequently measured by the *slump test*. A metal mold in the shape of a truncated cone 12 in. high is filled with fresh concrete in a carefully specified manner. Immediately upon being filled, the mold is lifted off, and the slump of the concrete is measured as the difference in height between the mold and the pile of concrete. The slump is a good measure of the total water content in the mix and should be kept as low as is compatible with workability. Slumps for concretes in building construction generally range from 2 to 5 in., although higher slumps are used with the aid of chemical admixtures.

The so-called ACI method of proportioning makes use of the slump test in connection with a set of tables that, for a variety of conditions (types of structures, dimensions of members, degree of exposure to weathering, etc.), permit one to estimate proportions that will result in the desired properties (Ref. 2.5). These preliminary selected proportions are checked and adjusted by means of trial batches to result in concrete of the desired quality. Inevitably, strength properties of a concrete of given proportions scatter from batch to batch. It is therefore necessary to select proportions that will furnish an average strength sufficiently greater than the specified design strength for even the accidentally weaker batches to be of adequate quality (for details, see Section 2.6). Discussion in detail of practices for proportioning concrete is beyond the scope of this volume; this topic is treated fully in Refs. 2.5 and 2.6, both for stone concrete and for lightweight aggregate concrete.

If the results of trial batches or field experience are not available, the ACI Code allows concrete to be proportioned based on other experience or information, if approved by the registered design professional overseeing the project. This alternative may not be applied for specified compressive strengths greater than 5000 psi.

On all but the smallest jobs, *batching* is carried out in special batching plants. Separate hoppers contain cement and the various fractions of aggregate. Proportions are controlled, by weight, by means of manually operated or automatic scales connected to the hoppers. The mixing water is batched either by measuring tanks or by water meters.

The principal purpose of *mixing* is to produce an intimate mixture of cement, water, fine and coarse aggregate, and possible admixtures of uniform consistency throughout each batch. This is achieved in machine mixers of the revolving-drum type. Minimum mixing time is 1 min for mixers of not more than 1 yd^3 capacity, with an additional 15 sec for each additional 1 yd^3. Mixing can be continued for a considerable time without adverse effect. This fact is particularly important in connection with ready mixed concrete.

On large projects, particularly in the open country where ample space is available, movable mixing plants are installed and operated at the site. On the other hand, in construction under congested city conditions, on smaller jobs, and frequently in highway construction, *ready mixed concrete* is used. Such concrete is batched in a stationary plant and then hauled to the site in trucks in one of three ways: (1) mixed completely at the stationary plant and hauled in a truck agitator, (2) transit-mixed, i.e., batched at the plant but mixed in a truck mixer, or (3) partially mixed at the plant with mixing completed in a truck mixer. Concrete should be discharged from the mixer or agitator within a limited time after the water is added to the batch. Although specifications often provide a single value for all conditions, the maximum mixing time should be based on the concrete temperature because higher temperatures lead to increased rates of *slump loss* and rapid setting. Conversely, lower temperatures increase the period during which the concrete remains workable. A good guide for maximum mixing time is to allow 1 hour at a temperature of 70°F, plus (or minus) 15 min for each 5°F drop (or rise) in concrete temperature for concrete temperatures between 40 and 90°F. Ten minutes may be used at 95°F, the practical upper limit for normal mixing and placing.

Much information on proportioning and other aspects of design and control of concrete mixtures will be found in Ref. 2.7.

2.5 Conveying, Placing, Compacting, and Curing

Conveying of most building concrete from the mixer or truck to the form is done in bottom-dump buckets or by pumping through steel pipelines. The chief danger during conveying is that of *segregation*. The individual components of concrete tend to segregate because of their dissimilarity. In overly wet concrete standing in containers or forms, the heavier gravel components tend to settle, and the lighter materials, particularly water, tend to rise. Lateral movement, such as flow within the forms, tends to separate the coarse gravel from the finer components of the mix.

Placing is the process of transferring the fresh concrete from the conveying device to its final place in the forms. Prior to placing, loose rust must be removed from reinforcement, forms must be cleaned, and hardened surfaces of previous concrete lifts must be cleaned and treated appropriately. Placing and consolidating are critical in their effect on the final quality of the concrete. Proper placement must avoid segregation, displacement of forms or of reinforcement in the forms, and poor bond between successive layers of concrete. Immediately upon placing, the concrete should be *consolidated*, usually by means of vibrators. Consolidation prevents honeycombing, ensures close contact with forms and reinforcement, and serves as a partial remedy to possible prior segregation. Consolidation is achieved by high-frequency, power-driven *vibrators*. These are of the *internal* type, immersed in the concrete, or of the *external* type, attached to the forms. The former are preferable but must be supplemented by the latter where narrow forms or other obstacles make immersion impossible (Ref. 2.8).

Fresh concrete gains strength most rapidly during the first few days and weeks. Structural design is generally based on the *28-day strength*, about 70 percent of which is reached at the end of the first week after placing. The final concrete strength depends greatly on the conditions of moisture and temperature during this initial period. The maintenance of proper conditions during this time is known as *curing*. Thirty percent of the strength or more can be lost by premature drying out of the concrete; similar amounts may be lost by permitting the concrete temperature to drop to 40°F or lower during the first few days unless the concrete is kept continuously moist for a long time thereafter. Freezing of fresh concrete may reduce its strength by 50 percent or more.

To prevent such damage, concrete should be protected from loss of moisture for at least 7 days and, in more sensitive work, up to 14 days. When high early strength cements are used, curing periods can be cut in half. Curing can be achieved by keeping exposed surfaces continually wet through sprinkling, ponding, or covering with plastic film or by the use of sealing compounds, which, when properly used, form evaporation-retarding membranes. In addition to improving strength, proper moist-curing provides better shrinkage control. To protect the concrete against low temperatures during cold weather, the mixing water, and occasionally the aggregates, are heated; temperature insulation is used where possible; and special admixtures are employed. When air temperatures are very low, external heat may have to be supplied in addition to insulation (Refs. 2.7, 2.9, and 2.10).

2.6 Quality Control

The quality of mill-produced materials, such as structural or reinforcing steel, is assured by the producer, who must exercise systematic quality controls, usually specified by pertinent ASTM standards. Concrete, in contrast, is produced at or close to the site, and its final qualities are affected by a number of factors, which have been discussed briefly. Thus, systematic quality control must be instituted at the construction site.

The main measure of the structural quality of concrete is its *compressive strength*. Tests for this property are made on cylindrical specimens of height equal to twice the diameter, usually 6×12 in. Impervious molds of this shape are filled with concrete during the operation of placement as specified by ASTM C 172, "Standard Method of Sampling Freshly Mixed Concrete," and ASTM C 31, "Standard Practice for Making and Curing Concrete Test Specimens in the Field." The cylinders are moist-cured at about 70°F, generally for 28 days, and then tested in the laboratory at a specified rate of loading. The compressive strength obtained from such tests is known as the *cylinder strength* f_c' and is the main property specified for design purposes.

To provide structural safety, continuous control is necessary to ensure that the strength of the concrete as furnished is in satisfactory agreement with the value called for by the designer. The ACI Code specifies that a pair of cylinders must be tested for each 150 yd^3 of concrete or for each 5000 ft^2 of surface area actually placed, but not less than once a day. As mentioned in Section 2.4, the results of strength tests of different batches mixed to identical proportions show inevitable scatter. The scatter can be reduced by closer control, but occasional tests below the cylinder strength specified in the design cannot be avoided. To ensure adequate concrete strength in spite of such scatter, the ACI Code stipulates that concrete quality is satisfactory if (1) no individual strength test result (the average of a pair of cylinder tests) falls below the required f_c' by more than 500 psi when f_c' is 5000 psi or less or by more than $0.10 f_c'$ when f_c' is more than 5000 psi, and (2) every arithmetic average of any three consecutive strength tests equals or exceeds f_c'.

It is evident that, if concrete were proportioned so that its mean strength were just equal to the required strength f_c', it would not pass these quality requirements, because about half of its strength test results would fall below the required f_c'. It is therefore necessary to proportion the concrete so that its mean strength f_{cr}', used as the basis for selection of suitable proportions, exceeds the required design strength f_c' by

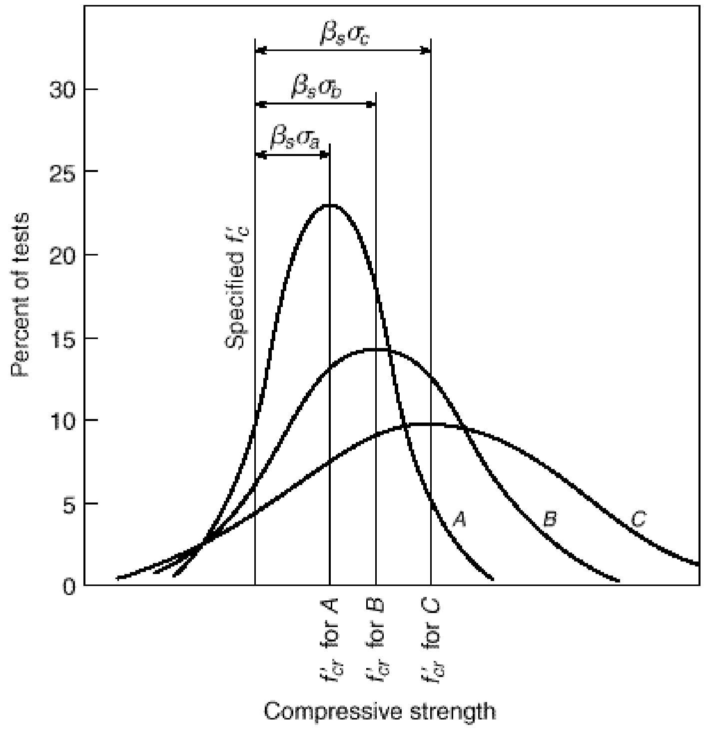


FIGURE 2.2
Frequency curves and average strengths for various degrees of control of concretes with specified design strength f'_c. (*Adapted from Ref. 2.11.*)

an amount sufficient to ensure that the two quoted requirements are met. The minimum amount by which the required mean strength must exceed f'_c can be determined only by statistical methods because of the random nature of test scatter. Requirements have been derived, based on statistical analysis, to be used as a guide to proper proportioning of the concrete at the plant so that the probability of strength deficiency at the construction site is acceptably low.

The basis for these requirements is illustrated in Fig. 2.2, which shows three normal frequency curves giving the distribution of strength test results. The specified design strength is f'_c. The curves correspond to three different degrees of quality control, curve A representing the best control, i.e., the least scatter, and curve C the worst control, with the most scatter. The degree of control is measured statistically by the standard deviation σ (σ_a for curve A, σ_b for curve B, and σ_c for curve C), which is relatively small for producer A and relatively large for producer C. All three distributions have the same probability of strength less than the specified value f'_c, i.e., each has the same fractional part of the total area under the curve to the left of f'_c. For any normal distribution curve, that fractional part is defined by the index β_s, a multiplier applied to the standard deviation σ; β_s is the same for all three distributions of Fig. 2.2. It is seen that, to satisfy the requirement that, say, 1 test in 100 will fall below f'_c (with the value of β_s thus determined), for producer A with the best quality control the mean strength f'_{cr} can be much closer to the specified f'_c than for producer C with the most poorly controlled operation.

On the basis of such studies, the ACI Code requires that concrete production facilities maintain records from which the standard deviation achieved in the particular facility can be determined. It then stipulates the minimum amount by which the required average strength f'_{cr}, aimed at when selecting concrete proportions, must exceed the specified design strength f'_c, depending on the standard deviation $\cdot$ as follows:

$$f'_{cr} = f'_c + 1.34\cdot \tag{2.1}$$

or

$$f'_{cr} = f'_c + 2.33\cdot - 500 \qquad \text{for } f'_c \leq 5000 \text{ psi} \tag{2.2a}$$

$$f'_{cr} = 0.90 f'_c + 2.33\cdot \qquad \text{for } f'_c \cdot 5000 \text{ psi} \tag{2.2b}$$

Equation (2.1) provides a probability of 1 in 100 that averages of three consecutive tests will be below the specified strength f'_c. Equations (2.2*a*) and (2.2*b*) provide a probability of 1 in 100 that an individual strength test will be more than 500 psi below the specified f'_c for f'_c up to 5000 psi or below $0.90 f'_c$ for f'_c over 5000 psi. If no adequate record of concrete plant performance is available, the average strength must exceed f'_c by at least 1000 psi for f'_c of 3000 psi, by at least 1200 psi for f'_c between 3000 and 5000 psi, and by $0.10 f'_c + 700$ psi for f'_c over 5000 psi, according to the ACI Code.

It is seen that this method of control recognizes the fact that occasional deficient batches are inevitable. The requirements ensure (1) a small probability that such strength deficiencies as are bound to occur will be large enough to represent a serious danger and (2) an equally small probability that a sizable portion of the structure, as represented by three consecutive strength tests, will be made of below-par concrete.

In spite of scientific advances, building in general, and concrete making in particular, retain some elements of an art; they depend on many skills and imponderables. It is the task of systematic *inspection* to ensure close correspondence between plans and specifications and the finished structure. Inspection during construction should be carried out by a competent engineer, preferably the one who produced the design or one who is responsible to the design engineer. The inspector's main functions in regard to materials quality control are sampling, examination, and field testing of materials; control of concrete proportioning; inspection of batching, mixing, conveying, placing, compacting, and curing; and supervision of the preparation of specimens for laboratory tests. In addition, the inspector must inspect foundations, formwork, placing of reinforcing steel, and other pertinent features of the general progress of work; keep records of all the inspected items; and prepare periodic reports. The importance of thorough inspection to the correctness and adequate quality of the finished structure cannot be emphasized too strongly.

This brief account of concrete technology represents the merest outline of an important subject. Anyone in practice who is actually responsible for any of the phases of producing and placing concrete must be familiar with the details in much greater depth.

2.7 Admixtures

In addition to the main components of concretes, *admixtures* are often used to improve concrete performance. There are admixtures to accelerate or retard setting and hardening, to improve workability, to increase strength, to improve durability, to decrease permeability, and to impart other properties (Ref. 2.12). The beneficial effects of particular admixtures are well established. Chemical admixtures should meet the requirements of ASTM C 494, "Standard Specification for Chemical Admixtures for Concrete."

Air-entraining agents are probably the most commonly used admixtures at the present time. They cause the entrainment of air in the form of small dispersed bubbles in the concrete. These improve workability and durability (chiefly resistance to freezing and thawing), and reduce segregation during placing. They decrease concrete density because of the increased void ratio and thereby decrease strength; however, this decrease can be partially offset by a reduction of mixing water without loss of workability. The chief use of air-entrained concretes is in pavements, but they are also used for structures, particularly for exposed elements (Ref. 2.13).

Accelerating admixtures are used to reduce setting time and accelerate early strength development. Calcium chloride is the most widely used accelerator because of its cost effectiveness, but it should not be used in prestressed concrete and should be used with caution in reinforced concrete in a moist environment, because of its tendency to promote corrosion of steel. Nonchloride, noncorrosive accelerating admixtures are available (Ref. 2.12).

Set-retarding admixtures are used primarily to offset the accelerating effect of high ambient temperature and to keep the concrete workable during the entire placing period. This helps to eliminate cracking due to form deflection and also keeps concrete workable long enough that succeeding lifts can be placed without the development of "cold" joints.

Certain organic and inorganic compounds are used to reduce the water requirement of a concrete mix for a given slump. Such compounds are termed *plasticizers*. Reduction in water demand may result in either a reduction in the water-cement ratio for a given slump and cement content, or an increase in slump for the same water-cement ratio and cement content. Plasticizers work by reducing the interparticle forces that exist between cement grains in the fresh paste, thereby increasing the paste fluidity. High-range water-reducing admixtures, or *superplasticizers*, are used to produce high-strength concrete (see Section 2.12) with a very low water-cement ratio while maintaining the higher slumps needed for proper placement and compaction of the concrete. They are also used to produce flowable concrete at conventional water-cement ratios. Superplasticizers differ from conventional water-reducing admixtures in that they do not act as retarders at high dosages; therefore, they can be used at higher dosage rates without severely slowing hydration (Refs. 2.12, 2.14, and 2.15). The specific effects of water-reducing admixtures vary with different cements, changes in water-cement ratio, mixing temperature, ambient temperature, and other job conditions, and trial batches are generally required.

Fly ash and *silica fume* are pozzolans, highly active silicas, that combine with calcium hydroxide, the soluble product of cement hydration (Section 2.2), to form more calcium silicate hydrate, the insoluble product of cement hydration. Pozzolans qualify as *mineral admixtures*, which are used to replace a part of the portland cement in concrete mixes. Fly ash, which is specified under ASTM C 618, "Standard Specification for Coal Fly Ash and Raw or Calcified Natural Pozzolan for Use as a Mineral Admixture in Concrete," is precipitated electrostatically as a by-product of the exhaust fumes of coal-fired power stations. It is very finely divided and reacts with calcium hydroxide in the presence of moisture to form a cementitious material. It tends to increase the strength of concrete at ages over 28 days. Silica fume, which is specified under ASTM C 1240, "Standard Specification for Use of Silica Fume as a Mineral Admixture in Hydraulic Cement Concrete, Mortar, and Grout," is a by-product resulting from the manufacture, in electric-arc furnaces, of ferro-silicon alloys and silicon metal. It is extremely finely divided and is highly cementitious when combined with portland cement. In contrast to fly ash, silica fume contributes mainly to strength gain at early ages, from 3 to 28 days. Both fly ash and silica fume, particularly the latter, have been important in the production of high-strength concrete (see Section 2.12). When either silica fume or fly ash, or both, are used, it is customary to refer to the *water–cementitious materials ratio* rather than the water-cement ratio. This typically may be as low as 0.25 for high-strength concrete, and ratios as low as 0.21 have been used (Refs. 2.16 and 2.17).

2.8 PROPERTIES IN COMPRESSION

a. Short-Term Loading

Performance of a structure under load depends to a large degree on the stress-strain relationship of the material from which it is made, under the type of stress to which the material is subjected in the structure. Since concrete is used mostly in compression, its compressive stress-strain curve is of primary interest. Such a curve is obtained by appropriate strain measurements in cylinder tests (Section 2.6) or on the compression side in beams. Figure 2.3 shows a typical set of such curves for normal-density concrete, obtained from uniaxial compressive tests performed at normal, moderate testing speeds on concretes that are 28 days old. Figure 2.4 shows corresponding curves for lightweight concretes having a density of 100 pcf.

All of the curves have somewhat similar character. They consist of an initial relatively straight elastic portion in which stress and strain are closely proportional, then begin to curve to the horizontal, reaching the maximum stress, i.e., the compressive strength, at a strain that ranges from about 0.002 to 0.003 for normal-density concretes, and from about 0.003 to 0.0035 for lightweight concretes (Refs. 2.18 and 2.19), the larger values in each case corresponding to the higher strengths. All curves show

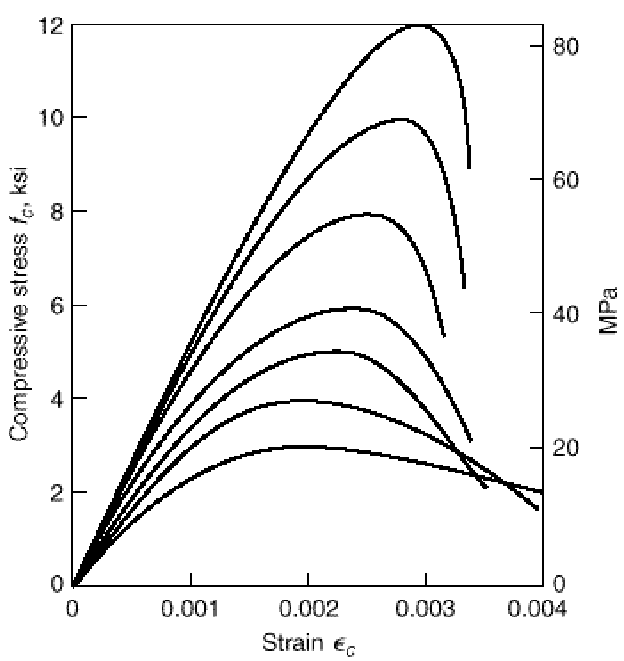


FIGURE 2.3
Typical compressive stress-strain curves for normal-density concrete with $w_c = 145$ pcf. (*Adapted from Refs. 2.18 and 2.19.*)

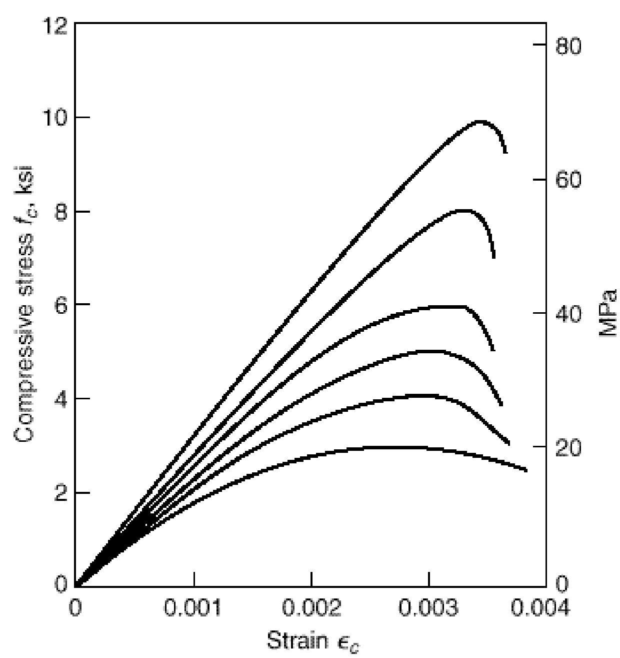


FIGURE 2.4
Typical compressive stress-strain curves for lightweight concrete with $w_c = 100$ pcf. (*Adapted from Refs. 2.18 and 2.19.*)

a descending branch after the peak stress is reached; however, the characteristics of the curves after peak stress are highly dependent upon the method of testing. If special procedures are followed in testing to ensure a constant strain rate while cylinder resistance is decreasing, long stable descending branches can be obtained (Ref. 2.20). In the absence of special devices, unloading past the point of peak stress may be rapid, particularly for the higher-strength concretes, which are generally more brittle than low-strength concrete.

In present practice, the specified compressive strength f'_c is commonly in the range from 3000 to 5000 psi for normal density cast-in-place concrete, and up to about 8000 psi for precast prestressed concrete members. Lightweight concrete strengths are somewhat below these values generally. The high-strength concretes, with f'_c to 15,000 psi or more, are used with increasing frequency, particularly for heavily loaded columns in high-rise concrete buildings and for long-span bridges (mostly prestressed) where a significant reduction in dead load may be realized by minimizing member cross-section dimensions. (See Section 2.12.)

The *modulus of elasticity* E_c (in psi units), i.e., the slope of the initial straight portion of the stress-strain curve, is seen to be larger the higher the strength of the concrete. For concretes in the strength range to about 6000 psi, it can be computed with reasonable accuracy from the empirical equation found in the ACI Code:

$$E_c = 33 w_c^{1.5} \sqrt{f'_c} \tag{2.3}$$

where w_c is the unit weight of the hardened concrete in pcf and f'_c is its strength in psi. Equation (2.3) was obtained by testing structural concretes with values of w_c from 90 to 155 pcf. For normal sand-and-stone concretes, with $w_c = 145$ pcf, E_c may be taken as

$$E_c = 57{,}000 \sqrt{f'_c} \tag{2.4}$$

For compressive strengths in the range from 6000 to 12,000 psi, the ACI Code equation may overestimate E_c for both normal-weight and lightweight material by as much as 20 percent. Based on research at Cornell University (Refs. 2.18 and 2.19), the following equation is recommended for normal-density concretes with f'_c in the range of 3000 to 12,000 psi, and for lightweight concretes from 3000 to 9000 psi:

$$E_c = \left(40{,}000 \sqrt{f'_c} + 1{,}000{,}000\right)\left(\frac{w_c}{145}\right)^{1.5} \tag{2.5}$$

where terms and units are as defined above for the ACI Code equations. When coarse aggregates with high moduli of elasticity are used, however, Eq. (2.4) may *underestimate* E_c. Thus, in cases where E_c is a key design criterion, it should be measured, rather than estimated using Eq. (2.3), (2.4), or (2.5).

Information on concrete strength properties such as those discussed is usually obtained through tests made 28 days after placing. However, cement continues to hydrate, and consequently concrete continues to harden, long after this age, at a decreasing rate. Figure 2.5 shows a typical curve of the gain of concrete strength with age for concrete made using Type I (normal) cement and also Type III (high early strength) cement, each curve normalized with respect to the 28-day compressive strength. High early strength cements produce more rapid strength gain at early ages, although the rate of strength gain at later ages is generally less. Concretes using Type III cement are often used in precasting plants, and often the strength f'_c is specified at 7 days, rather than 28 days.

It should be noted that the shape of the stress-strain curve for various concretes of the same cylinder strength, and even for the same concrete under various conditions

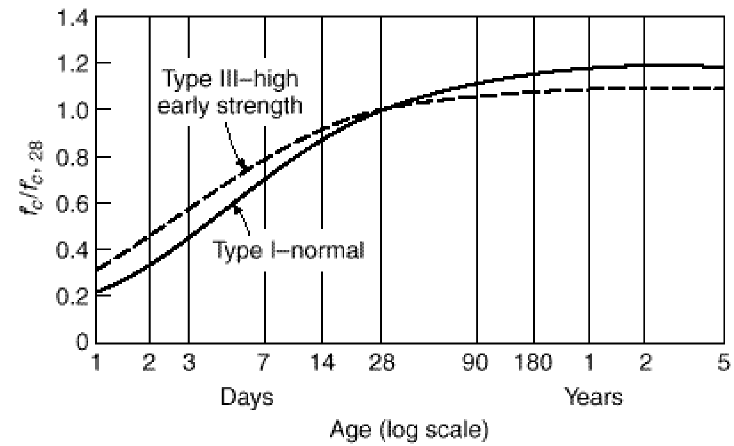


FIGURE 2.5
Effect of age on compressive strength f_c' for moist-cured concrete. (*Adapted from Ref. 2.21.*)

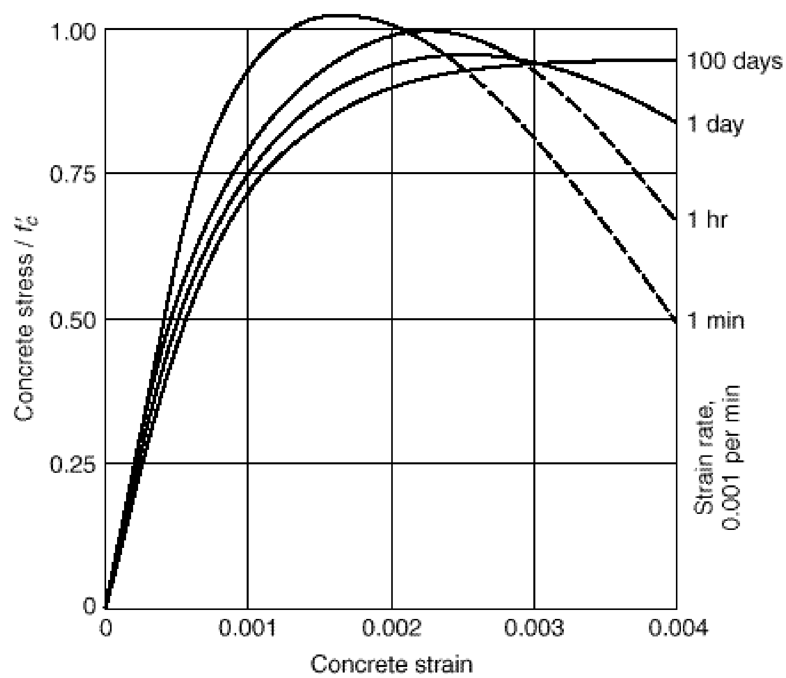


FIGURE 2.6
Stress-strain curves at various strain rates, concentric compression. (*Adapted from Ref. 2.22.*)

of loading, varies considerably. An example of this is shown in Fig. 2.6, where different specimens of the same concrete are loaded at different rates of strain, from one corresponding to a relatively fast loading (0.001 per min) to one corresponding to an extremely slow application of load (0.001 per 100 days). It is seen that the descending branch of the curve, indicative of internal disintegration of the material, is much more pronounced at fast than at slow rates of loading. It is also seen that the peaks of the curves, i.e., the maximum strengths reached, are somewhat smaller at slower rates of strain.

When compressed in one direction, concrete, like other materials, expands in the direction transverse to that of the applied stress. The ratio of the transverse to the longitudinal strain is known as *Poisson's ratio* and depends somewhat on strength, composition, and other factors. At stresses lower than about $0.7f'_c$, Poisson's ratio for concrete falls within the limits of 0.15 to 0.20.

b. Long-Term Loading

In some engineering materials, such as steel, strength and the stress-strain relationships are independent of rate and duration of loading, at least within the usual ranges of rate of stress, temperature, and other variables. In contrast, Fig. 2.6 illustrates the fact that the influence of time, in this case of rate of loading, on the behavior of concrete under load is pronounced. The main reason for this is that concrete creeps under load, while steel does not exhibit creep under conditions prevailing in buildings, bridges, and similar structures.

Creep is the slow deformation of a material over considerable lengths of time at constant stress or load. The nature of the creep process is shown schematically in Fig. 2.7. This particular concrete was loaded after 28 days with resulting instantaneous strain · $_{inst}$. The load was then maintained for 230 days, during which time creep was seen to have increased the total deformation to almost 3 times its instantaneous value. If the load were maintained, the deformation would follow the solid curve. If the load is removed, as shown by the dashed curve, most of the elastic instantaneous strain · $_{inst}$ is recovered, and some creep recovery is seen to occur. If the concrete is reloaded at some later date, instantaneous and creep deformations develop again, as shown.

Creep deformations for a given concrete are practically proportional to the magnitude of the applied stress; at any given stress, high-strength concretes show less creep than lower-strength concretes. As seen in Fig. 2.7, with elapsing time, creep proceeds at a decreasing rate and ceases after 2 to 5 years at a final value which, depending on concrete strength and other factors, is about 1.2 to 3 times the magnitude of the

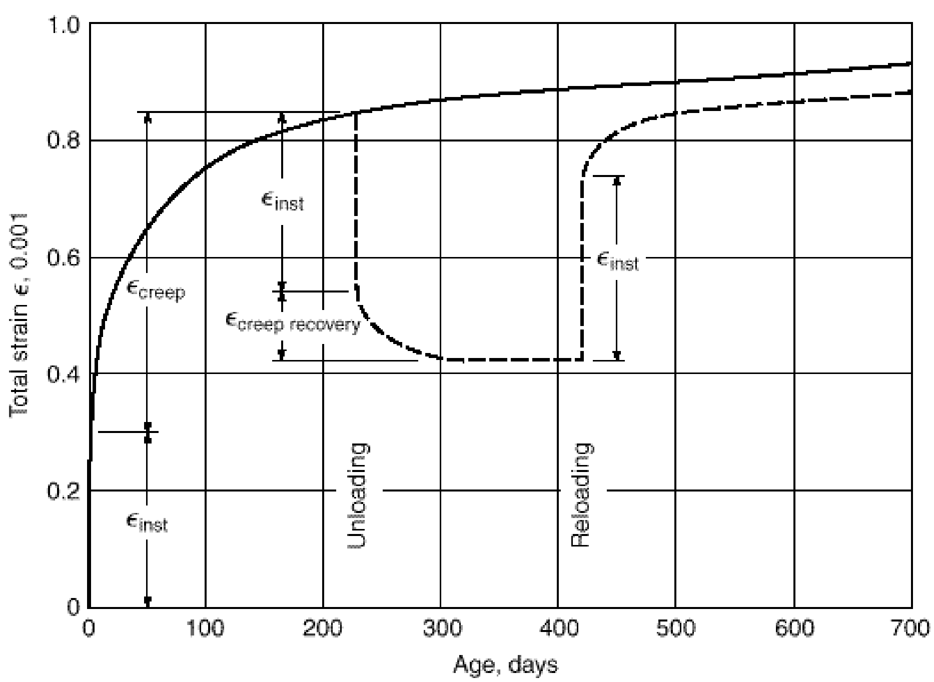


FIGURE 2.7
Typical creep curve (concrete loaded to 600 psi at age 28 days).

instantaneous strain. If, instead of being applied quickly and thereafter kept constant, the load is increased slowly and gradually, as is the case in many structures during and after construction, instantaneous and creep deformations proceed simultaneously. The effect is that shown in Fig. 2.6; i.e., the previously discussed difference in the shape of the stress-strain curve for various rates of loading is chiefly the result of the creep deformation of concrete.

For stresses not exceeding about half the cylinder strength, creep strains are approximately proportional to stress. Because initial elastic strains are also proportional to stress in this range, this permits definition of the *creep coefficient*:

$$C_{cu} = \frac{\epsilon_{cu}}{\epsilon_{ci}} \tag{2.6}$$

where ϵ_{cu} is the final asymptotic value of the additional creep strain and ϵ_{ci} is the initial, instantaneous strain when the load is first applied. Creep may also be expressed in terms of the *specific creep* δ_{cu}, defined as the additional time-dependent strain per psi stress. It can easily be shown that

$$C_{cu} = E_c \delta_{cu} \tag{2.7}$$

In addition to the stress level, creep depends on the average ambient relative humidity, being more than twice as large for 50 percent as for 100 percent humidity (Ref. 2.23). This is so because part of the reduction in volume under sustained load is caused by outward migration of free pore water, which evaporates into the surrounding atmosphere. Other factors of importance include the type of cement and aggregate, age of the concrete when first loaded, and concrete strength (Ref. 2.23). The creep coefficient for high-strength concrete is much less than for low-strength concrete. However, sustained load stresses are apt to be higher so that the creep *deformation* may be as great for high-strength concrete, even though the creep coefficient is less.

The values of Table 2.1, quoted from Ref. 2.24 and extended for high-strength concrete based on research at Cornell University, are typical values for average humidity conditions, for concretes loaded at the age of 7 days.

To illustrate, if the concrete in a column with $f'_c = 4000$ psi is subject to a long-time load that causes sustained stress of 1200 psi, then after several years under load the final value of the creep strain will be about $1200 \times 0.80 \times 10^{-6} = 0.00096$. Thus, if the column were 20 ft long, creep would shorten it by about $\frac{1}{4}$ in.

TABLE 2.1
Typical creep parameters

Compressive Strength		Specific Creep δ_{cu}		
psi	MPa	10^{-6} per psi	10^{-6} per MPa	Creep coefficient C_{cu}
3000	21	1.00	145	3.1
4000	28	0.80	116	2.9
6000	41	0.55	80	2.4
8000	55	0.40	58	2.0
10,000	69	0.28	41	1.6
12,000	83	0.22	33	1.4

The creep coefficient at any time, C_{ct}, can be related to the ultimate creep coefficient C_{cu}. In Ref. 2.21, Branson suggests the equation:

$$C_{ct} = \frac{t^{0.60}}{10 + t^{0.60}} C_{cu} \tag{2.8}$$

where t = time in days after loading.

In many special situations, e.g., slender members or frames, or in prestressed construction, the designer must take account of the combined effects of creep and shrinkage (Section 2.11). In such cases, rather than relying on the sample values of Table 2.1, more accurate information on creep parameters should be obtained, such as from Ref. 2.21 or 2.24.

Sustained loads affect not only the deformation but also the strength of concrete. The cylinder strength f'_c is determined at normal rates of test loading (about 35 psi per sec). Tests by Rüsch (Ref. 2.22) and at Cornell University (Refs. 2.25 and 2.26) have shown that, for concentrically loaded unreinforced concrete prisms and cylinders, the *strength under sustained load* is significantly smaller than f'_c, on the order of 75 percent of f'_c for loads maintained for a year or more. Thus, a member subjected to a sustained overload causing compressive stress of over 75 percent of f'_c may fail after a period of time, even though the load is not increased.

c. Fatigue

When concrete is subject to fluctuating rather than sustained loading, its *fatigue strength*, as for all other materials, is considerably smaller than its static strength. When plain concrete in compression is stressed cyclically from zero to maximum stress, its fatigue limit is from 50 to 60 percent of the static compressive strength, for 2,000,000 cycles. A reasonable estimate can be made for other stress ranges using the modified Goodman diagram (see Ref. 2.24). For other types of applied stress, such as flexural compressive stress in reinforced concrete beams or flexural tension in unreinforced beams or on the tension side of reinforced beams, the fatigue limit likewise appears to be about 55 percent of the corresponding static strength. These figures, however, are for general guidance only. It is known that the fatigue strength of concrete depends not only on its static strength but also on moisture condition, age, and rate of loading (see Ref. 2.27).

2.9 Properties in Tension

While concrete is best employed in a manner that uses its favorable compressive strength, its behavior in tension is also important. The conditions under which cracks form and propagate on the tension side of reinforced concrete flexural members depend strongly on both the tensile strength and the fracture properties of the concrete, the latter dealing with the ease with which a crack progresses once it has formed. Concrete tensile stresses also occur as a result of shear, torsion, and other actions, and in most cases member behavior changes upon cracking. Thus, it is important to be able to predict, with reasonable accuracy, the tensile strength of concrete and to understand the factors that control crack propagation.

a. Tensile Strength

There are considerable experimental difficulties in determining the true tensile strength of concrete. In *direct tension* tests, minor misalignments and stress concentrations in the gripping devices are apt to mar the results. For many years, tensile strength has been measured in terms of the *modulus of rupture* f_r, the computed flexural tensile stress at which a test beam of plain concrete fractures. Because this nominal stress is computed on the assumption that concrete is an elastic material, and because this bending stress is localized at the outermost surface, it is apt to be larger than the strength of concrete in uniform axial tension. It is thus a measure of, but not identical with, the real axial tensile strength.

More recently the result of the so-called *split-cylinder test* has established itself as a measure of the tensile strength of concrete. A 6 × 12 in. concrete cylinder, the same as is used for compressive tests, is inserted in a compression testing machine in the horizontal position, so that compression is applied uniformly along two opposite generators. Pads are inserted between the compression platens of the machine and the cylinder to equalize and distribute the pressure. It can be shown that in an elastic cylinder so loaded, a nearly uniform tensile stress of magnitude $2P/\pi dL$ exists at right angles to the plane of load application. Correspondingly, such cylinders, when tested, split into two halves along that plane, at a stress f_{ct} that can be computed from the above expression. P is the applied compressive load at failure, and d and L are the diameter and length of the cylinder respectively. Because of local stress conditions at the load lines and the presence of stresses at right angles to the aforementioned tension stresses, the results of the split-cylinder tests likewise are not identical with (but are believed to be a good measure of) the true axial tensile strength. The results of all types of tensile tests show considerably more scatter than those of compression tests.

Tensile strength, however determined, does not correlate well with the compressive strength f'_c. It appears that for sand-and-gravel concrete, the tensile strength depends primarily on the strength of bond between hardened cement paste and aggregate, whereas for lightweight concretes it depends largely on the tensile strength of the porous aggregate. The compressive strength, on the other hand, is much less determined by these particular characteristics.

Better correlation is found between the various measures of tensile strength and the square root of the compressive strength. The direct tensile strength, for example, ranges from about 3 to 5$\sqrt{f'_c}$ for normal-density concretes, and from about 2 to 3$\sqrt{f'_c}$ for all-lightweight concrete. Typical ranges of values for direct tensile strength, split-cylinder strength, and modulus of rupture are summarized in Table 2.2. In these expressions, f'_c is expressed in psi units, and the resulting tensile strengths are obtained in psi.

These approximate expressions show that tensile and compressive strengths are by no means proportional, and that any increase in compressive strength, such as that

TABLE 2.2
Approximate range of tensile strengths of concrete

	Normal-Weight Concrete, psi	Lightweight Concrete, psi
Direct tensile strength f'_t	3 to 5$\sqrt{f'_c}$	2 to 3$\sqrt{f'_c}$
Split-cylinder strength f_{ct}	6 to 8$\sqrt{f'_c}$	4 to 6$\sqrt{f'_c}$
Modulus of rupture f_r	8 to 12$\sqrt{f'_c}$	6 to 8$\sqrt{f'_c}$

achieved by lowering the water-cement ratio, is accompanied by a much smaller percentage increase in tensile strength.

The ACI Code contains the recommendation that the modulus of rupture f_r' be taken to equal $7.5\sqrt{f_c'}$ for normal-weight concrete, and that this value be multiplied by 0.85 for "sand-lightweight" and 0.75 for "all-lightweight" concretes, giving values of $6.4\sqrt{f_c'}$ and $5.6\sqrt{f_c'}$ respectively for those materials.

b. Tensile Fracture

The failure of concrete in tension involves both the formation and propagation of cracks. The field of fracture mechanics deals with the latter. While reinforced concrete structures have been successfully designed and built for over 150 years without the use of fracture mechanics, the brittle response of high-strength concretes (Section 2.12), in tension as well as compression, increases the importance of the fracture properties of the material as distinct from tensile strength. Research dealing with the shear strength of high-strength concrete beams and the bond between reinforcing steel and high-strength concrete indicates relatively low increases in these structural properties with increases in concrete compressive strength (Refs. 2.28 and 2.29). While shear and bond strength are associated with the $\sqrt{f_c'}$ for normal-strength concrete, tests of high-strength concrete indicate that increases in shear and bond strengths are well below values predicted using $\sqrt{f_c'}$, indicating that concrete tensile strength alone is not the governing factor. An explanation for this behavior is provided by research at the University of Kansas and elsewhere (Refs. 2.30 and 2.31) that demonstrates that the energy required to fully open a crack (i.e., after the crack has started to grow) is largely independent of compressive strength, water-cement ratio, and age. Design expressions reflecting this research are not yet available. The behavior is, however, recognized in the ACI Code by limitations on the maximum value of $\sqrt{f_c'}$ that may be used to calculate shear and bond strength, as will be discussed in Chapters 4 and 5.

2.10 STRENGTH UNDER COMBINED STRESS

In many structural situations, concrete is subjected simultaneously to various stresses acting in various directions. For instance, in beams much of the concrete is subject simultaneously to compression and shear stresses, and in slabs and footings to compression in two perpendicular directions plus shear. By methods well known from the study of engineering mechanics, any state of combined stress, no matter how complex, can be reduced to three principal stresses acting at right angles to each other on an appropriately oriented elementary cube in the material. Any or all of the principal stresses can be either tension or compression. If any one of them is zero, a state of *biaxial* stress is said to exist; if two of them are zero, the state of stress is *uniaxial*, either simple compression or simple tension. In most cases, only the uniaxial strength properties of a material are known from simple tests, such as the cylinder strength f_c' and the tensile strength f_t'. For predicting the strengths of structures in which concrete is subject to biaxial or triaxial stress, it would be desirable to be able to calculate the strength of concrete in such states of stress, knowing from tests only either f_c' or f_c' and f_t'.

In spite of extensive and continuing research, no general theory of the strength of concrete under combined stress has yet emerged. Modifications of various strength theories, such as maximum stress, maximum strain, the Mohr-Coulomb, and the octahedral shear stress theories, all of which are discussed in structural mechanics texts, have been adapted with varying partial success to concrete (Refs. 2.32 to 2.36). At present, none of these theories has been generally accepted, and many have obvious internal contradictions. The main difficulty in developing an adequate general strength theory lies in the highly nonhomogeneous nature of concrete, and in the degree to which its behavior at high stresses and at fracture is influenced by microcracking and other discontinuity phenomena (Ref. 2.37).

However, the strength of concrete has been well established by tests, at least for the biaxial stress state (Refs. 2.38 and 2.39). Results may be presented in the form of an interaction diagram such as Fig. 2.8, which shows the strength in direction 1 as a function of the stress applied in direction 2. All stresses are normalized in terms of the uniaxial compressive strength f_c'. It is seen that in the quadrant representing biaxial compression a strength increase as great as about 20 percent over the uniaxial compressive strength is attained, the amount of increase depending upon the ratio of f_2 to f_1. In the biaxial tension quadrant, the strength in direction 1 is almost independent of stress in direction 2. When tension in direction 2 is combined with compression in direction 1, the compressive strength is reduced almost linearly, and vice versa. For example, lateral compression of about half the uniaxial compressive strength will reduce the tensile strength by almost half compared with its uniaxial value. This fact is of great importance in predicting diagonal tension cracking in deep beams or shear walls, for example.

Experimental investigations into the triaxial strength of concrete have been few, due mainly to the practical difficulty of applying load in three directions simultaneously without introducing significant restraint from the loading equipment (Ref. 2.40). From information now available, the following tentative conclusions can be drawn rel-

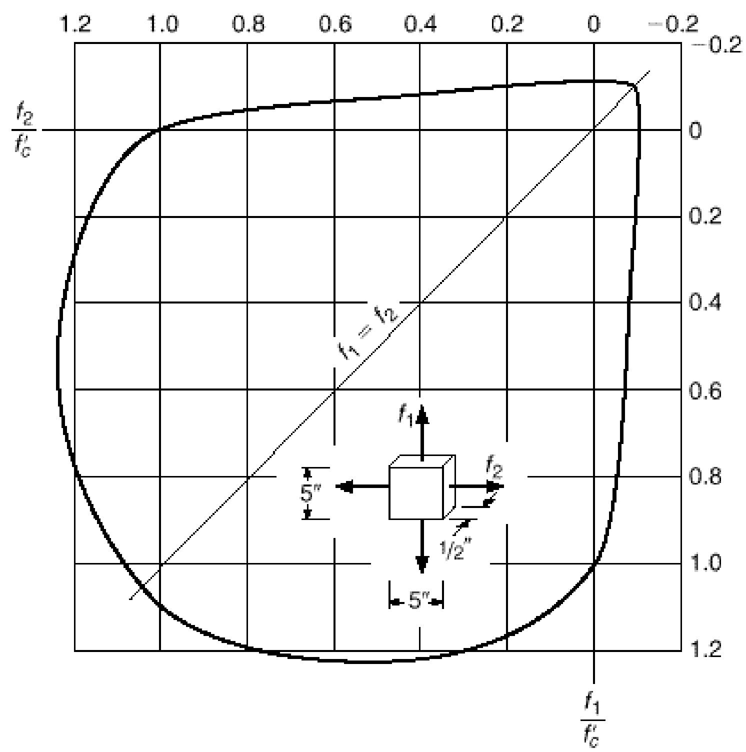


FIGURE 2.8
Strength of concrete in biaxial stress. (*Adapted from Ref. 2.39.*)

ative to the triaxial strength of concrete: (1) in a state of equal triaxial compression, concrete strength may be an order of magnitude larger than the uniaxial compressive strength; (2) for equal biaxial compression combined with a smaller value of compression in the third direction, a strength increase greater than 20 percent can be expected; and (3) for stress states including compression combined with tension in at least one other direction, the intermediate principal stress is of little consequence, and the compressive strength can be predicted safely based on Fig. 2.8.

In fact, the strength of concrete under combined stress cannot yet be calculated rationally and, equally important, in many situations in concrete structures it is nearly impossible to calculate all of the acting stresses and their directions; these are two of the main reasons for continued reliance on tests. Because of this, the design of reinforced concrete structures continues to be based more on extensive experimental information than on consistent analytical theory, particularly in the many situations where combined stresses occur.

2.11 Shrinkage and Temperature Effects

The deformations discussed in Section 2.8 were induced by stresses caused by external loads. Influences of a different nature cause concrete, even when free of any external loading, to undergo deformations and volume changes. The most important of these are shrinkage and the effects of temperature variations.

a. Shrinkage

As discussed in Sections 2.2 and 2.4, any workable concrete mix contains more water than is needed for hydration. If the concrete is exposed to air, the larger part of this free water evaporates in time, the rate and completeness of drying depending on ambient temperature and humidity conditions. As the concrete dries, it shrinks in volume, probably due to the capillary tension that develops in the water remaining in the concrete. Conversely, if dry concrete is immersed in water, it expands, regaining much of the volume loss from prior shrinkage. Shrinkage, which continues at a decreasing rate for several months, depending on the configuration of the member, is a detrimental property of concrete in several respects. When not adequately controlled, it will cause unsightly and often deleterious cracks, as in slabs, walls, etc. In structures that are statically indeterminate (and most concrete structures are), it can cause large and harmful stresses. In prestressed concrete it leads to partial loss of initial prestress. For these reasons it is essential that shrinkage be minimized and controlled.

As is clear from the nature of the process, a key factor in determining the amount of final shrinkage is the unit water content of the fresh concrete. This is illustrated in Fig. 2.9, which shows the amount of shrinkage for varying amounts of mixing water. The same aggregates were used for all tests, but in addition to and independently of the water content, the amount of cement was also varied from 376 to 1034 lb/yd^3 of concrete. This very large variation of cement content causes a 20 to 30 percent variation in shrinkage strain for water contents between 250 to 350 lb/yd^3, the range used for most structural concretes. Increasing the cement content increases the cement paste constituent of the concrete, where the shrinkage actually takes place, while reducing the aggregate content. Since most aggregates do not contribute to shrinkage, an increase in aggregate content can significantly decrease shrinkage. This is shown in Fig. 2.10, which compares the shrinkage of concretes with various aggregate contents

FIGURE 2.9
Effect of water content on drying shrinkage. (*From Ref. 2.4.*)

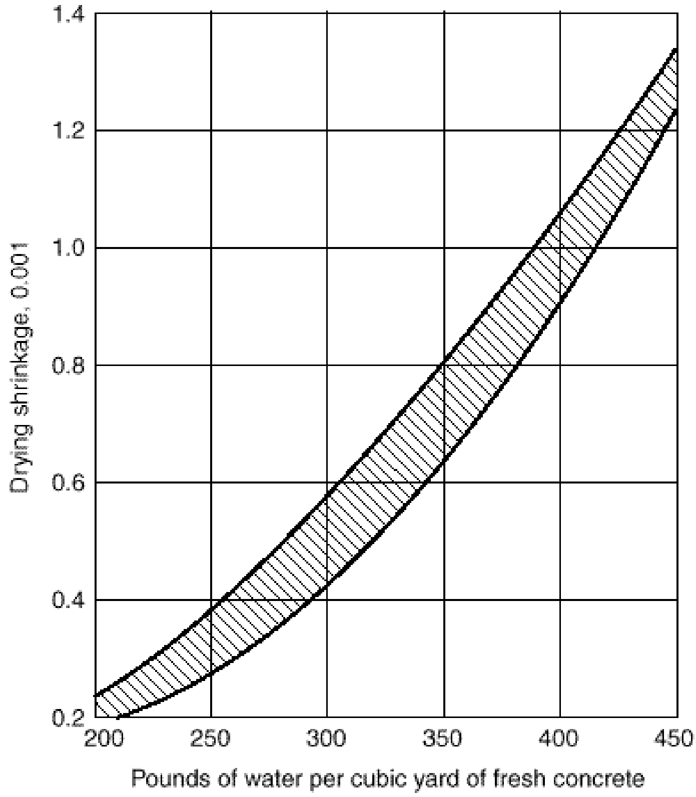
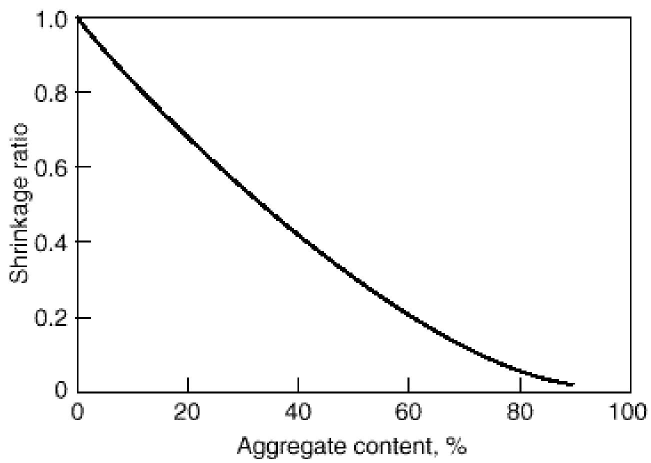


FIGURE 2.10
Influence of aggregate content in concrete (by volume) on the ratio of the shrinkage of concrete to the shrinkage of neat cement paste. (*Adapted from Ref. 2.24, based on data in Ref. 2.41.*)



with the shrinkage obtained for neat cement paste (cement and water alone). For example, increasing the aggregate content from 71 to 74 percent (at the same water-cement ratio) results in a 20 percent reduction in shrinkage (Ref. 2.24). Increased aggregate content may be obtained through the use of (1) a larger maximum size coarse aggregate (which also reduces the water content required for a given workability), (2) a concrete with lower workability, and (3) chemical admixtures to increase workability at lower water contents. It is evident that an effective means of reducing shrinkage involves both a reduction in water content and an increase in aggregate content. In addition, prolonged and careful curing is beneficial for shrinkage control.

Values of final shrinkage for ordinary concretes are generally on the order of 400×10^{-6} to 800×10^{-6}, depending on the initial water content, ambient temperature and humidity conditions, and the nature of the aggregate. Highly absorptive aggregates, such as some sandstones and slates, result in shrinkage values 2 and more times those obtained with less absorptive materials, such as granites and some limestones. Some lightweight aggregates, in view of their great porosity, easily result in much larger shrinkage values than ordinary concretes.

For some purposes, such as predicting the time-dependent loss of force in prestressed concrete beams, it is important to estimate the amount of shrinkage as a function of time. Long-term studies (Ref. 2.21) show that, for moist-cured concrete at any time t after the initial 7 days, shrinkage can be predicted satisfactorily by the equation

$$\epsilon_{sh,t} = \frac{t}{35 + t}\,\epsilon_{sh,u} \tag{2.9}$$

where $\epsilon_{sh,t}$ is the unit shrinkage strain at time t in days and $\epsilon_{sh,u}$ is the ultimate value after a long period of time. Equation (2.9) pertains to "standard" conditions, defined in Ref. 2.21 to exist for humidity not in excess of 40 percent and for an average thickness of member of 6 in., and it applies both for normal-weight and lightweight concretes. Modification factors are to be applied for nonstandard conditions, and separate equations are given for steam-cured members.

For structures in which a reduction in cracking is of particular importance, such as bridge decks, pavement slabs, and liquid storage tanks, the use of *expansive cement concrete* is appropriate. Shrinkage-compensating cement is constituted and proportioned such that the concrete will increase in volume after setting and during hardening. When the concrete is restrained by reinforcement or other means, the tendency to expand will result in compression. With subsequent drying, the shrinkage so produced, instead of causing a tension stress in the concrete that would result in cracking, merely reduces or relieves the expansive strains caused by the initial expansion (Ref. 2.42). Expansive cement is produced by adding a source of reactive aluminate to ordinary portland cement; approximately 90 percent of shrinkage-compensating cement is made up of the constituents of conventional portland cement. Of the three main types of expansive cements produced, only type K is commercially available in the United States; it is about 20 percent more expensive than ordinary portland cement (Ref. 2.43). Requirements for expansive cement are given in ASTM C 845, "Standard Specification for Expansive Hydraulic Cement." The usual admixtures can be used in shrinkage-compensating concrete, but trial mixes are necessary because some admixtures, particularly air-entraining agents, are not compatible with certain expansive cements.

b. Effect of Temperature Change

Like most other materials, concrete expands with increasing temperature and contracts with decreasing temperature. The effects of such volume changes are similar to those caused by shrinkage, i.e., temperature contraction can lead to objectionable cracking, particularly when superimposed on shrinkage. In indeterminate structures, deformations due to temperature changes can cause large and occasionally harmful stresses.

The coefficient of thermal expansion and contraction varies somewhat, depending upon the type of aggregate and richness of the mix. It is generally within the range of 4×10^{-6} to 7×10^{-6} per °F. A value of 5.5×10^{-6} is generally accepted as satisfactory for calculating stresses and deformations caused by temperature changes (Ref. 2.23).

2.12 High-Strength Concrete

In recent years there has been a rapid growth of interest in *high-strength concrete*. Although the exact definition is arbitrary, the term generally refers to concrete having uniaxial compressive strength in the range of about 8000 to 15,000 psi or higher. Such concretes can be made using carefully selected but widely available cements, sands, and stone; certain admixtures including high-range water-reducing superplasticizers, fly ash, and silica fume; plus very careful quality control during production (Refs. 2.44 and 2.45). In addition to higher strength in compression, most other engineering properties are improved, leading to use of the alternative term *high-performance concrete*.

The most common application of high-strength concretes has been in the columns of tall concrete buildings, where normal concrete would result in unacceptably large cross sections, with loss of valuable floor space. It has been shown that the use of the more expensive high-strength concrete mixes in columns not only saves floor area but is more economical than increasing the amount of steel reinforcement. Concrete of up to 12,000 psi was specified for the lower-story columns of 311 South Wacker Drive in Chicago (see Fig. 2.11), having a total height of 946 ft. Formerly holding the height record, it has been superseded by taller buildings; the present record is held by Central Plaza in Hong Kong, which has a total height of 1230 ft (Ref. 2.46).

For bridges, too, smaller cross sections bring significant advantages, and the resulting reduction in dead load permits longer spans. The higher elastic modulus and lower creep coefficient result in reduced initial and long-term deflections, and in the case of prestressed concrete bridges, initial and time-dependent losses of prestress force are less. Other recent applications of high-strength concrete include offshore oil structures, parking garages, bridge deck overlays, dam spillways, warehouses, and heavy industrial slabs (Ref. 2.47).

An essential requirement for high-strength concrete is a low water-cement ratio. For normal concretes, this usually falls in the range from about 0.40 to 0.60 by weight, but for high-strength mixes it may be 0.25 or even lower. To permit proper placement of what would otherwise be a zero slump mix, high-range water-reducing admixtures, or superplasticizers, are essential and may increase slumps to as much as 6 or 8 in. Other additives include fly ash and, most notably, silica fume (see Section 2.7).

Much research in recent years has been devoted to establishing the fundamental and engineering properties of high-strength concretes, as well as the engineering characteristics of structural members made with the material (Refs. 2.28, 2.29, and 2.48 to 2.54). A large body of information is now available, permitting the engineer to use high-strength concrete with confidence when its advantages justify the higher cost. The compressive strength curves in Figs. 2.3 and 2.4 illustrate important differences compared with normal concrete, including higher elastic modulus and an extended range of linear elastic response; disadvantages include brittle behavior (see Fig. 2.12) and somewhat reduced ultimate strain capacity. Creep coefficients are reduced, as indicated in Table 2.1. Strength under sustained load is a higher fraction of standard cylinder strength (Refs. 2.25 and 2.26), and information now available confirms its

FIGURE 2.11
311 South Wacker Drive, Chicago, which is among the world's tallest buildings. High-strength concrete with f'_c = 12,000 psi was used in the lower stories. *(Courtesy of Portland Cement Association.)*

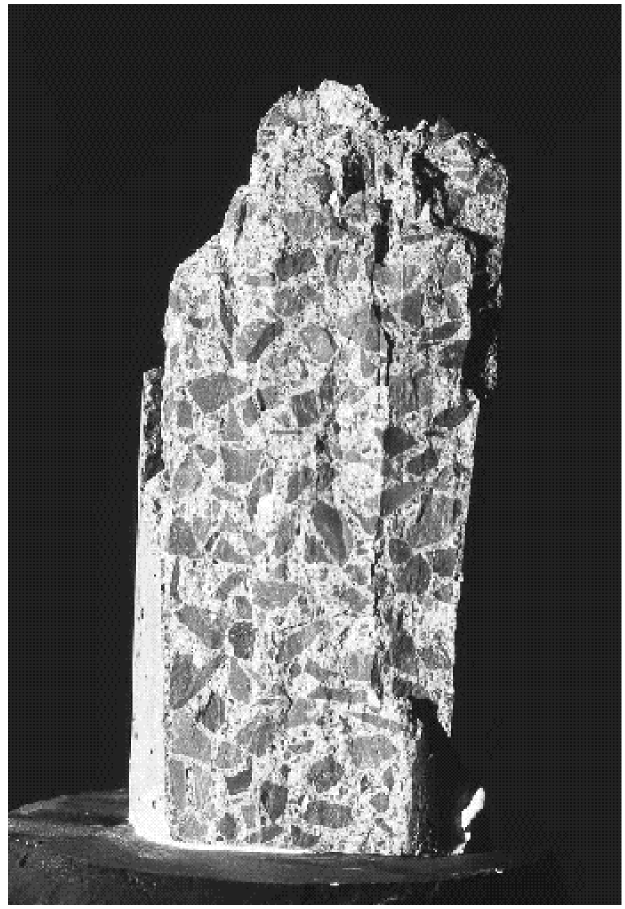


FIGURE 2.12
High-strength concrete test cylinder after uniaxial loading to failure; note the typically smooth fracture surface, with little aggregate interlock.

improved durability and abrasion resistance (Refs. 2.52 and 2.55). As broader experience is gained in practical applications, and as design codes are gradually updated to recognize the special properties of higher-strength concretes now available, much wider use can be expected.

2.13 REINFORCING STEELS FOR CONCRETE

The useful strength of ordinary reinforcing steels in tension as well as compression, i.e., the yield strength, is about 15 times the compressive strength of common structural concrete, and well over 100 times its tensile strength. On the other hand, steel is a high-cost material compared with concrete. It follows that the two materials are best used in combination if the concrete is made to resist the compressive stresses and the steel the tensile stresses. Thus, in reinforced concrete beams, the concrete resists the compressive force, longitudinal steel reinforcing bars are located close to the tension face to resist the tension force, and usually additional steel bars are so disposed that they resist the inclined tension stresses that are caused by the shear force in the beams. However, reinforcement is also used for resisting compressive forces primarily where it is desired to reduce the cross-sectional dimensions of compression members, as in the lower-floor columns of multistory buildings. Even if no such necessity exists, a minimum amount of reinforcement is placed in all compression members to safeguard them against the effects of small accidental bending moments that might crack and even fail an unreinforced member.

For most effective reinforcing action, it is essential that steel and concrete deform together, i.e., that there be a sufficiently strong *bond* between the two materials to ensure that no relative movements of the steel bars and the surrounding concrete occur. This bond is provided by the relatively large *chemical adhesion* that develops at the steel-concrete interface, by the *natural roughness* of the mill scale of hot-rolled reinforcing bars, and by the closely spaced rib-shaped *surface deformations* with which reinforcing bars are furnished in order to provide a high degree of interlocking of the two materials.

Additional features that make for the satisfactory joint performance of steel and concrete are the following:

1. The *thermal expansion coefficients* of the two materials, about 6.5×10^{-6} for steel vs. an average of 5.5×10^{-6} for concrete, are sufficiently close to forestall cracking and other undesirable effects of differential thermal deformations.
2. While the *corrosion resistance* of bare steel is poor, the concrete that surrounds the steel reinforcement provides excellent corrosion protection, minimizing corrosion problems and corresponding maintenance costs.
3. The *fire resistance* of unprotected steel is impaired by its high thermal conductivity and by the fact that its strength decreases sizably at high temperatures. Conversely, the thermal conductivity of concrete is relatively low. Thus, damage caused by even prolonged fire exposure, if any, is generally limited to the outer layer of concrete, and a moderate amount of concrete cover provides sufficient thermal insulation for the embedded reinforcement.

Steel is used in two different ways in concrete structures: as reinforcing steel and as prestressing steel. Reinforcing steel is placed in the forms prior to casting of the concrete. Stresses in the steel, as in the hardened concrete, are caused only by the loads on the structure, except for possible parasitic stresses from shrinkage or similar causes. In contrast, in prestressed concrete structures large tension forces are applied to the reinforcement prior to letting it act jointly with the concrete in resisting external loads. The steels for these two uses are very different and will be discussed separately.

2.14 Reinforcing Bars

The most common type of reinforcing steel (as distinct from prestressing steel) is in the form of round bars, often called *rebars*, available in a large range of diameters from about $\frac{3}{8}$ to $1\frac{3}{8}$ in. for ordinary applications and in two heavy bar sizes of about $1\frac{3}{4}$ and $2\frac{1}{4}$ in. These bars are furnished with surface deformations for the purpose of increasing resistance to slip between steel and concrete. Minimum requirements for these deformations (spacing, projection, etc.) have been developed in experimental research. Different bar producers use different patterns, all of which satisfy these requirements. Figure 2.13 shows a variety of current types of deformations.

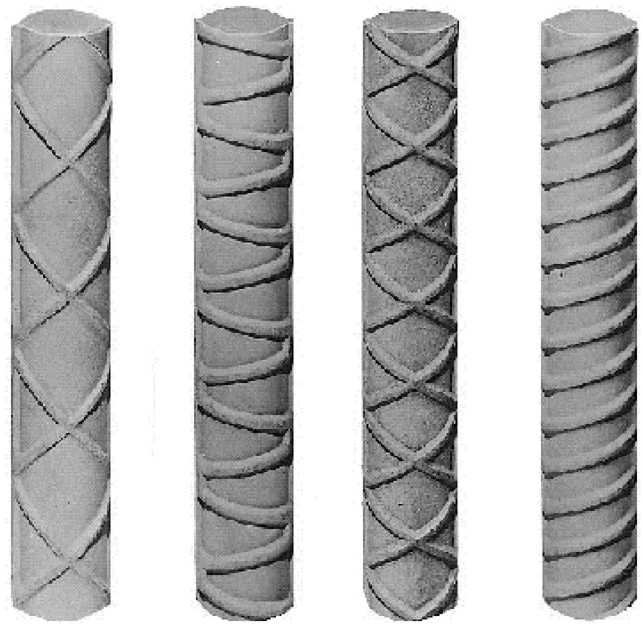


FIGURE 2.13
Types of deformed reinforcing bars.

For many years, bar sizes have been designated by numbers, Nos. 3 to 11 being commonly used and Nos. 14 and 18 representing the two special large-sized bars previously mentioned. Designation by number, instead of by diameter, was introduced because the surface deformations make it impossible to define a single easily measured value of the diameter. The numbers are so arranged that the unit in the number designation corresponds closely to the number of $\frac{1}{8}$ in. of diameter size. A No. 5 bar, for example, has a nominal diameter of $\frac{5}{8}$ in. Bar sizes are rolled into the surface of the bars for easy identification.

For a number of years, ASTM standards have included a second designation for bar size, the International System of Units (SI), with the size being identified using the nominal diameter in millimeters. To limit the number of bar designations, reinforcing bar producers in the United States have converted to the SI system for marking the bars. Thus, Nos. 3 to 11 bars are marked with Nos. 10 to 36, and Nos. 14 and 18 bars with Nos. 43 and 57. Both systems are still used in the ASTM standards, and the older, customary system is used in the 2002 ACI Code. To recognize the dual system of identifying and marking the bars, the customary bar designation system is retained throughout this text, followed by the SI bar designations in parentheses, such as No. 6 (No. 19). Table A.1 of Appendix A gives areas and weights of standard bars. Tables A.2 and A.3 give similar information for groups of bars.

a. Grades and Strengths

In reinforced concrete, a long-term trend is evident toward the use of higher-strength materials, both steel and concrete. Reinforcing bars with 40 ksi yield stress, almost standard 30 years ago, have largely been replaced by bars with 60 ksi yield stress, both because they are more economical and because their use tends to reduce steel congestion in the forms. Bars with yield stress of 75 ksi are used increasingly in columns. Table 2.3 lists all presently available reinforcing steels, their grade designations, the

TABLE 2.3
Summary of minimum ASTM strength requirements

Product	ASTM Specification	Designation	Minimum Yield Strength, psi (MPa)	Minimum Tensile Strength, psi (MPa)
Reinforcing bars	A 615	Grade 40 Grade 60 Grade 75	40,000 (280) 60,000 (420) 75,000 (520)	60,000 (420) 90,000 (620) 100,000 (690)
	A 706	Grade 60	60,000 (420) [78,000 (540) maximum]	80,000 (550)[a]
	A 996	Grade 40 Grade 50 Grade 60	40,000 (280) 50,000 (350) 60,000 (420)	60,000 (420) 80,000 (550) 90,000 (620)
Deformed bar mats	A 184	Same as reinforcing bars		
Zinc-coated bars	A 767	Same as reinforcing bars		
Epoxy-coated bars	A 775, A 934	Same as reinforcing bars		
Stainless-steel bars[b]	A 955	Same as reinforcing bars		
Wire Plain	A 82		70,000 (480)	80,000 (550)
Deformed	A 496		75,000 (515)	85,000 (585)
Welded wire reinforcement Plain W1.2 and larger Smaller than W1.2	A 185		65,000 (450) 56,000 (385)	75,000 (515) 70,000 (485)
Deformed	A 497		70,000 (480)	80,000 (550)
Prestressing tendons Seven-wire strand	A 416	Grade 250 (stress-relieved)	212,500 (1465)	250,000 (1725)
		Grade 250 (low-relaxation)	225,000 (1555)	250,000 (1725)
		Grade 270 (stress-relieved)	229,500 (1580)	270,000 (1860)
		Grade 270 (low-relaxation)	243,000 (1675)	270,000 (1860)
Wire	A 421	Stress-relieved	199,750 (1375) to 212,500 (1465)[c]	235,000 (1620) to 250,000 (1725)[c]
		Low-relaxation	211,500 (1455) to 225,000 (1550)[c]	235,000 (1620) to 250,000 (1725)[c]
Bars	A 722	Type I (plain) Type II (deformed)	127,500 (800) 120,000 (825)	150,000 (1035) 150,000 (1035)
Compacted strand[b]	A 779	Type 245 Type 260 Type 270	241,900 (1480) 228,800 (1575) 234,900 (1620)	247,000 (1700) 263,000 (1810) 270,000 (1860)

[a] But not less than 1.25 times the actual yield strength.
[b] Not listed in ACI 318.
[c] Minimum strength depends on wire size.

ASTM specifications that define their properties (including deformations) in detail, and their two main minimum specified strength values. Grade 40 bars are no longer available in sizes larger than No. 6 (No. 19), Grade 50 bars are available in sizes up to No. 8 (No. 25),[†] and Grade 75 bars are available in sizes from No. 6 (No. 19) upward.

The conversion to SI units described above also applies to the strength grades. Thus, Grade 40 is also designated as Grade 280 (for a yield strength of 280 MPa), Grade 60 is designated Grade 420, and Grade 75 is designated Grade 520. The values, 280, 420, and 520, result in minimum yield strengths of 40.6, 60.9, and 75.4 ksi; i.e., reinforcing steel is slightly stronger than implied by the grade in ksi. Grades based on inch-pound units will be used in this text.

Welding of reinforcing bars in making splices, or for convenience in fabricating reinforcing cages for placement in the forms, may result in metallurgical changes that reduce both strength and ductility, and special restrictions must be placed both on the type of steel used and the welding procedures. The provisions of ASTM A 706 relate specifically to welding.

The ACI Code permits reinforcing steels up to $f_y = 80$ ksi. Such high-strength steels usually yield gradually but have no yield plateau (see Fig. 2.15). In this situation it is required that at the specified minimum yield strength the total strain shall not exceed 0.0035. This is necessary to make current design methods, which were developed for sharp-yielding steels with a yield plateau, applicable to such higher-strength steels. Under special circumstances steel in this higher-strength range has its place, e.g., in lower-story columns of high-rise buildings.

To allow bars of various grades and sizes to be easily distinguished, which is necessary to avoid accidental use of lower-strength or smaller-size bars than called for in the design, all deformed bars are furnished with rolled-in markings. These identify the producing mill (usually with an initial), the bar size (Nos. 3 to 18 under the Inch-Pound system and Nos. 10 to 57 under the SI system), the type of steel (*S* for billet, *W* for low-alloy, a rail sign for rail steel, and *A* for axle, corresponding respectively to ASTM Specifications A 615, A 706, A 996, the latter for both rail and axle steel), and an additional marking to identify higher-strength steels. Grade 60 (420) bars have either one longitudinal line or the number 60 (4); Grade 75 (520) bars have either two longitudinal lines or the number 75 (5). The identification marks are shown in Fig. 2.14. As mentioned earlier, the SI system is used exclusively for bars rolled by mills in the United States.

b. Stress-Strain Curves

The two chief numerical characteristics that determine the character of bar reinforcement are its *yield point* (generally identical in tension and compression) and its *modulus of elasticity* E_s. The latter is practically the same for all reinforcing steels (but not for prestressing steels) and is taken as $E_s = 29{,}000{,}000$ psi.

In addition, however, the shape of the stress-strain curve, and particularly of its initial portion, has significant influence on the performance of reinforced concrete members. Typical stress-strain curves for American reinforcing steels are shown in Fig. 2.15. The complete stress-strain curves are shown in the left part of the figure; the right part gives the initial portions of the curves magnified 10 times.

[†] In practice, very little Grade 50 reinforcement is produced.

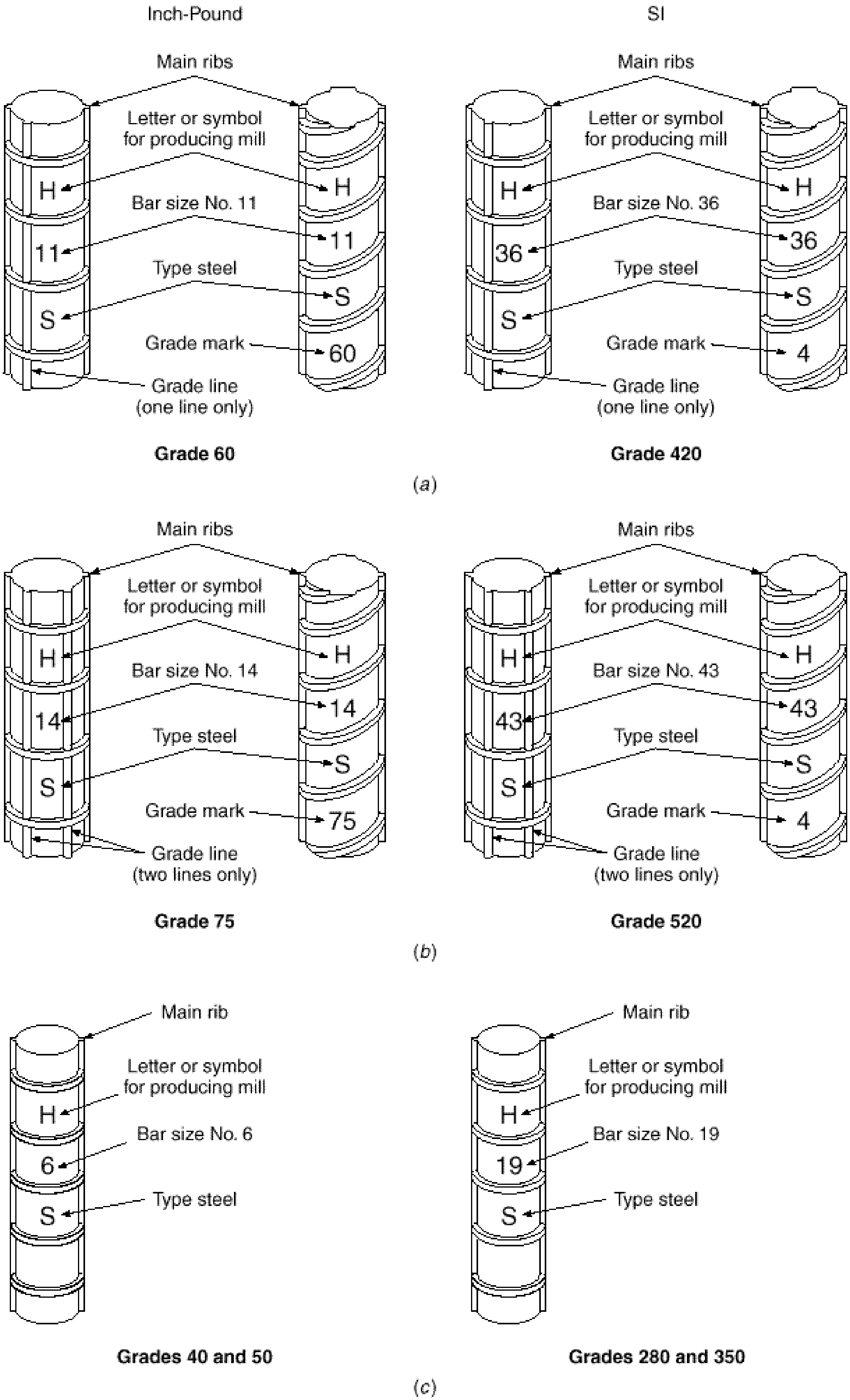

Inch-Pound
SI
Main ribs
Letter or symbol for producing mill
Bar size No. 11
Type steel
Grade mark
Grade line (one line only)
Grade 60
Bar size No. 36
Grade 420
(a)
Bar size No. 14
Grade line (two lines only)
Grade 75
Bar size No. 43
Grade 520
(b)
Main rib
Bar size No. 6
Grades 40 and 50
Bar size No. 19
Grades 280 and 350
(c)

FIGURE 2.14
Marking system for reinforcing bars meeting ASTM Specifications A 615, A 706, and A 996: (*a*) Grades 60 and 420; (*b*) Grades 75 and 520; (*c*) Grades 40, 50, 280, and 350. (*Adapted from Ref. 2.56.*) (Facing page.)

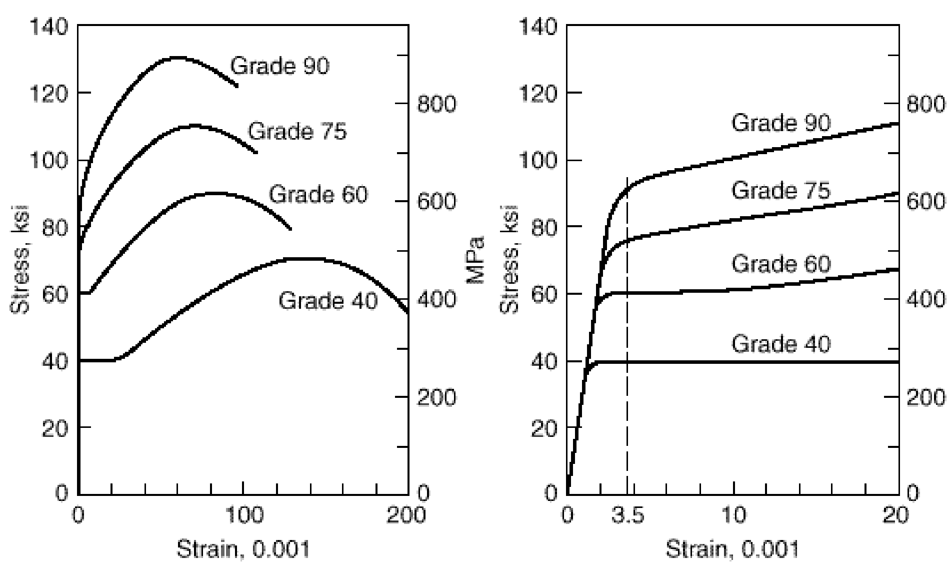
Low-carbon steels, typified by the Grade 40 curve, show an elastic portion followed by a *yield plateau*, i.e., a horizontal portion of the curve where strain continues to increase at constant stress. For such steels, the yield point is that stress at which the yield plateau establishes itself. With further strains, the stress begins to increase again, though at a slower rate, a process that is known as *strain-hardening*. The curve flattens out when the *tensile strength* is reached; it then turns down until fracture occurs. Higher-strength carbon steels, e.g., those with 60 ksi yield stress or higher, either have a yield plateau of much shorter length or enter strain-hardening immediately without any continued yielding at constant stress. In the latter case, the ACI Code specifies that the yield stress f_y be the stress corresponding to a strain of 0.0035, as shown in Fig. 2.15. Low alloy, high-strength steels rarely show any yield plateau and usually enter strain-hardening immediately upon beginning to yield.

c. Fatigue Strength

In highway bridges and some other situations, both steel and concrete are subject to large numbers of stress fluctuations. Under such conditions, steel, just like concrete (Section 2.8c), is subject to *fatigue*. In metal fatigue, one or more microscopic cracks form after cyclic stress has been applied a significant number of times. These fatigue cracks occur at points of stress concentrations or other discontinuities and gradually increase with increasing numbers of stress fluctuations. This reduces the remaining uncracked cross-sectional area of the bar until it becomes too small to resist the applied force. At this point the bar fails in a sudden, brittle manner.

For reinforcing bars it has been found (Refs. 2.27 and 2.57) that the fatigue strength, i.e., the stress at which a given stress fluctuation between f_{max} and f_{min} can be applied 2 million times or more without causing failure, is practically independent of the grade of steel. It has also been found that the stress range, i.e., the algebraic difference between maximum and minimum stress, $f_1 = f_{max} - f_{min}$, that can be sustained without fatigue failure depends on f_{min}. Further, in deformed bars the degree of stress

FIGURE 2.15
Typical stress-strain curves for reinforcing bars.



concentration at the location where the rib joins the main cylindrical body of the bar tends to reduce the safe stress range. This stress concentration depends on the ratio r/h, where r is the base radius of the deformation and h its height. The radius r is the transition radius from the surface of the bar to that of the deformation; it is a fairly uncertain quantity that changes with roll wear as bars are being rolled.

On the basis of extensive tests (Ref. 2.57), the following formula has been developed for design:

$$f_r = 21 - 0.33 f_{min} + 8\frac{r}{h} \tag{2.10}$$

where f_r = safe stress range, ksi
f_{min} = minimum stress; positive if tension, negative if compression
r/h = ratio of base radius to height of rolled-on deformation (in the common situation where r/h is not known, a value of 0.3 may be used)

Where bars are exposed to fatigue regimes, stress concentrations such as welds or sharp bends should be avoided since they may impair fatigue strength.

d. Coated Reinforcing Bars

Galvanized or epoxy-coated reinforcing bars are often specified in order to minimize corrosion of reinforcement and consequent spalling of concrete under severe environmental conditions, such as in bridge decks or parking garages subject to deicing chemicals, port and marine structures, and wastewater treatment plants.

ASTM A 767, "Standard Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement," includes requirements for the zinc coating material, the galvanizing process, the class or weight of coating, finish and adherence of coating, and the method of fabrication. Bars are usually galvanized after cutting and bending. Supplementary requirements pertain to coating of sheared ends and repair of damaged coating if bars are fabricated after galvanizing.

Epoxy-coated bars, presently more widely used than galvanized bars, are governed by ASTM A 775, "Standard Specification for Epoxy-Coated Reinforcing Steel Bars," which includes requirements for the coating material, surface preparation prior to coating, method of application, and limits on coating thickness, and by ASTM A 934, "Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars." Under ASTM A 775, the coating is applied to straight bars in a production-line operation, and the bars are cut and bent after coating. Under ASTM A 934, bars are bent to final shape prior to coating. Cut ends and small spots of damaged coating are suitably repaired after fabrication. Extra care is required in the field to ensure that the coating is not damaged during shipment and placing and that repairs are made if necessary.

2.15 WELDED WIRE REINFORCEMENT

Apart from single reinforcing bars, *welded wire reinforcement* (also described as *welded wire fabric*) is often used for reinforcing slabs and other surfaces, such as shells, and for shear reinforcement in thin beam webs, particularly in prestressed beams. Welded wire reinforcement consists of sets of longitudinal and transverse cold-drawn steel wires at right angles to each other and welded together at all points of intersection. The size and spacing of wires may be the same in both directions or may be different, depending on the requirements of the design.

The notation used to describe the type and size of welded wire fabric involves a letter-number combination. ASTM uses the letter "W" to designate smooth wire and letter "D" to describe deformed wire. The number following the letter gives the cross-sectional area of the wire in hundredths of a square inch. For example, a W5.0 wire is a smooth wire with a cross-sectional area of 0.05 in^2. A W5.5 wire has a cross-sectional area of 0.055 in^2. D6.0 indicates a deformed wire with a cross-sectional area of 0.06 in^2. Welded wire fabric having a designation $4 \times 4 -$ W5.0 $\times$ W5.0 has wire spacings 4 in. in each way with smooth wire of cross-sectional area 0.05 in^2 in each direction. Sizes and spacings for common types of welded wire fabric and cross-sectional areas of steel per foot, as well as weight per 100 ft^2, are shown in Table A.12 of Appendix A.

ASTM Specifications A 185 and A 497 pertain to smooth and deformed welded wire fabric respectively, as shown in Table 2.3. Because the yield stresses shown are specified at a strain of 0.005, the ACI Code requires that f_y be taken equal to 60 ksi unless the stress at a strain of 0.0035 is used.

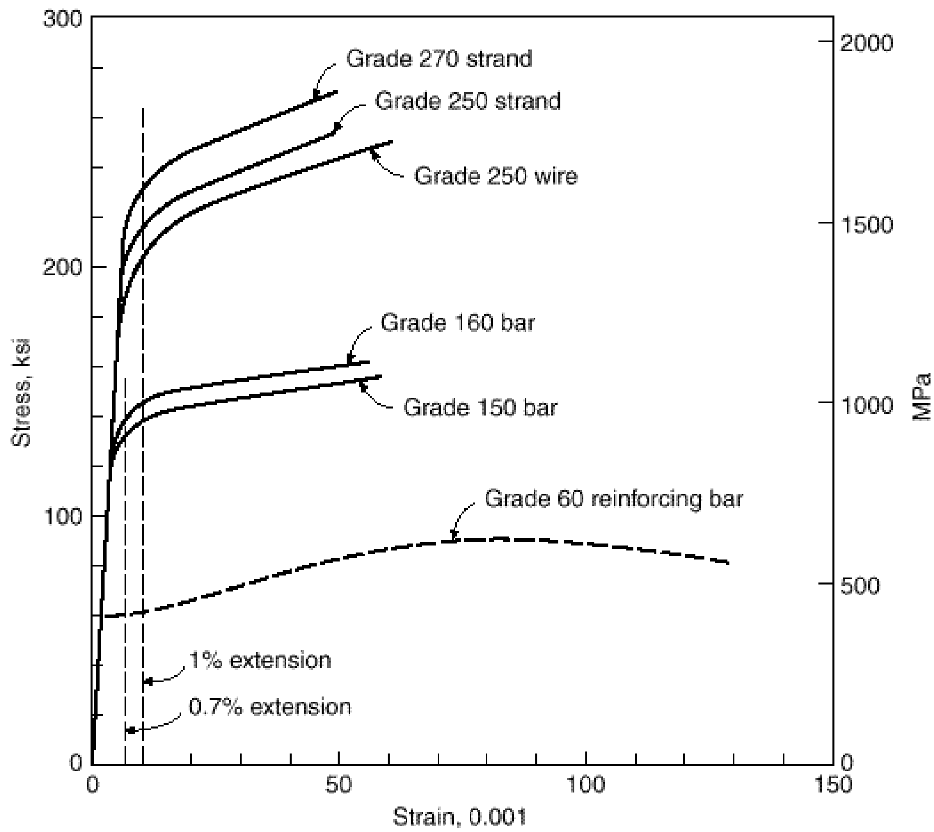
2.16 PRESTRESSING STEELS

Prestressing steel is used in three forms: round wires, stranded cable, and alloy steel bars. Prestressing wire ranges in diameter from 0.192 to 0.276 in. It is made by cold-drawing high-carbon steel after which the wire is stress-relieved by heat treatment to produce the prescribed mechanical properties. Wires are normally bundled in groups of up to about 50 individual wires to produce prestressing tendons of the required strength. Stranded cable, more common than wire in U.S. practice, is fabricated with six wires wound around a seventh of slightly larger diameter. The pitch of the spiral winding is between 12 and 16 times the nominal diameter of the strand. Strand diameters range from 0.250 to 0.600 in. Alloy steel bars for prestressing are available in diameters from 0.750 to 1.375 in. as plain round bars and from 0.625 to 2.50 in. as deformed bars. Specific requirements for prestressing steels are found in ASTM A 421, "Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete"; ASTM A 416, "Standard Specification for Steel Strand, Uncoated Seven-Wire Stress-Relieved for Prestressed Concrete"; and ASTM A 722, "Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete." Table A.15 of Appendix A provides design information for U.S. prestressing steels.

a. Grades and Strengths

The tensile strengths of prestressing steels range from about 2.5 to 6 times the yield strengths of commonly used reinforcing bars. The grade designations correspond to the minimum specified tensile strength in ksi. For the widely used seven-wire strand, three grades are available: Grade 250 (f_{pu} = 250 ksi), Grade 270, and Grade 300, although the last is not yet recognized in ASTM A 421. Grade 270 strand is used most often. For alloy steel bars, two grades are used: the regular Grade 150 is most common, but special Grade 160 bars may be ordered. Round wires may be obtained in Grades 235, 240, and 250, depending on diameter.

FIGURE 2.16
Typical stress-strain curves for prestressing steels.



b. Stress-Strain Curves

Figure 2.16 shows stress-strain curves for prestressing wires, strand, and alloy bars of various grades. For comparison, the stress-strain curve for a Grade 60 reinforcing bar is also shown. It is seen that, in contrast to reinforcing bars, prestressing steels do not show a sharp yield point or yield plateau; i.e., they do not yield at constant or nearly constant stress. Yielding develops gradually, and in the inelastic range the curve continues to rise smoothly until the tensile strength is reached. Because well-defined yielding is not observed in these steels, the yield strength is somewhat arbitrarily defined as the stress at a total elongation of 1 percent for strand and wire and at 0.7 percent for alloy steel bars. Figure 2.16 shows that the yield strengths so defined represent a good limit below which stress and strain are fairly proportional, and above which strain increases much more rapidly with increasing stress. It is also seen that the spread between tensile strength and yield strength is smaller in prestressing steels than in reinforcing steels. It may further be noted that prestressing steels have significantly less ductility.

While the modulus of elasticity E_s for bar reinforcement is taken as 29,000,000 psi, the effective modulus of prestressing steel varies, depending on the type of steel (e.g., strand vs. wire or bars) and type of use, and is best determined by test or supplied by the manufacturer. For unbonded strand (i.e., strand not embedded in concrete), the modulus may be as low as 26,000,000 psi. For bonded strand, E_s is usually about 27,000,000 psi, while for smooth round wires E_s is about 29,000,000 psi, the same as for reinforcing bars. The elastic modulus of alloy steel bars is usually taken as $E_s = 27{,}000{,}000$ psi.

c. Relaxation

When prestressing steel is stressed to the levels that are customary during initial tensioning and at service loads, it exhibits a property known as *relaxation*. Relaxation is defined as the loss of stress in stressed material held at constant length. (The same basic phenomenon is known as creep when defined in terms of change in strain of a material under constant stress.) To be specific, if a length of prestressing steel is stressed to a sizable fraction of its yield strength f_{py} (say, 80 to 90 percent) and held at a constant strain between fixed points such as the ends of a beam, the steel stress f_p will gradually decrease from its initial value f_{pi}. In prestressed concrete members this stress relaxation is important because it modifies the internal stresses in the concrete and changes the deflections of the beam some time after initial prestress was applied.

The amount of relaxation varies, depending on the type and grade of steel, the time under load, and the initial stress level. A satisfactory estimate for ordinary stress-relieved strand and wires can be obtained from Eq. (2.11), which was derived from more than 400 relaxation tests of up to 9 years' duration:

$$\frac{f_p}{f_{pi}} = 1 - \frac{\log t}{10} \cdot \frac{f_{pi}}{f_{py}} - 0.55 \cdot \tag{2.11}$$

where f_p is the final stress after t hours, f_{pi} is the initial stress, and f_{py} is the nominal yield stress (Ref. 2.58). In Eq. (2.11), log t is to the base 10, and $f_{pi} \cdot f_{py}$ not less than 0.55; below that value essentially no relaxation occurs.

The tests on which Eq. (2.11) is based were carried out on round, stress-relieved wires and are equally applicable to stress-relieved strand. In the absence of other information, results may be applied to alloy steel bars as well.

Low-relaxation strand has replaced stress-relieved strand as the industry standard. According to ASTM A 416, such steel must exhibit relaxation after 1000 hours of not more than 2.5 percent when initially stressed to 70 percent of specified tensile strength and not more than 3.5 percent when loaded to 80 percent of tensile strength. For low-relaxation strand, Eq. (2.11) is replaced by

$$\frac{f_p}{f_{pi}} = 1 - \frac{\log t}{45} \cdot \frac{f_{pi}}{f_{py}} - 0.55 \cdot \tag{2.12}$$

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